Appendix E

Geology and Soils

Appendix E.1

Preliminary Geotechnical Engineering Investigation

GeoConcepts INC
Geology - Geotechnical Engineering

PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION

Proposed Five to Eight Story Building with One Level Subgrade Parking

Tract: Rancho Sausal Redondo, Lot: LT 38, Arb: 65
6136 West Manchester Avenue
8651 South La Tijera Boulevard
Westchester, CA

for

CV 6136 Manchester, LLC
c/o CityView
Attn: Stephan Roberts
1901 Avenue of the Stars, Suite 1900
Los Angeles, CA 90067

Project 6058

February 7, 2022

PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION

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APPENDICES

INTRODUCTION

This report presents the results of a Preliminary Geotechnical Engineering Investigation on a portion of the subject property. The proposed project will utilize the 2020 City of Los Angeles Building Code. 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 AC Martin.
- Review of seven 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.

Partner Engineering and Science prepared the topographic base map utilized in this investigation. Preliminary building plans were prepared by AC Martin and incorporated onto the base map for this investigation.

The scope of this investigation is limited to the project area explored as depicted on the Plot Map. This report has not been prepared for use by other parties or for purposes other than the proposed project. GeoConcepts, Inc. should be consulted to determine if additional work is required when our work is used by others or if the scope of the project has changed. If the project is delayed for more than one year, this office should be contacted to verify the current site conditions and to prepare an update report.

PROPOSED DEVELOPMENT

It is our understanding that the site will be developed with a five to eight story mixed use development with one to two levels of subgrade parking. Anticipated foundations will range from 8 to 10 kips per lineal foot and 1000-1300 kips for column foundations. The proposed development is depicted on the enclosed Plot Map and Cross Sections.

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

SITE DESCRIPTION

Location and Description

Access to the property is via Manchester Avenue from La Tijera Boulevard (see Location Map in Appendix I). The site is developed with a one-story commercial building and parking area.

Adjacent sites are developed with a tire shop to the east, bounded by Manchester Avenue to the north, bounded by Truxton Avenue to the west and bounded by La Tijera Boulevard to the south. Adjacent structures to the east are partially along the property line.

Drainage

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

Groundwater

The subsurface exploration encountered a water seep at a depth of 35 feet in one boring. It is anticipated that due to the presence of water in only one hole of the explorations, that the encountered water is a water seep and not a static groundwater level. The depth to groundwater, when encountered in the explorations, is only valid for the date of exploration. Based on the Seismic Hazard Zone Report by the California Geological Survey (formerly Division of Mines and Geology), the depth to historical high groundwater level is about 40 feet below the surface. Seasonal fluctuations of groundwater levels may occur by varying amounts of rainfall, irrigation and recharge.

FIELD EXPLORATION

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

The field exploration of the site was conducted on November 17 and 18, 2021. The geotechnical conditions were mapped by a representative of this office (refer to Exploration Logs). Subsurface exploration was performed by drill rig into the underlying earth materials. Explorations were excavated to a maximum depth of 45 feet. All explorations were backfilled and tamped upon completion of down-hole observation. However, some settlement within exploration areas should be anticipated.

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

Undisturbed and bulk samples representative of the earth materials were obtained and transported to our laboratory. Undisturbed Modified California (MC) samples were obtained within the explorations through the use of a thin-walled steel sampler with successive blows of a 140-pound drop hammer dropped thirty inches (30"). MC samples were retained in brass rings of two and one-half inches (2½") in diameter and one inch (1") in height. The samples were transported in moisture tight containers. The results of the laboratory testing and a summary of the test procedures are included within Appendix II.

SUMMARY OF FINDINGS

Previous Work

The northern portion subject site was previously graded with compacted fill placed under review of The Twining Laboratories in their report dated May 23, 1957. The materials were compacted to 95 percent of maximum density for the building areas and 90% for the driveway and paved areas based on AASHTO T99-49. The fill was placed for support of a commercial building in the northwestern portion of the lot. The fill was approved by the City of Los Angeles, Department of Building and Safety in a letter dated June 5, 1957.

The southern portion of the subject site was previously graded with compacted fill under the review of Enviropro, Inc., in their reports dated May 19, 1993, August 24, 1993, and October 21, 1993. Due to contamination of the onsite soils, soils were imported that consisted of silty sand and sand with silts and gravels. The materials were compacted to 95 percent of the maximum density, based on ASTM D1557-78. The fill was placed as backfill of removed gasoline and diesel tanks and pump stations of a demolished gas station. It should be noted that for deeper portions of the grading, slurry was utilized as backfill up to approximately 10 feet below grade. The bottom of the deepest excavation was about 36 feet and was a 5 by 5 foot square at the bottom that increases to a 20 by 20 square at a depth of 10 feet. The fill was approved by the City of Los Angeles, Department of Building and Safety in a letter dated November 2, 1993. The approximate location is depicted on the Plot Map.

The northern portion of the subject site was previously explored by Jerry Kovacs & Associates in their report dated January 20, 1997, to address the remodel of and additions to the onsite commercial building. Five test pits and three borings were excavated to a maximum depth of 20 feet. The site explorations generally encountered fill materials and alluvium. The earth materials encountered by the previous consultant are similar to the materials currently encountered. The report was reviewed by the City of Los Angeles, Department of Building and Safety and approved in a letter dated February 11, 1997.

The northern portion of the subject site was subsequently graded with under the review Jerry Kovacs & Associates . Fill was placed in the area of the addition to a maximum depth of 6 feet. The compacted fill placed utilized onsite materials. The compaction report, dated July 14, 1997, was reviewed by the City of Los Angeles, Department of Building and Safety and approved in a letter dated July 22, 1997.

The southern portion of the subject site was previously explored by Giles Engineering Associates, Inc. in their report dated June 11, 1998, to address a new Del Taco restaurant. Six borings were excavated to a maximum depth of (30) feet. The site explorations generally encountered fill materials and alluvium. The earth materials encountered by the previous consultant are similar to the materials currently encountered. The report was reviewed by the City of Los Angeles, Department of Building and Safety and approved in a letter dated July 21, 1998.

The southern portion of the subject site was subsequently graded under the review of Giles Engineering Associates, Inc. Fill was placed in the area of the proposed Del Taco restaurant to a maximum depth of 10 feet. The compacted fill placed utilized onsite materials and import soils. The compaction report, dated November 9, 1998, was reviewed by the City of Los Angeles, Department of Building and Safety and approved in a letter dated January 22, 1999.

Stratigraphy

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

Artificial Fill (Af)

Artificial fill was encountered on the subject site. Fill was encountered in all of the borings ranging with a thickness of 1 foot. Although, based on the obtained site development history noted above deeper fill is present. . Fill generally consists of sandy silt to 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 reddish brown to orangish brown, moderately dense to very dense, silty sand to sandy silt. These deposits were encountered within all of the exploratory borings.

Excavation Characteristics

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

Landslides

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

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

Seismic Hazards

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.

Holocene-Active Faults

The following active faults are capable of producing seismic waves (ground shaking) on the subject property. Recent publications have reclassified active faults as Holocene-active faults. A Holocene-active Fault as defined by Department of Conservation California Geological Survey (CGS) is one which has moved during the past 11,700 years. This age boundary is an absolute age (number of years before present) and is not a radiocarbon ^{14}C age determination, which requires calibration in order to derive an absolute age. The following faults are considered to be Holocene-active and therefore subject to the regulations under the AP Act.

The San Andreas Fault zone (13) is the dominant Holocene-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 widespread 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 (20) 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 (14) 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 (12) 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 (7) 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 (6) 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 Raymond fault (10) 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 Raymond fault has a minimum slip rate of 0.1 to 0.2 mm per year and is capable of generating an earthquake of Mw 6.0 to 7.0 (SCEC).

The Sierra Madre fault zone (17) is often divided into five main segments: Vasquez Creek fault, Clamshell fault, Sawpit Canyon fault, Duarte fault and the Cucamonga fault. 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 (15) 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 the major strike slip movement has become inactive and dip slip movement is active present.

Pre-Holocene Faults

Pre-Holocene faults are faults that have not moved in the past 11,700 years and thus do not meet the criteria of "Holocene-active fault" as defined in the A-P Act and State Mining and Geology Board (SMGB) regulations. This class of fault may be still capable of surface rupture but is not regulated under the A-P Act. Depending on available site-specific and regional data such as proximity to other Holocene-active faults, average recurrence, variability in recurrence, the timing of the most recent surface rupturing earthquake, and case studies from other surface rupturing earthquakes, the project geologist may, but is not required to, recommend setbacks. Engineered solutions can also be considered by a licensed engineer operating within his or her field of practice. The following faults may be capable of producing seismic waves (ground shaking) on the subject property.

The Santa Monica fault (11) extends east from the coastline in Pacific Palisades through Santa Monica and West Los Angeles and merges with the Hollywood fault. 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 (18) 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 (5) 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 (1) 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 (9) 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 Holocene-active and/or Pre-Holocene 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 a Holocene-active fault. A fault is a fracture in the crust of the earth along which rocks on one side have moved relative to those on the other side. No known Holocene-active fault is mapped on the subject site.

Ground Shaking

Ground shaking caused by an earthquake is likely to occur at the site during the lifetime of the development due to the proximity of several Holocene-active and Pre-Holocene faults. Generally, on a regional scale, quantitative predictions of ground motion values are linked to peak acceleration and repeatable acceleration, which are a response to earthquake magnitudes relative to the fault distance from the subject property. Southern California major earthquakes are generally the result of large-scale earth processes in which the Pacific plate slides northwestward relative to the North American plate at about 2 inches/year.

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

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

Tsunamis & Seiches

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

The 1964 Alaskan Earthquake produced sea waves of less than four feet in the Los Angeles Harbor. The 1960 Chilean Earthquake produced sea waves of about five feet at Redondo Beach. Little data exists to evaluate the potential for a local tsunami generated off the coast of Southern California. Historically, two documented tsunamis have been generated off the coast of Southern California. The 1812 Santa Barbara Earthquake was reported to generate (10) to

(12) foot high sea waves at Gaviota. The 1927 Point Arguello Ms 7.3 Earthquake produced run-up heights of (5) feet at Port San Luis.

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

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

Earthquake Induced Landslides

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

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

Liquefaction

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

A detailed subsurface analysis can be performed to determine the liquefaction potential on the subject site and provide recommendations to mitigate the effects of liquefaction. A proposal for a detailed analysis will be prepared if requested.

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.

Dry Sand Settlement

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

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

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

Earthquake Magnitude	Peak Horizontal Ground Acceleration	Groundwater Level During Testing	Groundwater Level During Earthquake
6.59 (10% probability of exceedance in 50 years)	0.582 ($2/3*PGA_m$)	40 feet	40 feet

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

Total Settlement (in)	Differential Settlement (in)
0.46	0.23

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.

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.

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.

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

4. Based upon field observations, laboratory testing and analysis, the alluvium found in the exploratory borings should possess sufficient strength to support the proposed mixed-use development.

RECOMMENDATIONS

Specific

1. The proposed five to eight story building with one to two levels subgrade parking should be supported on foundations embedded into alluvium at subgrade depth (about 15 to 25 feet deep). Foundations should be designed as outlined the Foundations section below.
2. A slurry backfill was utilized as backfill material for previously existing underground storage tanks. Foundations proposed in the area of the slurry backfill should be deepened through the slurry into the alluvium.
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 subgrade parking excavations, subgrade wall backfills, and foundation excavations. Minor removal and recompaction at the subgrade elevation may be required for slab support if the subgrade soils are disturbed during the excavation.

Flatland Grading

1. Prior to commencement of work, a pre-grading meeting shall be held. Participants at this meeting will consist of the contractor, the owner or his representative, and the soils engineer. The purpose of the meeting is to avoid misunderstanding of the recommendations set forth in this report that might cause delays in the project.

2. Prior to placement of fill, all vegetation, rubbish, and other deleterious material should be disposed of offsite. The proposed structures should be staked out in the field by a surveyor. This staking should, as a minimum, include areas for overexcavation, toes of slopes, tops of cuts, setbacks, and easements. All staking shall be offset from the proposed grading area at least five feet (5'). Line and grade verification is not provided by GeoConcepts, Inc.

3. The natural ground, which is determined to be satisfactory for the support of the filled ground, shall then be scarified to a depth of at least six inches (6") and moistened as required. The scarified ground should be compacted to at least 90 percent of the maximum laboratory density (ASTM D 1557).

4. The fill soils shall consist of materials approved by the project Soils Engineer or his representative. These materials may be obtained from the excavation areas and any other approved sources, and by blending soils from one or more sources. The material used shall be free from organic vegetable matter and other deleterious substances and shall not contain rocks greater than eight inches (8") in diameter nor of a quantity sufficient to make compaction difficult.

5. The approved fill material shall be placed in approximately level layers six inches (6") thick and moistened as required. Each layer shall be thoroughly mixed to attain uniformity of moisture in each layer.

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

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

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

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

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

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

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

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

Foundations

It is recommended that the proposed structure be founded into alluvium at subgrade depth (~15 feet).

The minimum continuous footing size is (24) inches wide and (24) inches deep into the alluvium, measured from the lowest adjacent grade. Continuous footings may be proportioned, using a bearing value of (3500) pounds per square foot. Column footings placed into the alluvium may be proportioned, using a bearing value of (4000) pounds per square foot, and should be a minimum of (2) feet in width and (24) inches deep, below the lowest adjacent grade. Bearing pressures are allowed to increase by 20% for every foot of depth up to a maximum bearing pressure of 5000 psf.

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

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

Lateral loads may be resisted by friction at the base of the foundations and by passive resistance within the alluvium. A coefficient of friction of (0.4) 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 square foot, with a maximum earth pressure of (3000) pounds per square foot. When combining passive and friction for resistance of lateral loads, the passive component should be reduced by one-third.

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

For the portion of the foundation system that will be at or near the onsite slurry backfill, the foundations shall bridge over the slurry or be deepened into the alluvium. Foundations founded into the slurry backfill will <u>not</u> be acceptable.

Settlement

Settlement of the proposed multistory building with one level of subgrade parking will occur. Settlement of (1/8) to (1/4) inches between walls, within 20 feet or less, of each other, and under similar loading conditions, are considered normal. Total settlement on the order of (3/4) inches should be anticipated.

Expansive Soils

Expansive soils were not encountered on the subject property that are anticipated to adversely 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.

Hydroconsolidation

Hydroconsolidation is settlement of soils that collapse when they become saturated. Hydroconsolidation potential is greatest at the subject site for the upper soils due to the potential of saturation from irrigation and rainfall. The amount of hydroconsolidation settlement of the upper soils can be reduced by proper maintenance of the subject site. Plumbing lines should maintained in leak free condition, site drainage should be maintained as outlined in the Drainage and Maintenance section above, and landscape watering should be kept to a minimum to reduce infiltration of moisture to the deeper soils. Hydroconsolidation can occur in deeper soils due to elevated groundwater levels. The depth to historic groundwater is greater than (40) feet at the subject site. Based upon the depth to the historic groundwater, hydroconsolidation of the deeper soils should not pose any significant hazard at the subject site. In addition, based on the depth of the proposed subgrade levels water from surface infiltration will not affect the bearing soils.

Excavations

Excavations ranging in vertical height up to 25 feet will be required for the subgrade parking excavations. Minor amounts of remedial grading, up to three feet, may be required at the base of the excavation due to possible disturbance of the subgrade soils during excavation. Conventional excavation equipment may be used to make these excavations. Excavations should expose alluvium. These soils are suitable for non-surcharged vertical excavations up to 5 feet. Excavations above 5 feet should be trimmed back at 1:1 (H:V) slope gradient 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 500 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.

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 encountered during exploration at a depth of 35 feet below grade. Therefore, it is anticipated that the proposed piles in excess of 35 feet in depth will encounter water. Piles placed below the water level will require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube having a diameter of not less than 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 ensure that the tip of the tremie tube is never raised above the surface of the concrete.

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

Vibrated Piles

Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to 1.4 the flange width. The allowable passive value may be doubled for isolated piles, spaced a minimum of 2 times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed alluvium. If a vibratory method of solider pile installation is utilized, predrilling may be performed prior to installation of the steel beams. If predrilling is performed, it is recommended that the bore diameter be at least 3 inches smaller than the largest dimension of the pile to prevent excessive loss in the frictional component of the pile capacity. Predrilling should not be conducted below the proposed excavation bottom.

If a vibratory method is utilized, the owner should be aware of the potential risks associated with vibratory efforts, which typically involve inducing settlement within the vicinity of the pile which could result in a potential for damage to existing improvements in the area. The level of vibration that results from the installation of the piles should not exceed a threshold where occupants of nearby structures are disturbed, despite higher vibration tolerances that a building may endure without deformation or damage. The main parameter used for vibration assessment is peak particle velocity in units of inch per second (in/sec). The acceptable range of peak particle velocity should be evaluated based on the age and condition of adjacent structures, as well as the tolerance of human response to vibration. Based on Table 19 of the *Transportation and Construction Induced Vibration Guidance Manual* (Caltrans 2004), a continuous source of vibrations (ex. vibratory pile driving) which generates a maximum peak particle velocity of 0.5 in/sec is considered tolerable for modern industrial / commercial buildings and new residential structures. The Client should be aware that a lower value may be necessary if older or fragile structures are in the immediate vicinity of the site.

Vibrations should be monitored and record with seismographs during pile installation to detect the magnitude of vibration and oscillation experienced by adjacent structures. If the vibrations exceed the acceptable range during installation, the shoring contractor should modify the installation procedure to reduce the values to within the acceptable range. Vibration monitoring is not the responsibility of the Geotechnical Engineer. If vibratory construction techniques will be implemented, it is recommended that qualified consultant be retained to provide site specific recommendations for vibration thresholds and monitoring.

Lagging

To develop the full lateral support, provisions should be implemented to assure firm contact between the lagging and the undisturbed earth materials. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the lagging to the earth materials. It is recommended that the lagging and slurry backfill be installed the same day as excavation.

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 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. Equivalent fluid pressures for the design of cantilevered shoring are presented in the following table:

Shoring Height (ft)	Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure
15 feet	24 pcf
25 feet	32 pcf

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

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. The maximum deflection shall not exceed one-half inch (1/2) inch at the top of the shored embankment where a structure is within 1:1 (h:v) plane projected up from the base of the excavation, and for a maximum lateral deflection of (1) inch provided there are no structures within a 1:1 (h:v) plane projected up from the base of excavation. It is estimated that the deflection could be on the order of one-half inch at the top of the shored embankment. If greater deflection occurs during construction, additional bracing may be necessary to minimize settlement of adjacent buildings and utilities in adjacent streets and alleys. If desired to reduce the deflection, a greater active pressure could be used in the shoring design. Where internal bracing is used, the rakers should be tightly wedged to minimize deflection. The proper installation of the raker braces and the wedging will be critical to the performance of the shoring.

Monitoring

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

Shoring Observations

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

Excavations Maintenance – Erosion Control

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

Open Excavations:

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

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

Retaining walls should be designed to resist an active earth pressure such as that exerted by compacted backfill. Retaining walls up to 25 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.

Drained Condition

Surface Slope of Retained Material Horizontal to Vertical	Active Equivalent Fluid Weight p.c.f.	At-Rest Pressure Fluid Weight p.c.f.
Level (15 Feet)	40	70
Level (25 Feet)	47	70

Un-drained (Hydrostatic) Condition

Surface Slope of Retained Material Horizontal to Vertical	Hydrostatic Active Equivalent Fluid Weight p.c.f.	Hydrostatic At-Rest Pressure Fluid Weight p.c.f.
Level (15 Feet)	80	100
Level (25 Feet)	85	100

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

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.

   As an alternative to the perforated pipe system, the drainage system may consist of rock pockets. The rock pockets should consist of a 1'x1'x1' of ¾" gravel spaced at a maximum of 8' on center. The weep hole pipe through the wall at each rock pocket should be a minimum 4" diameter. Where space does not permit a 1'x1'x1' gravel pocket (such as where space behind the wall is less than 12") the thickness of the gravel pocket may be reduced to minimum of 4" provided that H'xW'X4">1 cubic foot. A request for modification may be required by the City of Los Angeles for gravel pockets with the reduced thickness.

   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

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 25 feet in height may be designed per the following table. The seismic equivalent fluid pressure is in addition to static active earth pressures.

The seismic loading is based on a horizontal acceleration coefficient of ½ of 2/3 PGA_M = 0.29.

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

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.

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

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

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

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

Decking

Exterior decking slabs on grade should be reinforced with minimum #4 reinforcing bars, placed at (16) inches on center each way and supported on alluvium. Provisions for cracks should be incorporated into the design and construction of the decking. Concrete slabs should have sufficient control joints spaced at a maximum of approximately 8 feet. Decking planned adjacent to lawns, planters or adjacent to descending slopes should be provided with a 12-inch thickened edge. The deck reinforcement should be bent down into the edge. These recommendations are considered minimums unless superseded by the project structural engineer.

Paving

It is recommended that the existing fill materials be removed and recompacted to (95) percent of the maximum density for support of the proposed paving. In addition, the recommended removals should extend a minimum of one foot below the proposed paving section.

Concrete paving shall have a minimum thickness of 5 inches and shall be underlain by 4 inches of aggregate base. A subgrade modulus of 120 pounds per cubic inch may be assumed for design of concrete paving. Slabs on grade should be reinforced with minimum #4 reinforcing bars, placed at (16) inches on center each way. These recommendations are considered as minimum unless superceded by the structural engineer. For standard crack control maximum expansion joint spacing of 15 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended.

REVIEWS

Plan Review and Plan Notes

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

The final plans should reflect the following:

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

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

3. The project geotechnical engineer must review all grading.

4. The project geotechnical engineer shall review all foundations.

Construction Review

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

The property owner should take an active role in project safety by assigning responsibility and authority to individuals qualified in appropriate construction safety principles and practices. Generally, site safety should be assigned to the general contractor or construction manager that is in control of the site and has the required expertise, which includes but not limited to construction means, methods and safety precautions.

LIMITATIONS

General

This report is intended to be used only in its entirety. No portion or section of the report, by itself, is designed to completely represent any aspect of the project described herein. If any reader requires additional information or has questions regarding this report, GeoConcepts, Inc. should be contacted.

Subsurface conditions were interpreted on the basis of our field explorations and past experience. Although, between exploratory excavations, subsurface earth materials may vary in type, strength and many other properties from those interpreted. The findings, conclusions and recommendations presented herein are for the soil conditions encountered in the specific locations. Earth materials and conditions immediately adjacent to, or beneath those observed may have different characteristics, such as, earth type, physical properties and strength. Other soil conditions due to non-uniformity of the soil conditions or manmade alterations may be revealed during construction. If subsurface conditions differ from those encountered in the described exploration, this office should be advised immediately so that further recommendations may be made if required. If it is desired to minimize the possibility of such changes, additional explorations and testing can/should be performed.

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

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

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

CONSTRUCTION NOTICE

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

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

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

Respectfully submitted,
GEOCONCEPTS, INC.

Raffi Dermendjian
Project Engineer
PE C. 88261
RD: 6058-1

Distribution: (3) Addressee

APPENDIX I

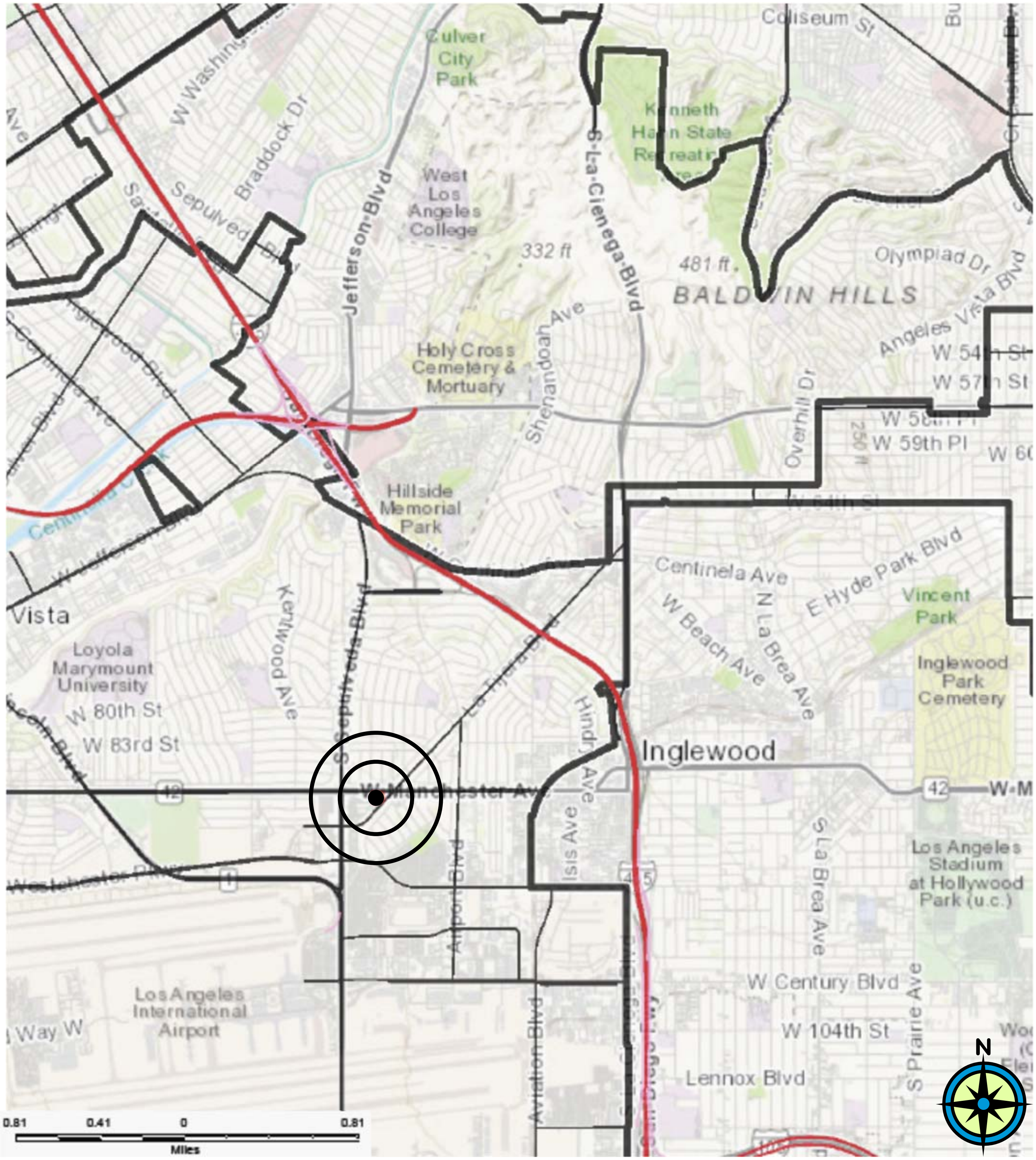
SITE INFORMATION

Location Map
Groundwater Map
Regional Geologic Map
USGS Fault Map
Earthquake Zone Map

Plot Map
Cross Sections

Field Exploration
Borings 1 through 7

LOCATION MAP

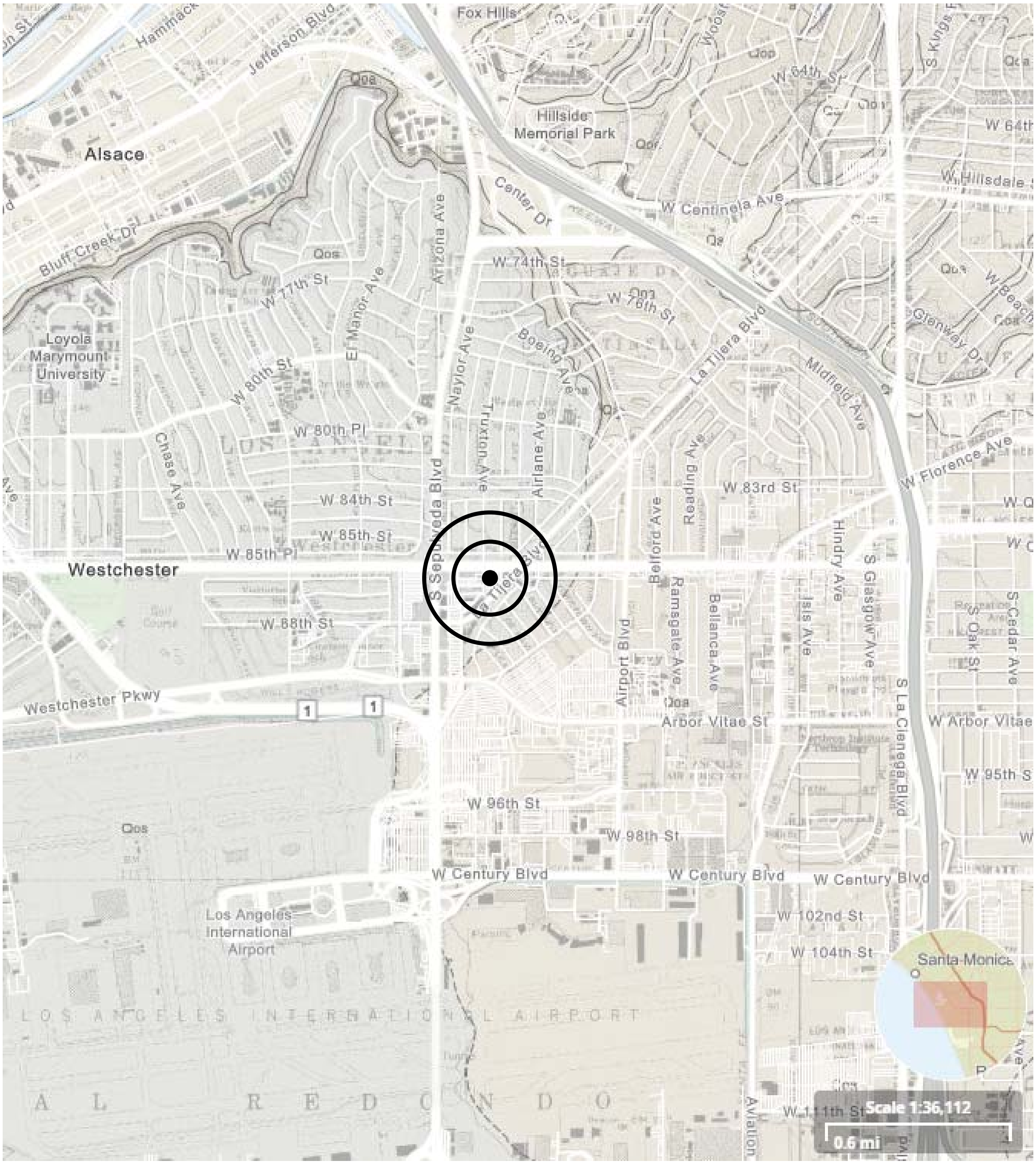


Reference:	City of Los Angeles, Navigate LA	Scale: As Shown

GROUNDWATER MAP

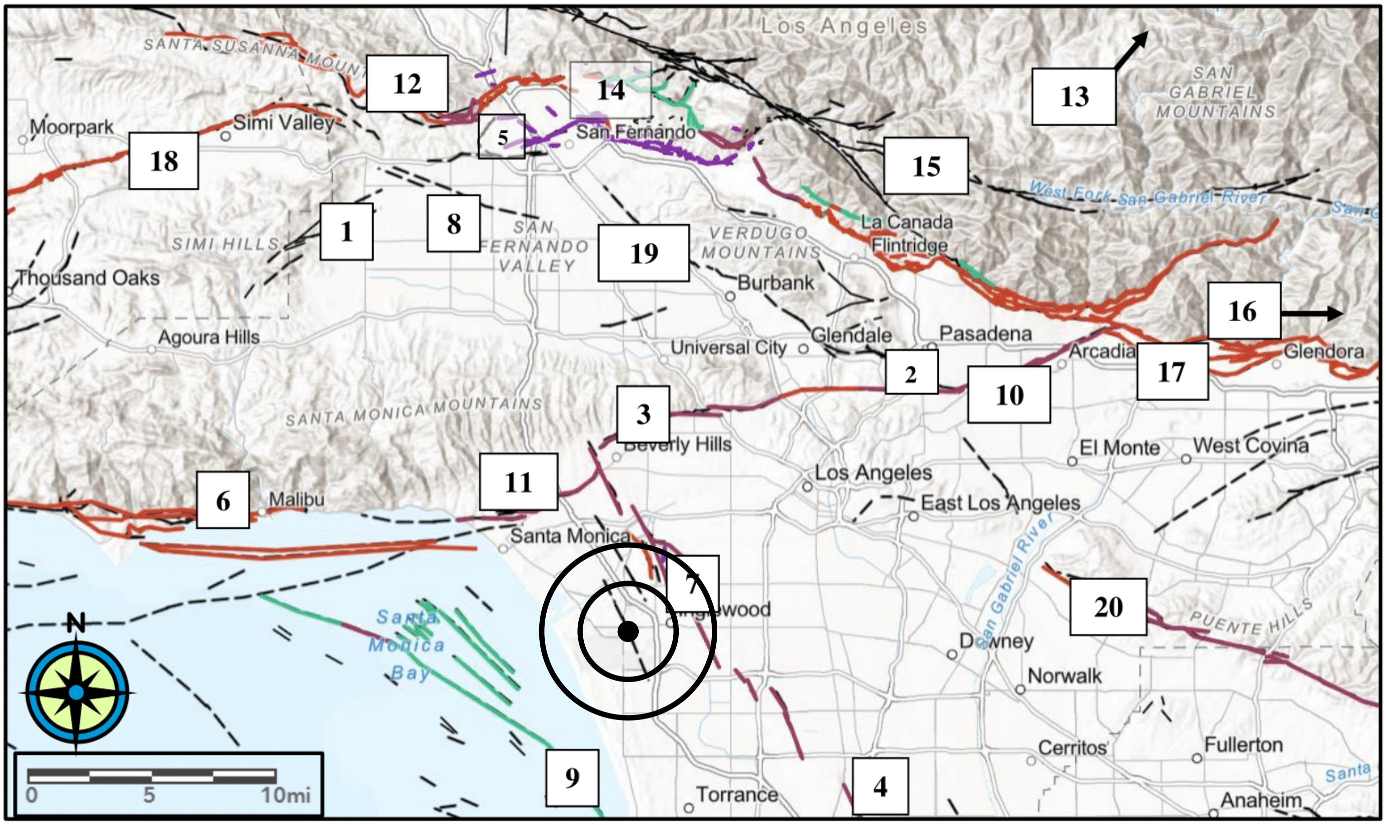
Reference:	State of California Seismic Hazard Report, Venice Quadrangle	Scale: As Shown

REGIONAL GEOLOGIC MAP



Reference:	Dibblee; Geologic Map of the Venice Quadrangle	Scale: As Shown

USGS FAULT MAP



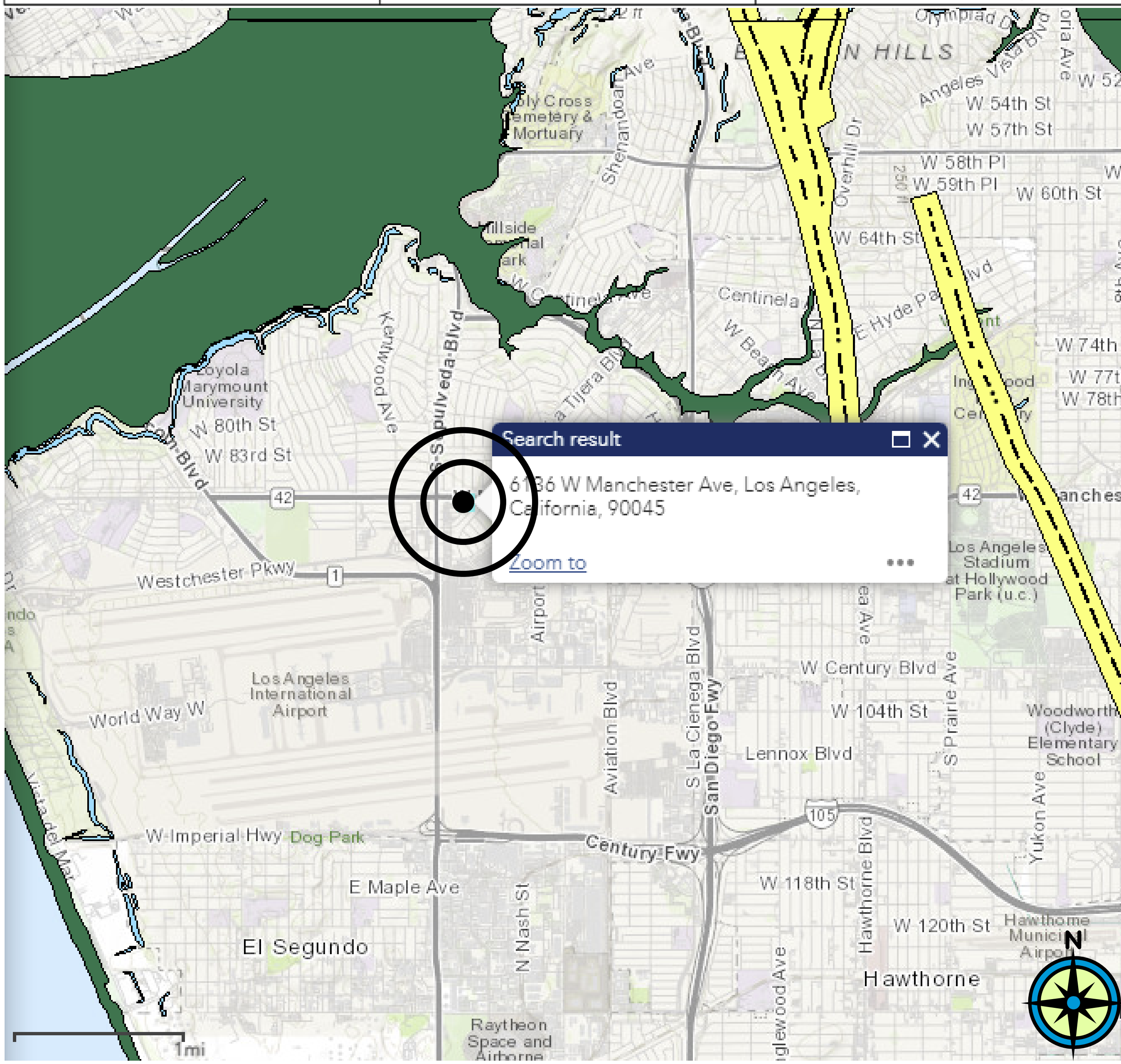
Historic High Magnitude Quaternary Fault Activity
Approximate Location
> 1.6 million years
> 750,000 years
> 130,000 years
> 15,000 years
> 150 years
Class B*
Unknown

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

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

EARTHQUAKE ZONE MAP

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



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

BORING: B-1

ADDRESS: 6136 Manchester Road | PROJECT NO.: 6058

DATE LOGGED: November 17, 2021 | LOGGED BY: CG

ATTITUDES (b - bedding, j - joint, s - shear, f - fault)	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	DESCRIPTION
					0	0.0' - 4.0" **CONCRETE,**
						4.0" - 1.0' **FILL; Af,** sandy silt, brown, moist, fine grained, pebbles
						1.0' - 41.0' **ALLUVIUM; Qal,** 1.0' - 25.0' silty sand, reddish brown to orangish brown, moist, fine to medium grained
	6	119	43	X	5	
	4	116	37	X	10	
	4	118	57	X	15	
	5	115	74	X	20	
	9	111	38	X	25	25.0' - 30.0' clayey sand, reddish brown, slightly moist to moist, fine grained
	9	118	47	X	30	30.0' - 45.0' silty sand to sand, orangish brown to reddish brown, slightly moist to wet, fine to medium grained
	10	116	60	X	35	@35.0' moist to wet, wet at tip
50 blow for 4 inches	7	117	79	X	40	@40.0' moist to wet, wet at tip @40.0' - 45.0' silty sand to sand, orangish brown to reddish brown, medium grained, based on cuttings
					45	Total Depth - 45.0 Feet Water Seepage - 35.0 Feet 8 Inch Hollow Stem Auger with Autohammer
					50	

BORING: B-2

ADDRESS: 6136 Manchester Road | PROJECT NO.: 6058

DATE LOGGED: November 17, 2021 | LOGGED BY: CG

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

ATTITUDES	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	GRAPHIC LOG	DESCRIPTION
					0		0.0' - 4.0" **ASPHALT,**
							4.0" - 1.0' **FILL; Af,** sandy silt, brown, moist, fine grained, pebbles
							1.0' - 38.5' **ALLUVIUM; Qal,**
	6	106	14				1.0' - 38.5' silty sand, reddish brown to orangish brown, slightly moist to moist, fine to medium grained
					5		
	6	114	41				@7.5' - 22.5' mottled
					10		
	6	116	59				
					15		
50 blows for 4 inches	8	122	50				
					20		
	8	115	54				@22.5' - 38.5' orangish brown to yellowish brown, moist, medium grained
					25		
50 blows for 6 inches	8	114	50				
					30		
	5	118	70				@32.5' - 38.5' mottled
					35		
50 blows for 3 inches	3	114	50				
							Total Depth - 38.5 Feet No Groundwater 8 Inch Hollow Stem Auger with Autohammer
					40		
					45		
					50		

BORING: B-3

ADDRESS: 6136 Manchester Road | PROJECT NO.: 6058

DATE LOGGED: November 17, 2021 | LOGGED BY: CG

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

ATTITUDES	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	DESCRIPTION
					0	0.0' - 4.0" **ASPHALT,**
						4.0" - 1.0' **FILL; Af,** sandy silt, brown, moist, fine grained, pebbles
						1.0' - 38.5' **ALLUVIUM; Qal,**
	7	104	8	X	5	1.0' - 10.0' sandy silt, brown, slightly moist, fine grained
	6	118	48	X	10	10.0' - 41.0' silty sand, reddish brown to orangish brown, slightly moist to wet, fine to medium grained
	7	113	65	X	15	
50 blows for 5 inches	8	118	50	X	20	
	7	112	55	X	25	
50 blows for 5 inches	8	116	77	X	30	
	7	117	50	X	35	
50 blows for 3 inches	4	114	50	X	40	@40.0' moist to wet, mottled
					45	
					50	

Total Depth - 41.0 Feet
No Groundwater
8 Inch Hollow Stem Auger with Autohammer

BORING: B-4

ADDRESS: 6136 Manchester Road | PROJECT NO.: 6058

DATE LOGGED: November 17, 2021 | LOGGED BY: CG

ATTITUDES (b - bedding, j - joint, s - shear, f - fault)	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	DESCRIPTION
					0	0.0' - 4.0" **ASPHALT,**
						4.0" - 1.0' **FILL; Af,** sandy silt, brown, moist, fine grained, pebbles
						1.0' - 38.5' **ALLUVIUM; Qal,**
	4	114	51	X	5	1.0 - 10.0' sandy silt, medium brown, slightly moist, fine grained
	4	116	43	X	10	10.0' - 41.0' silty sand, reddish brown, slightly moist to moist, fine to medium grained
	6	114	68	X	15	@15.0' - 25.0' mottled
	3	116	69	X	20	
	7	113	37	X	25	
	7	118	45	X	30	
	6	117	50	X	35	
50 blows for 5 inches	3	114	50	X	40	@40.0' orangish brown and yellowish brown, mottled
						Total Depth - 41.0 Feet No Groundwater 8 Inch Hollow Stem Auger with Autohammer
					45	
					50	

BORING: B-5

ADDRESS: 6136 Manchester Road — PROJECT NO.: 6058

DATE LOGGED: November 18, 2021 — LOGGED BY: CG

ATTITUDES (b - bedding, j - joint, s - shear, f - fault)	WATER CONTENT, %	UNIT DRY WEIGHT, PCF	BLOWS/FOOT	SAMPLES	DEPTH, FT	DESCRIPTION
					0	0.0' - 4.0" **ASPHALT,**
						4.0" - 1.0' **FILL; Af,** sandy silt, medium brown, moist, fine grained, pebbles
						1.0' - 41.0' **ALLUVIUM; Qal,**
						1.0' - 15.0' silty sand, reddish brown to orangish brown, mottled, slightly moist, fine to medium grained
	5	111	49	X	5	
	6	115	60	X	10	
50 blows for 5 inches	5	117	50	X	15	15.0' - 25.0' silty sand, yellowish brown and orangish brown, mottled, slightly moist, fine grained
50 blows for 5 inches	3	116	50	X	20	
50 blows for 4 inches	5	118	93	X	25	25.0' - 41.0' silty sand, light brown to yellowish brown, slightly moist to moist, medium grained
50 blows for 6 inches	8	113	50	X	30	
50 blows for 5 inches	4	113	50	X	35	
50 blows for 4 inches	3	116	50	X	40	
					41	Total Depth - 41.0 Feet No Groundwater 8 Inch Hollow Stem Auger with Autohammer
					45	
					50	

BORING: B-6

ADDRESS: 6136 Manchester Road — PROJECT NO.: 6058

DATE LOGGED: November 18, 2021 — LOGGED BY: CG

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' - 4.0" **ASPHALT,**
							4.0" - 1.0' **FILL; Af,** sandy silt, medium brown, moist, fine grained, pebbles
	2	115	76		5		1.0' - 41.0' **ALLUVIUM; Qal,** 1.0' - 41.0' silty sand, slightly moist, reddish brown to orangish brown, fine to medium grained
	6	119	74		10		
50 blows for 5 inches	2	116	50		15		@15.0' - 35.0' reddish brown to orangish brown
50 blows for 4 inches	3	122	96		20		
50 blows for 4 inches	4	120	94		25		
50 blows for 5 inches	3	119	94		30		
50 blows for 5 inches	6	111	50		35		@35.0' - 41.0' orangish brown
50 blows for 5 inches	4	118	50		40		
							Total Depth - 41.0 Feet No Groundwater 8 Inch Hollow Stem Auger with Autohammer
					45		
					50		

BORING: B-7

ADDRESS: 6136 Manchester Road | PROJECT NO.: 6058

DATE LOGGED: November 18, 2021 | LOGGED BY: CG

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' - 4.0" **ASPHALT,**
							4.0" - 1.0' **FILL; Af,** sandy silt, medium brown, moist, fine grained, pebbles
							1.0' - 41.0' **ALLUVIUM; Qal,**
	4	119	86		5		1.0' - 10.0' silty sand to sandy silt, reddish brown to orangish brown, slightly moist, fine grained
	5	113	66		10		10.0' - 40.0' silty sand, orangish brown, slightly moist to moist, fine to medium grained
50 blows for 4 inches	4	119	89		15		
50 blows for 4 inches	3	114	50		20		
50 blows for 6 inches	6	115	50		25		
50 blows for 4 inches	2	113	50		30		
50 blows for 5 inches	3	110	50		35		
	21	94	48		40		40.0' - 41.0' clayey silt to silty clay, slightly moist, grayish green, fine grained, mottled
					45		Total Depth - 41.0 Feet No Groundwater 8 Inch Hollow Stem Auger with Autohammer
					50		

APPENDIX II

LABORATORY TESTING

Laboratory Procedures

Laboratory Recapitulation 1
Laboratory Recapitulation 2

Figures S.1 through S.5
Figures C.1 through C.24

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.

Expansion Index Tests

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

Grain Size Analysis

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

pH (CTM 643)

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

Minimum Resistivity (CTM 643)

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

Chloride Content (CTM 422)

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

Sulfate Content (CTM 417)

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

LABORATORY RECAPITULATION 1 PROJECT: 6136 Manchester Road PROJECT NO.: 6058						
Exploration	**Depth (ft)**	**Material**	**Dry Density In Situ (P.C.F.)**	**Moisture Content (%)**	**Cohesion (K.S.F.)**	**Friction Angle (degree)**
B-1	1	Af				
B-1	5	Qal	119.4	5.6	0.2	33
B-1	10	Qal	115.7	3.6		
B-1	15	Qal	117.8	4.2		
B-1	20	Qal	115	5.3		
B-1	25	Qal	111.4	8.7		
B-1	30	Qal	117.8	8.7		
B-1	35	Qal	115.5	9.8		
B-1	40	Qal	116.6	6.7		
B-2	2.5	Qal	106.4	6		
B-2	7.5	Qal	114.4	5.9		
B-2	12.5	Qal	116.1	5.7		
B-2	17.5	Qal	121.6	7.9		
B-2	22.5	Qal	115.1	7.7		
B-2	27.5	Qal	113.8	7.9		
B-2	32.5	Qal	118	5.5		
B-2	37.5	Qal	114	3.3		
B-3	5	Qal	104.2	6.6		
B-3	10	Qal	118.3	6.4		
B-3	15	Qal	113.5	6.7	0.225	32
B-3	20	Qal	118.2	7.9		
B-3	25	Qal	112.4	6.6		
B-3	30	Qal	116.5	7.6		
B-3	35	Qal	117.4	7.4		
B-3	40	Qal	114.1	4		
B-4	5	Qal	114.2	3.8		
B-4	10	Qal	115.7	4	0.225	31
B-4	15	Qal	113.8	6.4		
B-4	20	Qal	116.4	3.3		
B-4	25	Qal	112.7	7.4		
B-4	30	Qal	117.9	6.6		
B-4	35	Qal	116.8	6		
B-4	40	Qal	114.2	3.4		
B-5	5	Qal	111.3	5.1		
B-5	10	Qal	114.9	5.7	0.2	32
B-5	15	Qal	117.2	4.8		
B-5	20	Qal	116.2	2.8		
B-5	25	Qal	118.1	4.7		

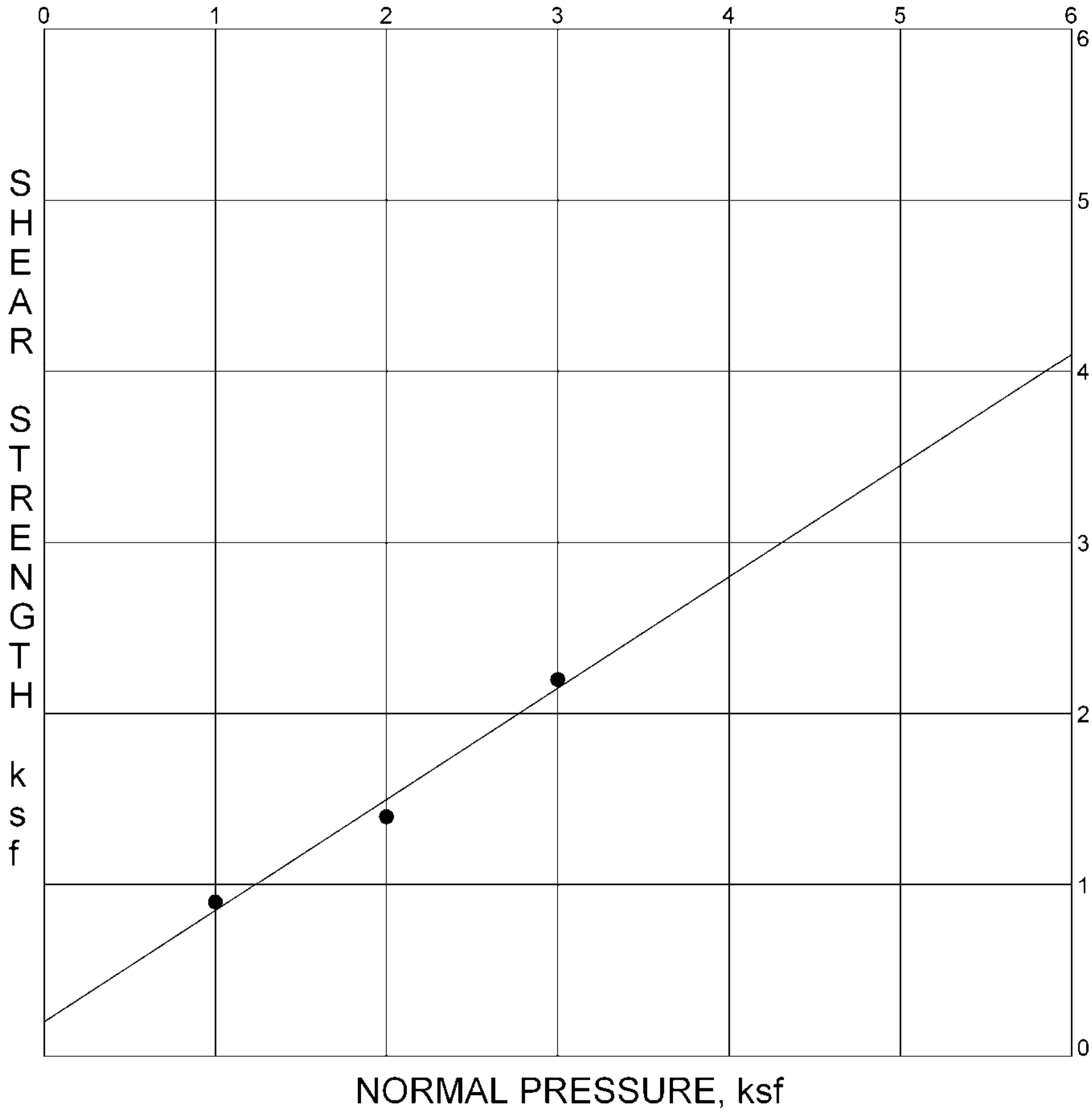
B-5	30	Qal	113.2	7.7		
B-5	35	Qal	112.8	4.4		
B-5	40	Qal	116.3	2.7		
B-6	5	Qal	115.4	2	0.25	27
B-6	10	Qal	119.3	5.7		
B-6	15	Qal	115.6	1.9		
B-6	20	Qal	122	2.8		
B-6	25	Qal	120.4	4		
B-6	30	Qal	118.8	3.3		
B-6	35	Qal	111.2	5.9		
B-6	40	Qal	118.2	3.8		
B-7	5	Qal	119.2	3.7		
B-7	10	Qal	112.6	4.5		
B-7	15	Qal	118.5	3.6		
B-7	20	Qal	113.6	2.9		
B-7	25	Qal	115.4	6.4		
B-7	30	Qal	113	2		
B-7	35	Qal	110.4	2.8		
B-7	40	Qal	94	20.9		

LABORATORY RECAPITULATION 2
PROJECT: 6136 Manchester Road
PROJECT NO.: 6058

Exploration	Depth (ft)	pH	As-Is Soil Resistivity (ohm-cm)	Minimum Soil Resistivity (ohm-cm)	Chloride (%)	Sulphate (%)	Expansion Index
B-1	1	8.9	43000	2700	0.001	0.016	
B-5	10	8.96	22000	4600	0.001	0.001	
B-5	15						24

PROJECT LOCATION: 6136 Manchester Road | PROJECT NO.: 6058

SAMPLE LOCATION: B-1 @ 5.0 | DESCRIPTION: Qal



Test Results

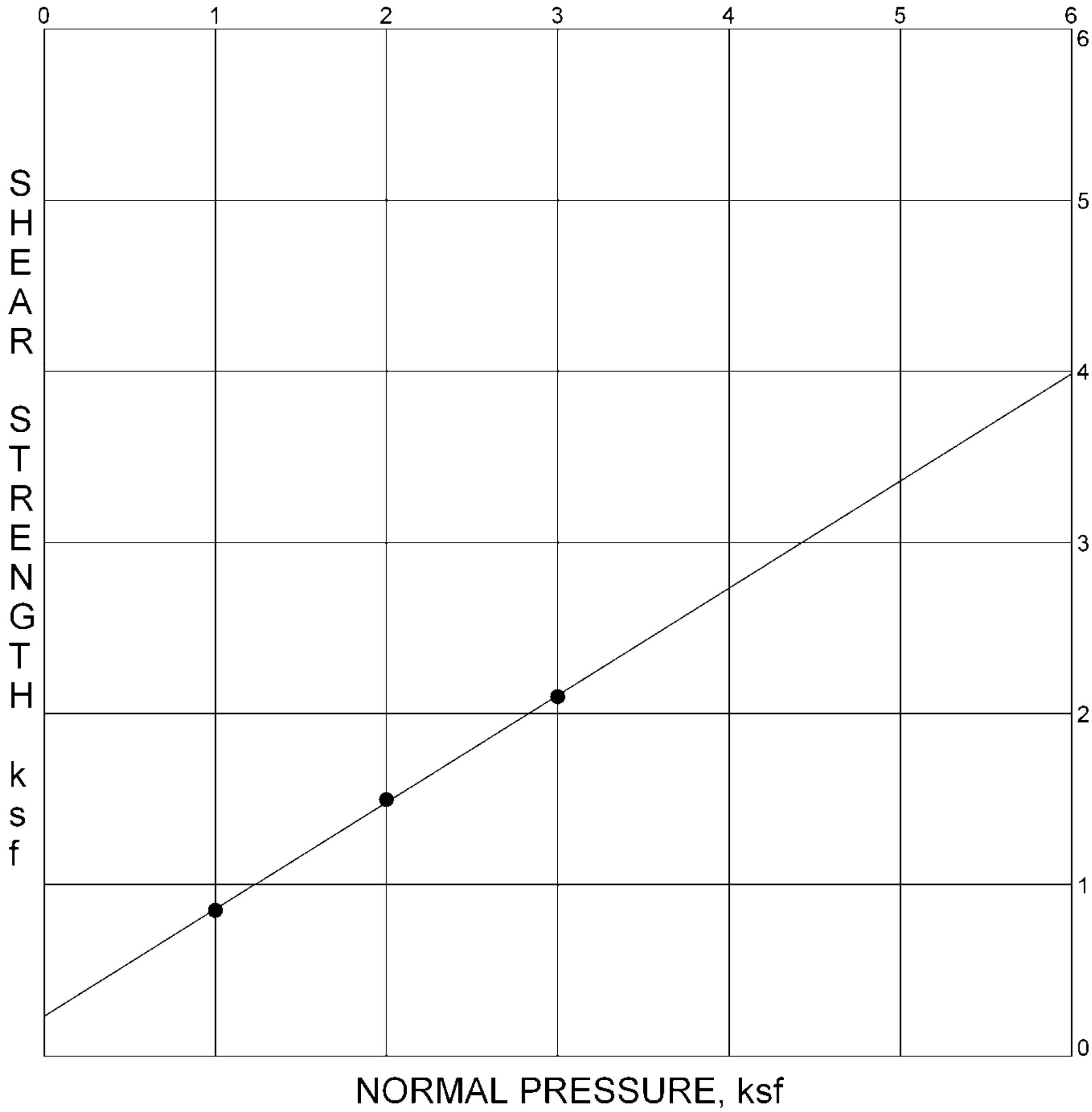
Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 5.6	Dry Density: 119.4	Phi (deg): 33.0
Saturated: 15.2		Cohesion (ksf): 0.200

SHEAR TEST DIAGRAM

Figure S.1

PROJECT LOCATION: 6136 Manchester Road | PROJECT NO.: 6058

SAMPLE LOCATION: B-3 @ 15.0 | DESCRIPTION: Qal



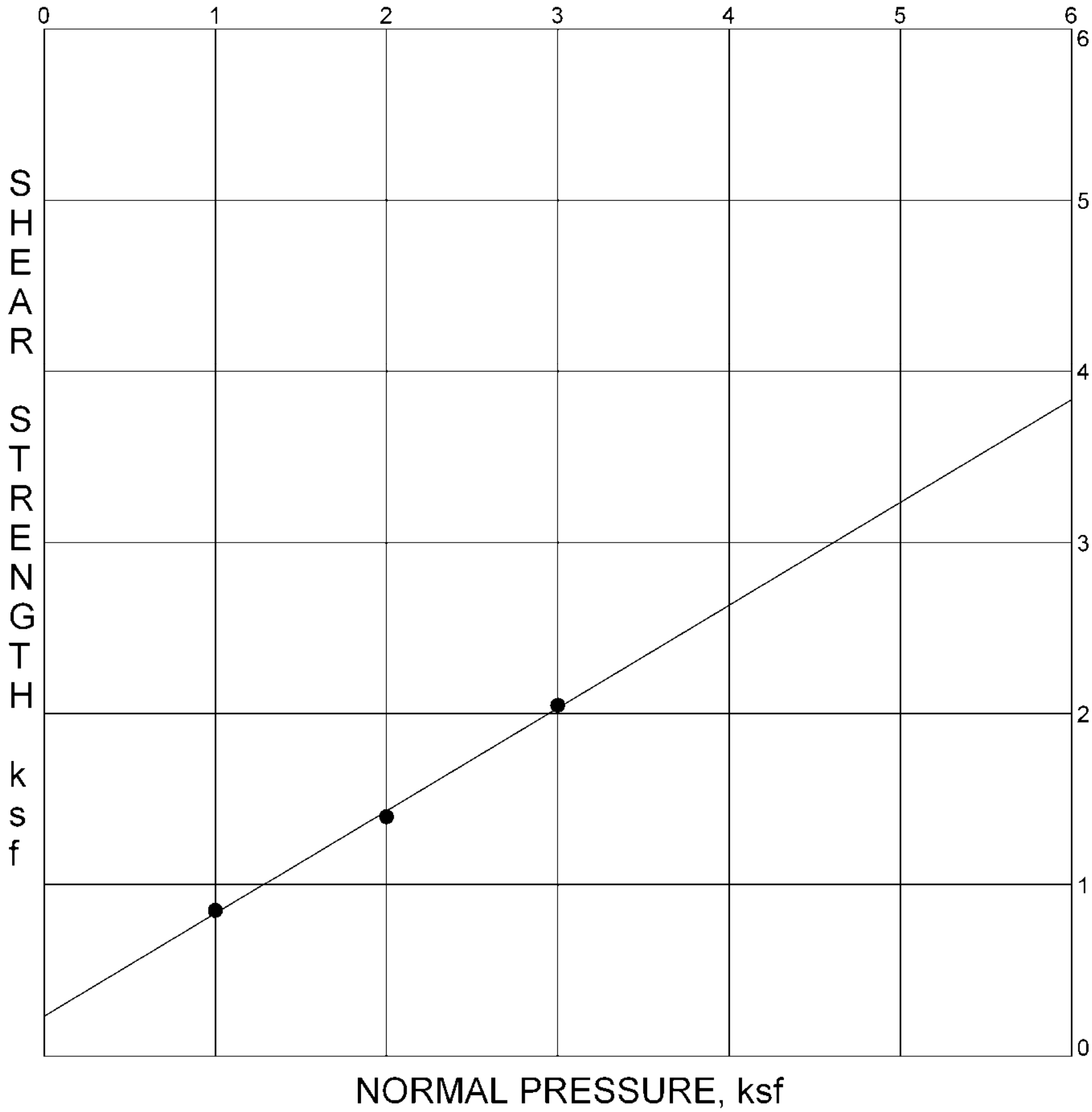
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 6.7	Dry Density: 113.5	Phi (deg): 32.0
Saturated: 17.9		Cohesion (ksf): 0.225

SHEAR TEST DIAGRAM

Figure S.2

PROJECT LOCATION: 6136 Manchester Road	PROJECT NO.: 6058
SAMPLE LOCATION: B-4 @ 10.0	DESCRIPTION: Qal



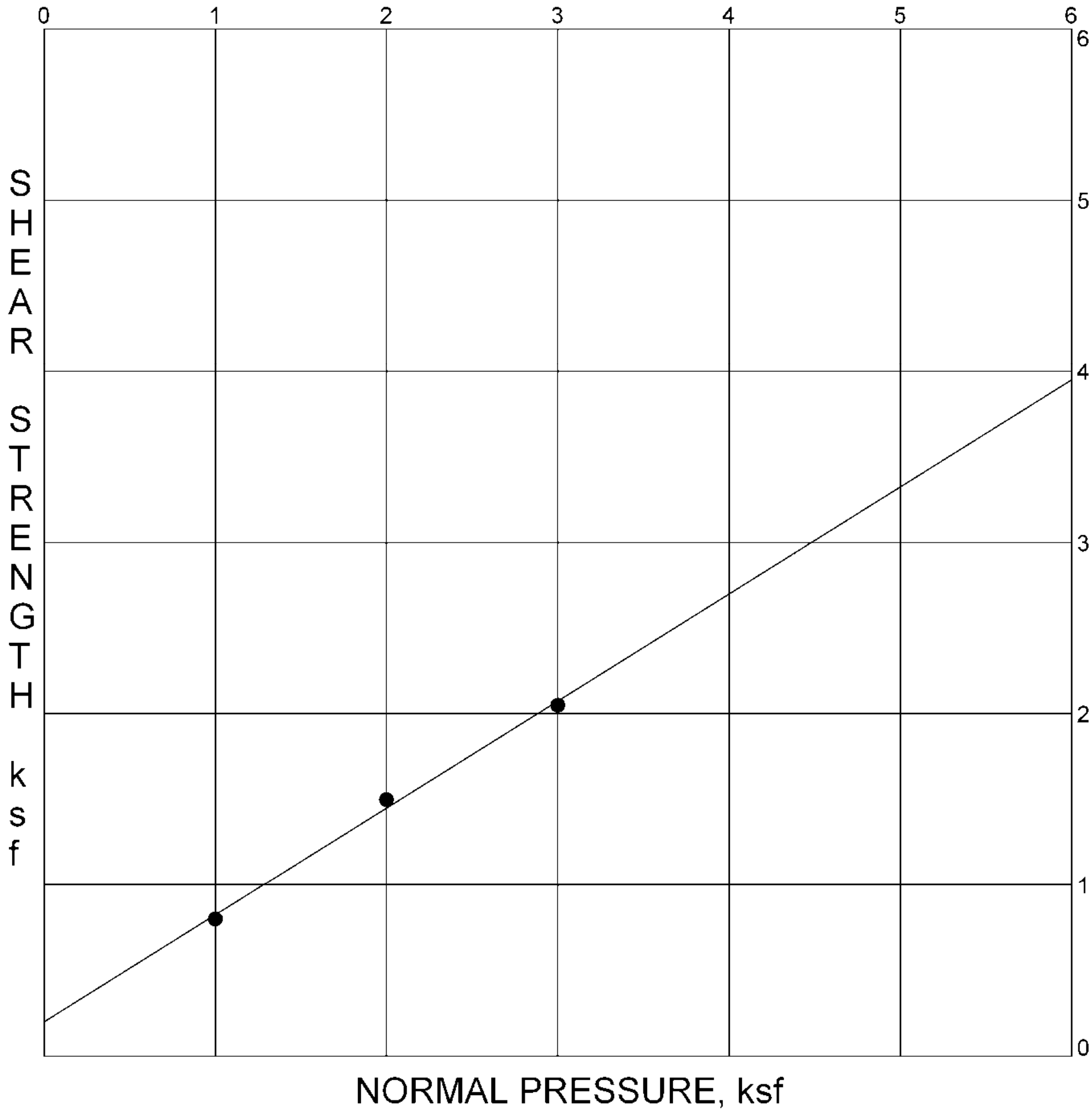
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 4.0	Dry Density: 115.7	Phi (deg): 31.0
Saturated: 16.9		Cohesion (ksf): 0.225

SHEAR TEST DIAGRAM

Figure S.3

PROJECT LOCATION: 6136 Manchester Road	PROJECT NO.: 6058
SAMPLE LOCATION: B-5 @ 10.0	DESCRIPTION: Qal



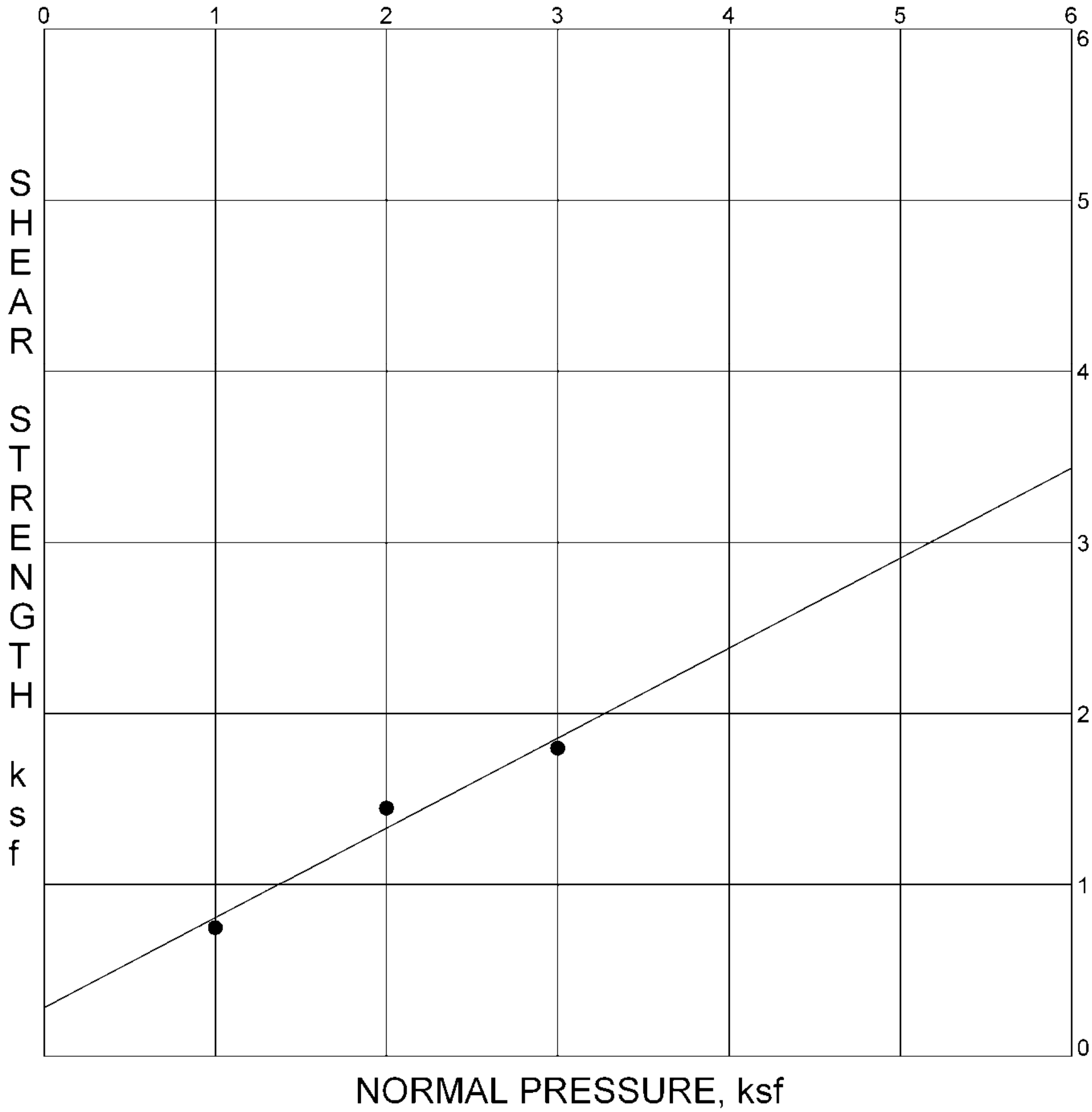
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 5.7	Dry Density: 114.9	Phi (deg): 32.0
Saturated: 17.2		Cohesion (ksf): 0.200

SHEAR TEST DIAGRAM

Figure S.4

PROJECT LOCATION: 6136 Manchester Road	PROJECT NO.: 6058
SAMPLE LOCATION: B-6 @ 5.0	DESCRIPTION: Qal



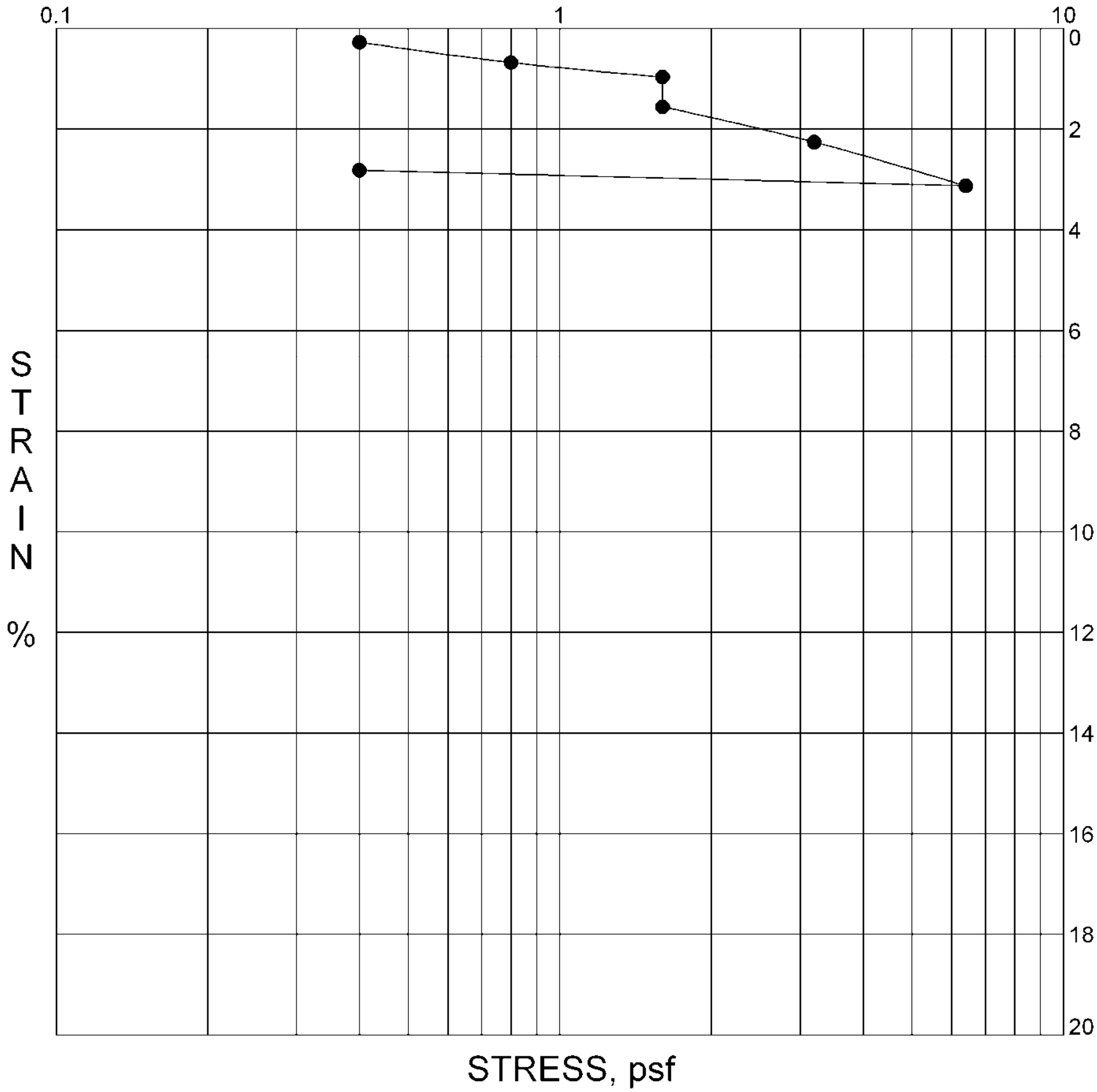
Test Results

Moisture Content (%)	Density (pcf)	Ultimate Strength
Insitu: 2.0	Dry Density: 115.4	Phi (deg): 27.0
Saturated: 17.0		Cohesion (ksf): 0.275

SHEAR TEST DIAGRAM

Figure S.5

PROJECT LOCATION: 6136 Manchester Road	PROJECT NO.: 6058
SAMPLE LOCATION: B-1 @ 10.0	DESCRIPTION: Qal



Test Results

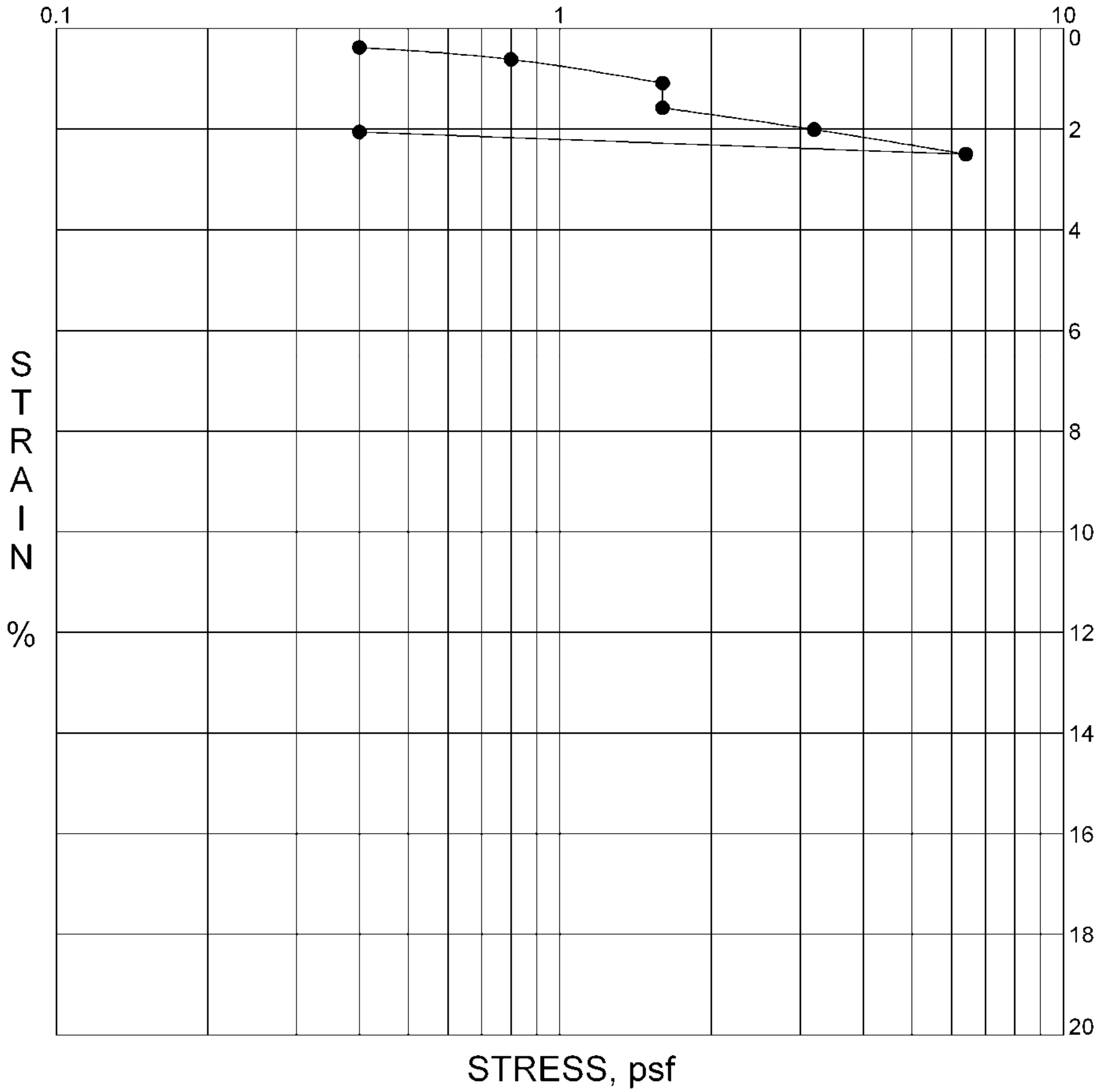
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 3.6	Dry Density: 115.7	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.1

PROJECT LOCATION: 6136 Manchester Road | PROJECT NO.: 6058

SAMPLE LOCATION: B-1 @ 20.0 | DESCRIPTION: Qal



Test Results

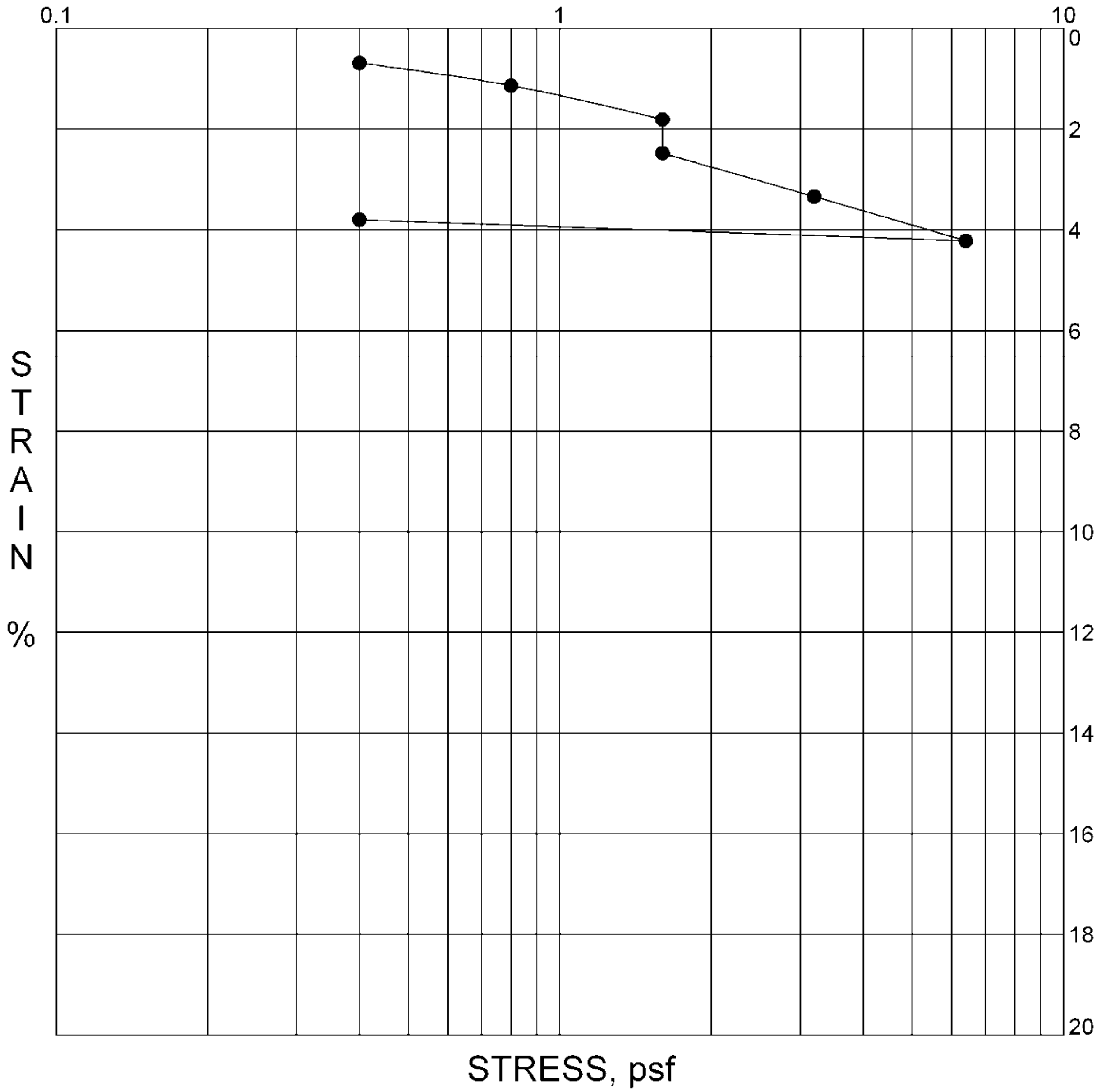
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 5.3	Dry Density: 115.0	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.2

PROJECT LOCATION: 6136 Manchester Road | PROJECT NO.: 6058

SAMPLE LOCATION: B-1 @ 30.0 | DESCRIPTION: Qal



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 8.7	Dry Density: 117.8	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

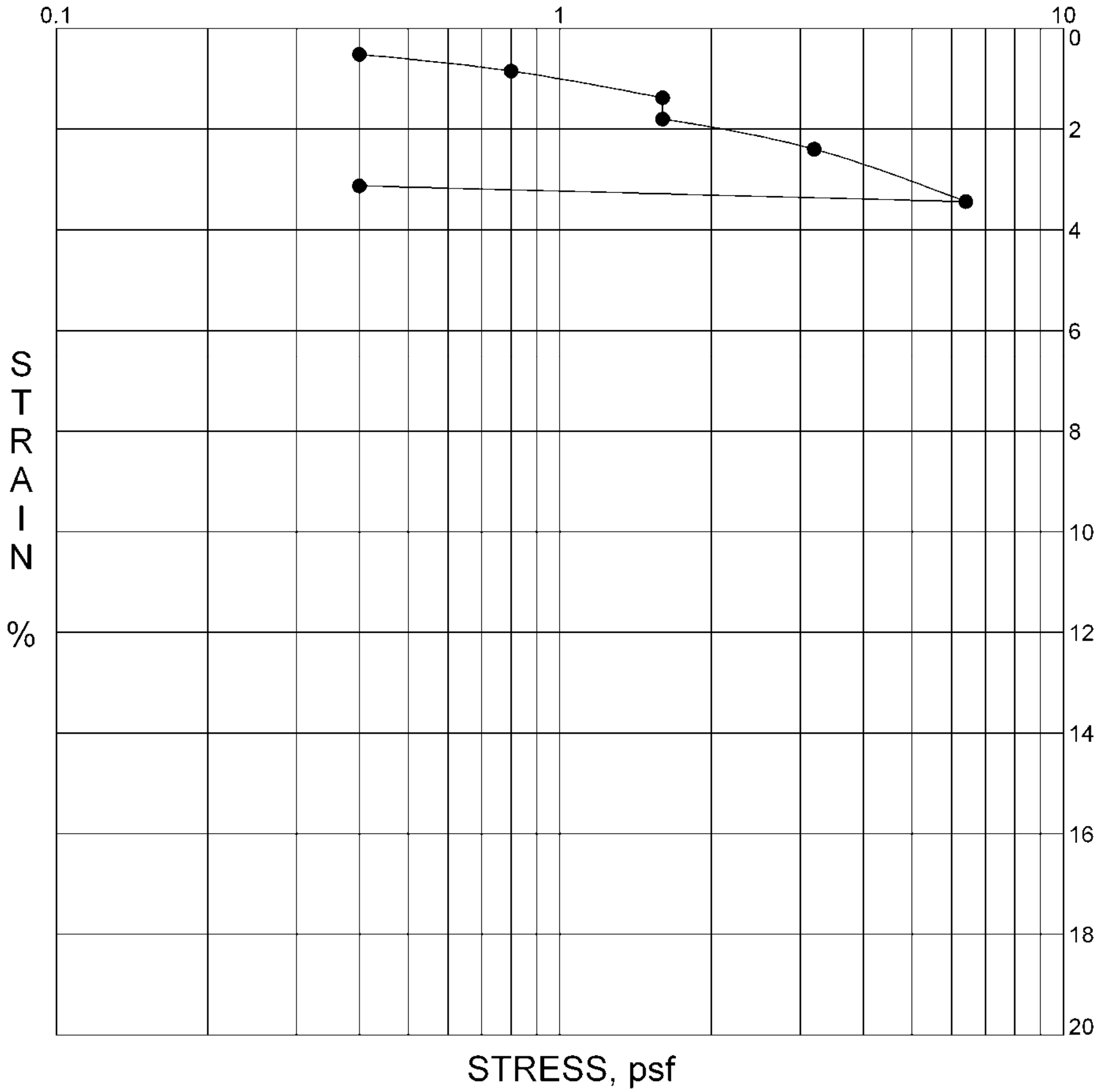
Figure C.3

PROJECT LOCATION: 6136 Manchester Road

PROJECT NO.: 6058

SAMPLE LOCATION: B-1 @ 40.0

DESCRIPTION: Qal



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 6.7	Dry Density: 116.6	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

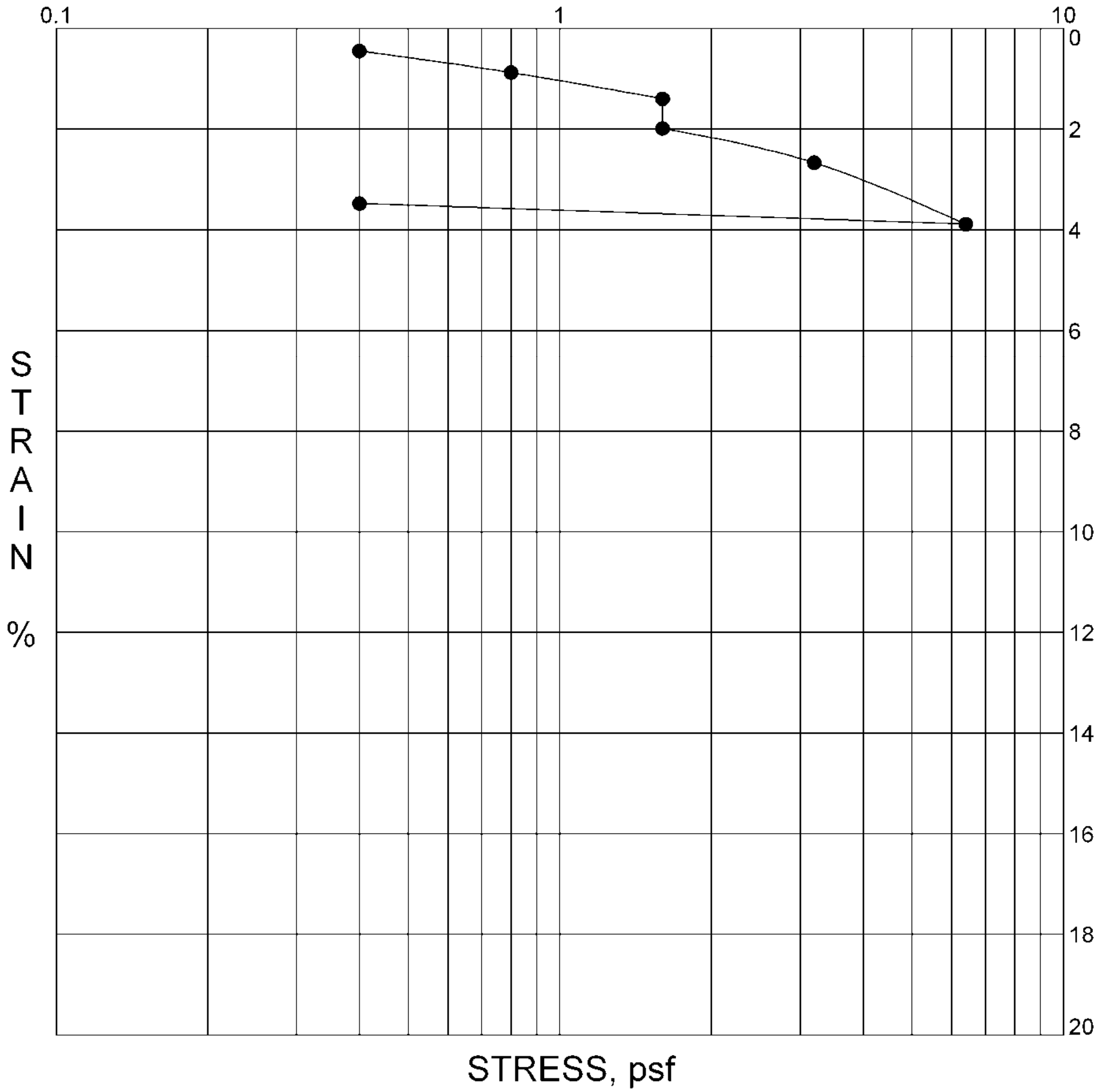
Figure C.4

PROJECT LOCATION: 6136 Manchester Road

PROJECT NO.: 6058

SAMPLE LOCATION: B-2 @ 17.5

DESCRIPTION: Qal



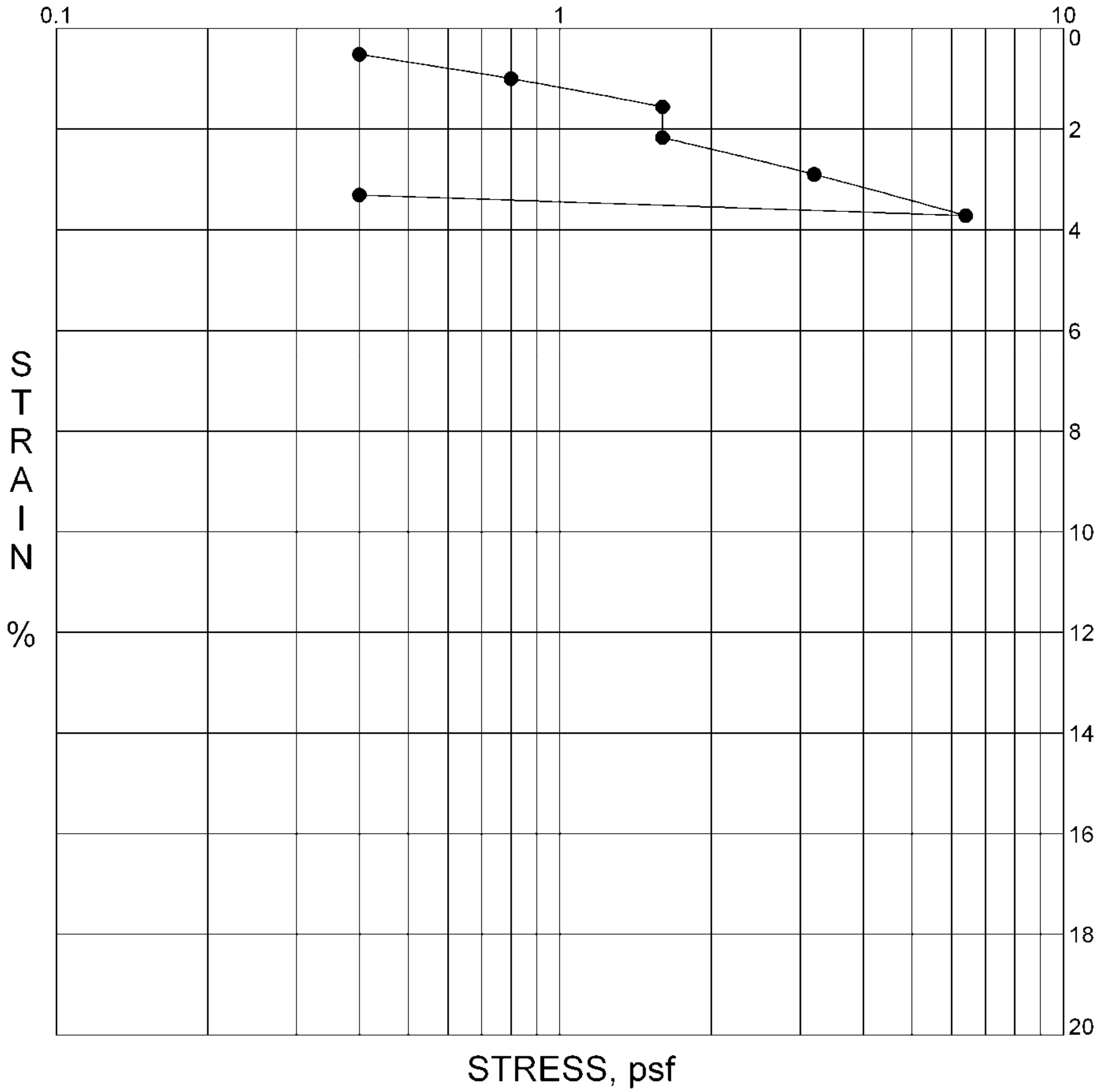
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 7.9	Dry Density: 121.6	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.5

PROJECT LOCATION: 6136 Manchester Road	PROJECT NO.: 6058
SAMPLE LOCATION: B-2 @ 27.5	DESCRIPTION: Qal



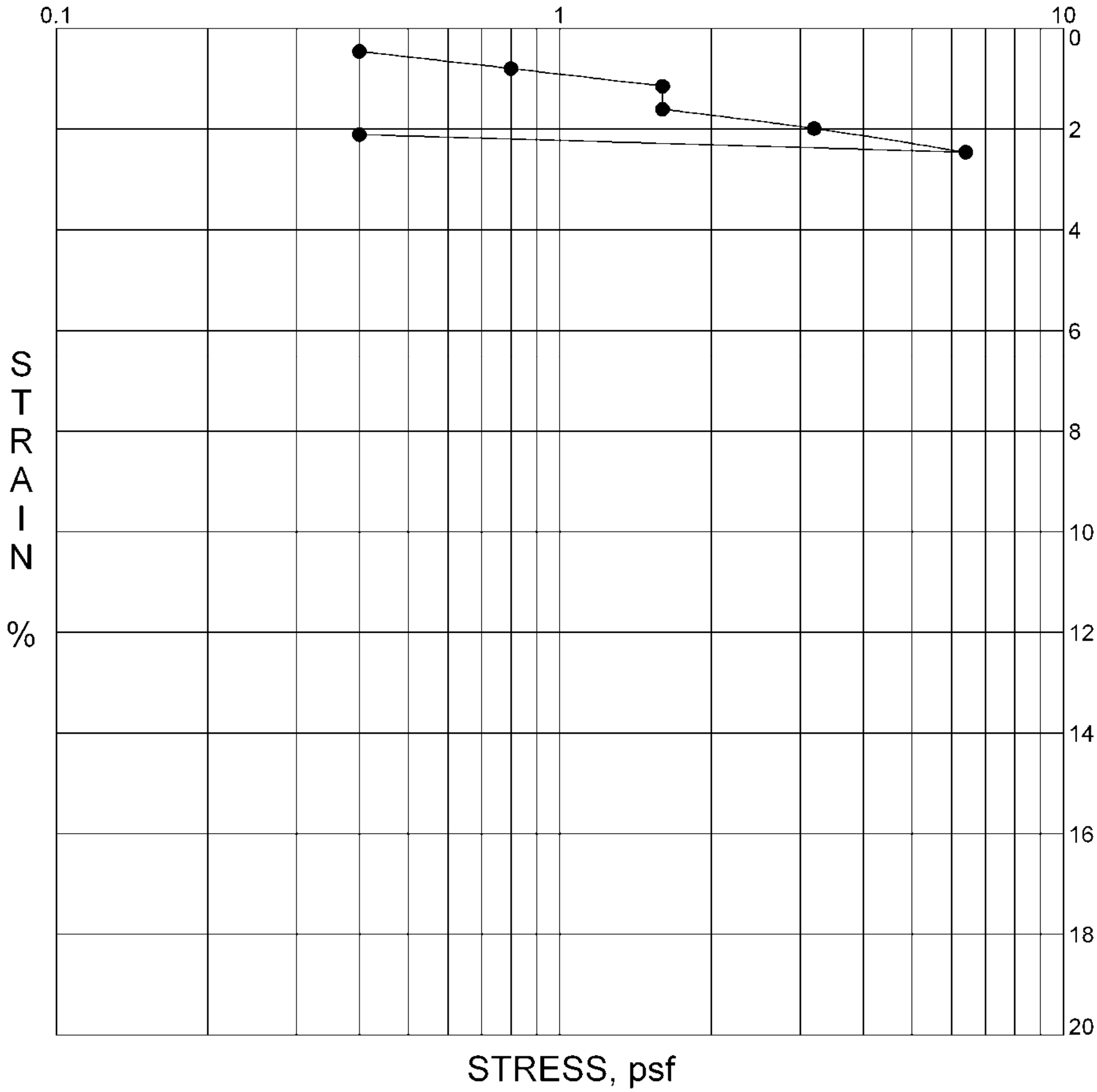
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 7.9	Dry Density: 113.8	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.6

PROJECT LOCATION: 6136 Manchester Road	PROJECT NO.: 6058
SAMPLE LOCATION: B-2 @ 37.5	DESCRIPTION: Qal



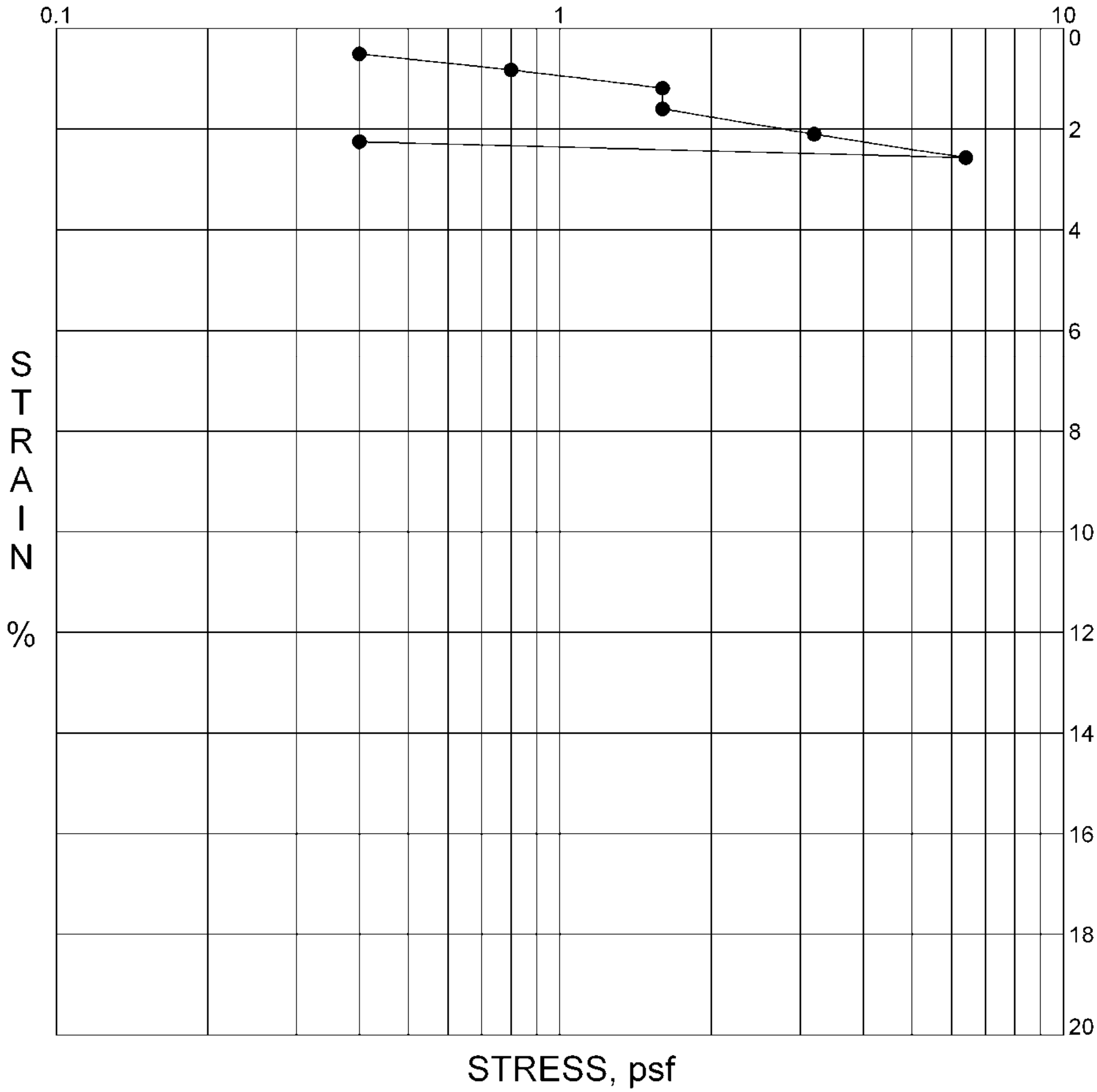
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 3.3	Dry Density: 114.0	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.7

PROJECT LOCATION: 6136 Manchester Road	PROJECT NO.: 6058
SAMPLE LOCATION: B-3 @ 10.0	DESCRIPTION: Qal



Test Results

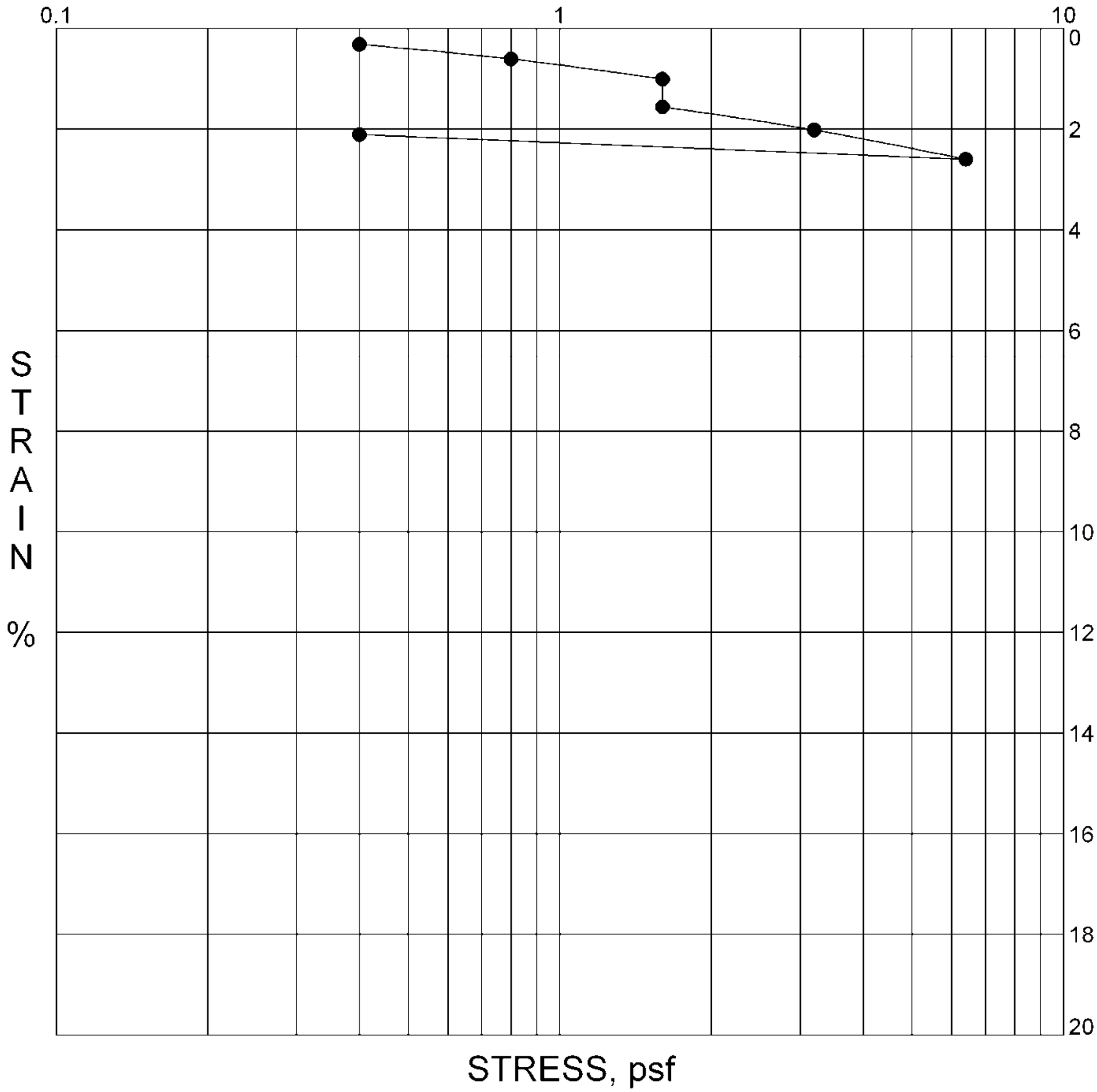
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 6.4	Dry Density: 118.3	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.8

PROJECT LOCATION: 6136 Manchester Road | PROJECT NO.: 6058

SAMPLE LOCATION: B-3 @ 20.0 | DESCRIPTION: Qal



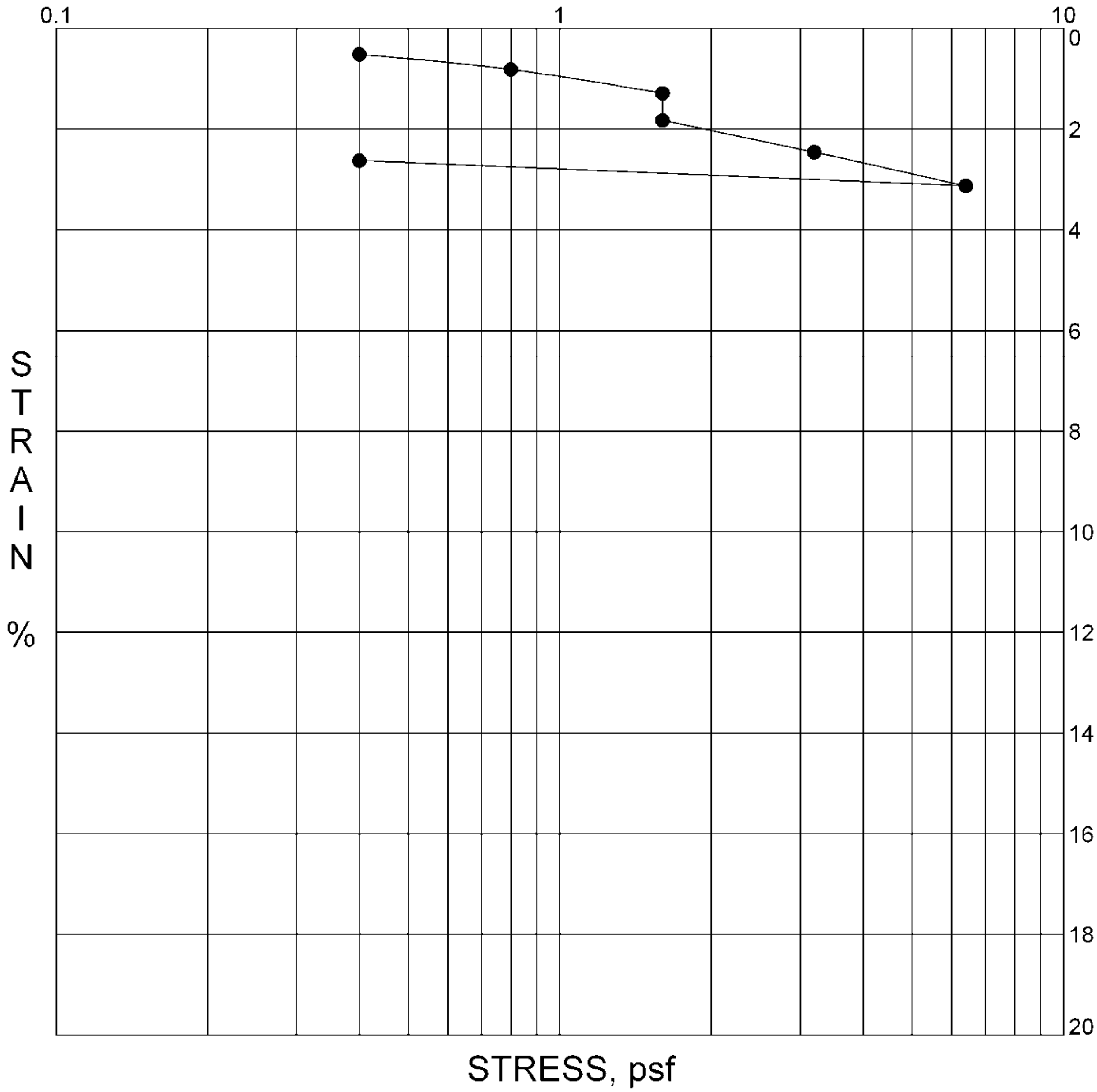
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 7.9	Dry Density: 118.2	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.9

PROJECT LOCATION: 6136 Manchester Road	PROJECT NO.: 6058
SAMPLE LOCATION: B-3 @ 30.0	DESCRIPTION: Qal



Test Results

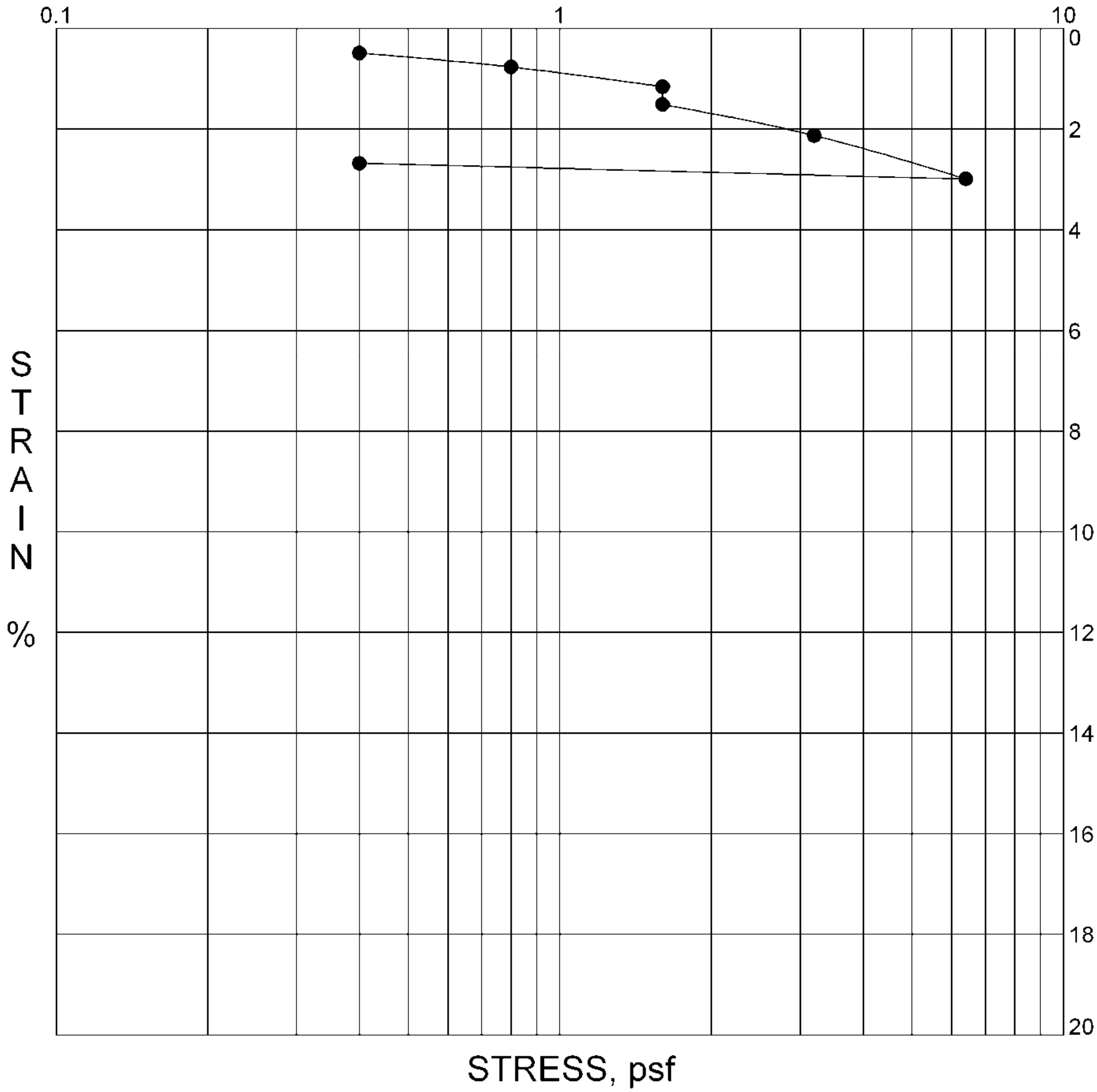
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 7.6	Dry Density: 112.2	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.10

PROJECT LOCATION: 6136 Manchester Road PROJECT NO.: 6058

SAMPLE LOCATION: B-3 @ 40.0 DESCRIPTION: Qal



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 4.0	Dry Density: 114.1	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

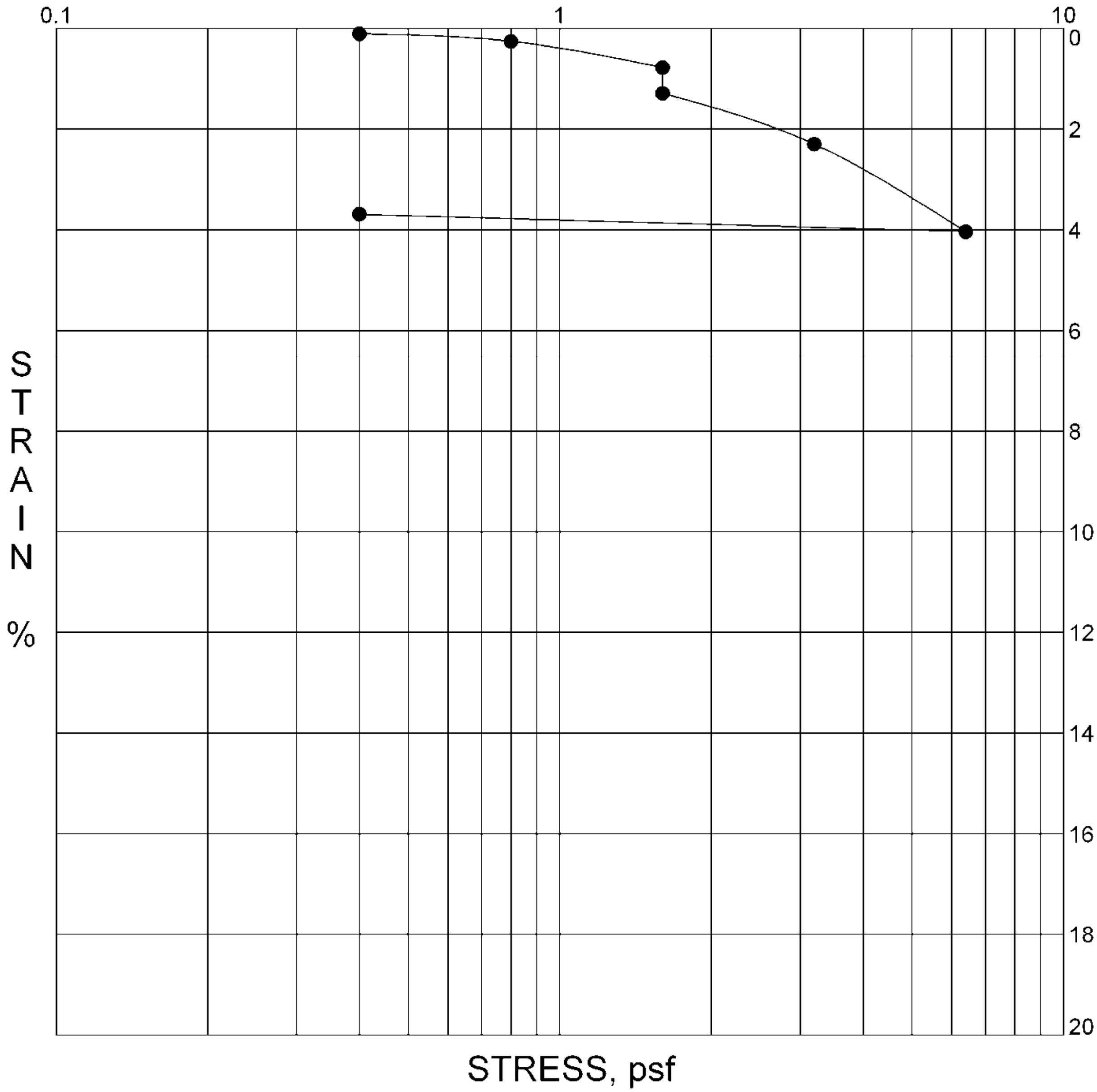
Figure C.11

PROJECT LOCATION: 6136 Manchester Road

PROJECT NO.: 6058

SAMPLE LOCATION: B-4 @ 15.0

DESCRIPTION: Qal



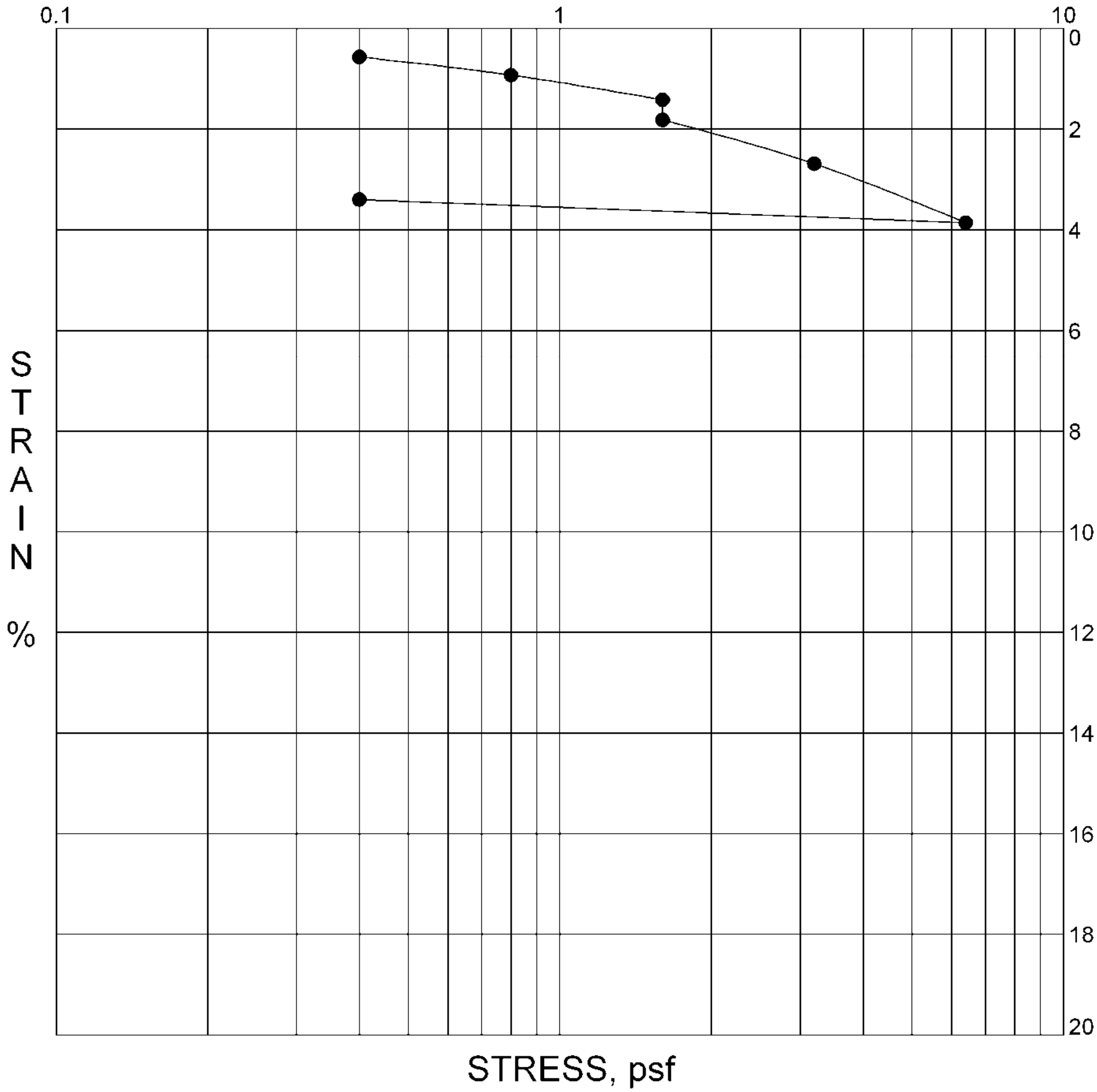
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 6.4	Dry Density: 113.8	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.12

PROJECT LOCATION: 6136 Manchester Road PROJECT NO.: 6058
SAMPLE LOCATION: B-4 @ 25.0 DESCRIPTION: Qal

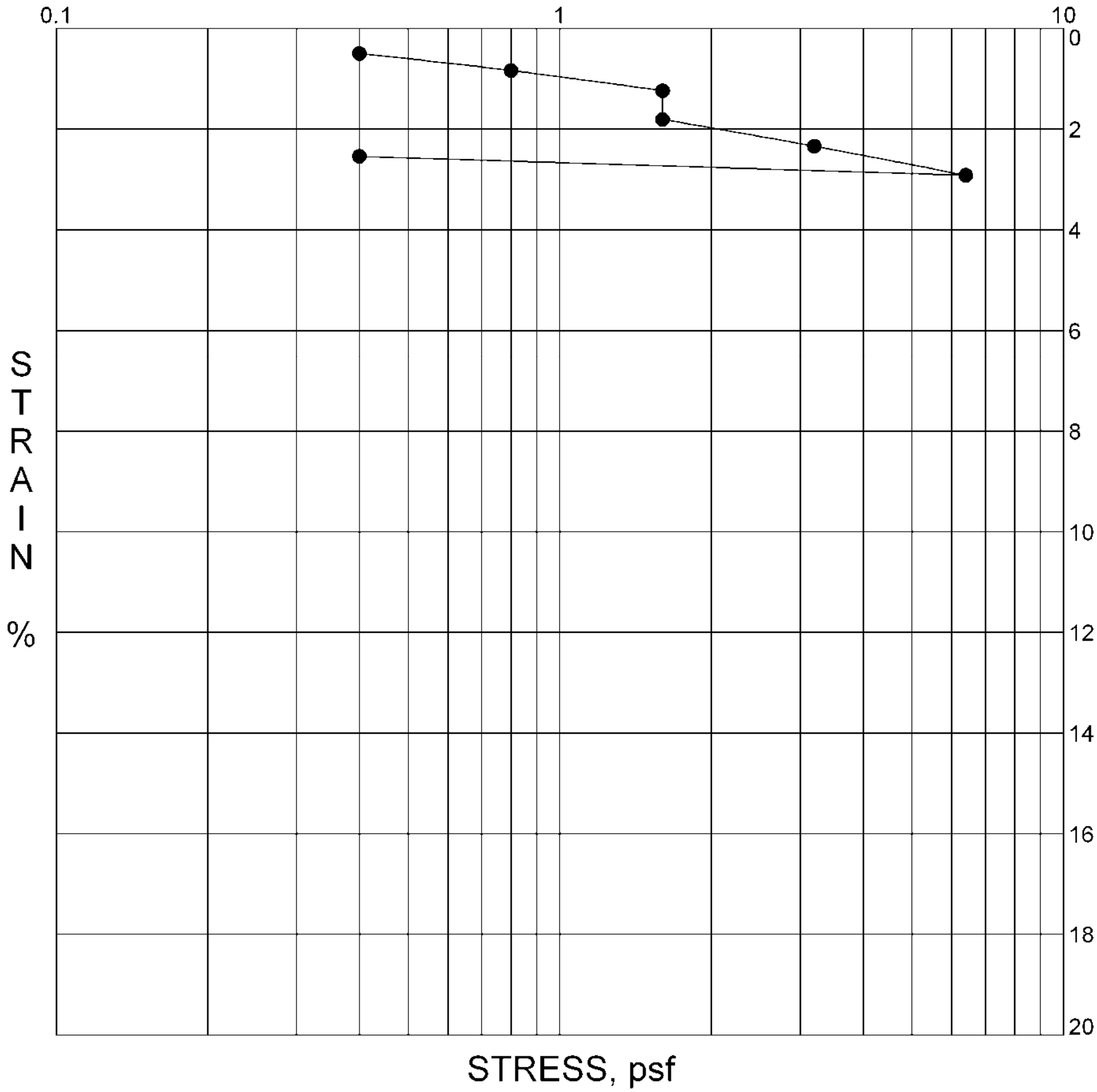


Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 7.4	Dry Density: 118.1	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM Figure C.13

PROJECT LOCATION: 6136 Manchester Road	PROJECT NO.: 6058
SAMPLE LOCATION: B-4 @ 35.0	DESCRIPTION: Qal



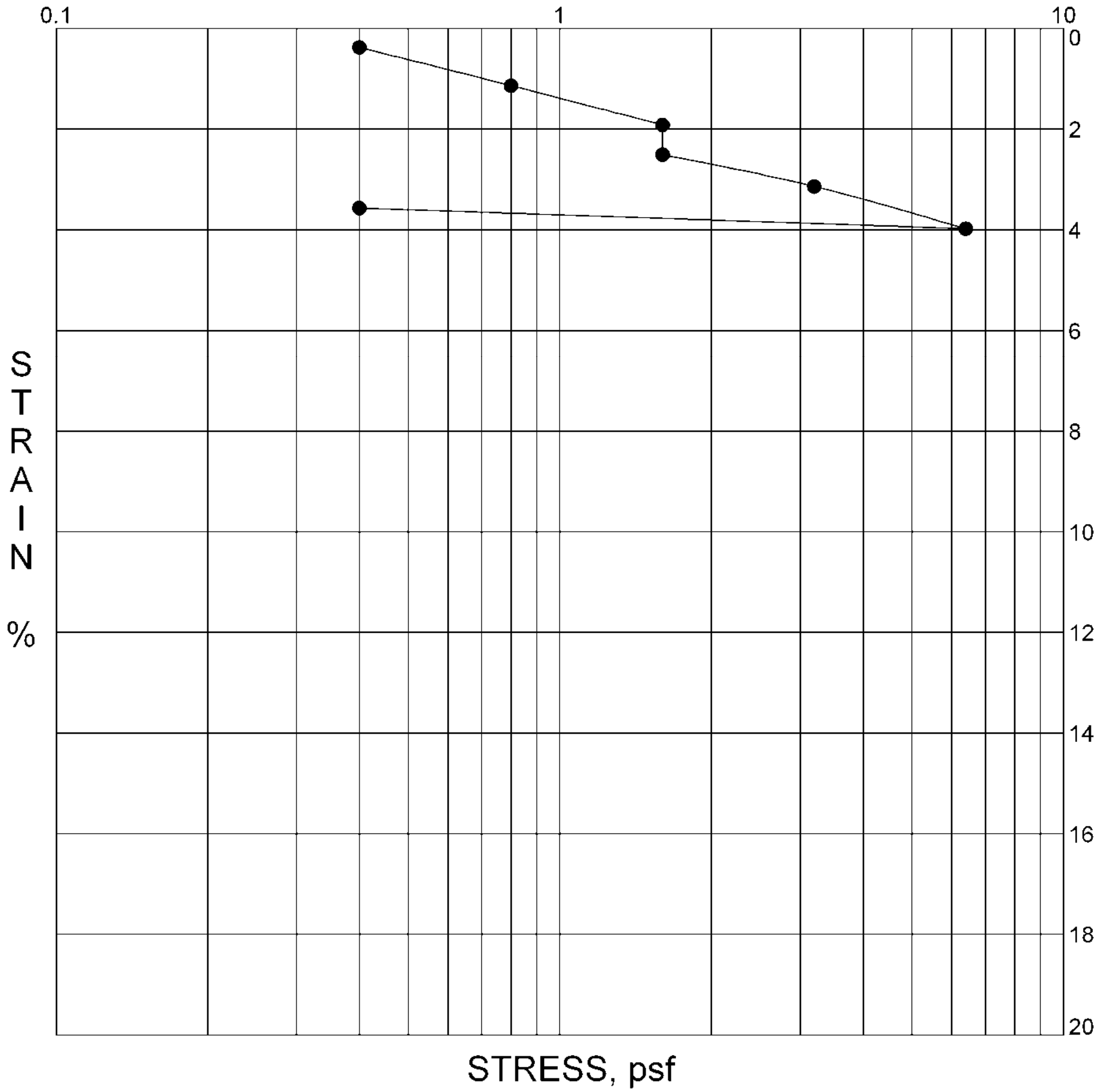
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 6.0	Dry Density: 116.8	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.14

PROJECT LOCATION: 6136 Manchester Road	PROJECT NO.: 6058
SAMPLE LOCATION: B-5 @ 10.0	DESCRIPTION: Qal



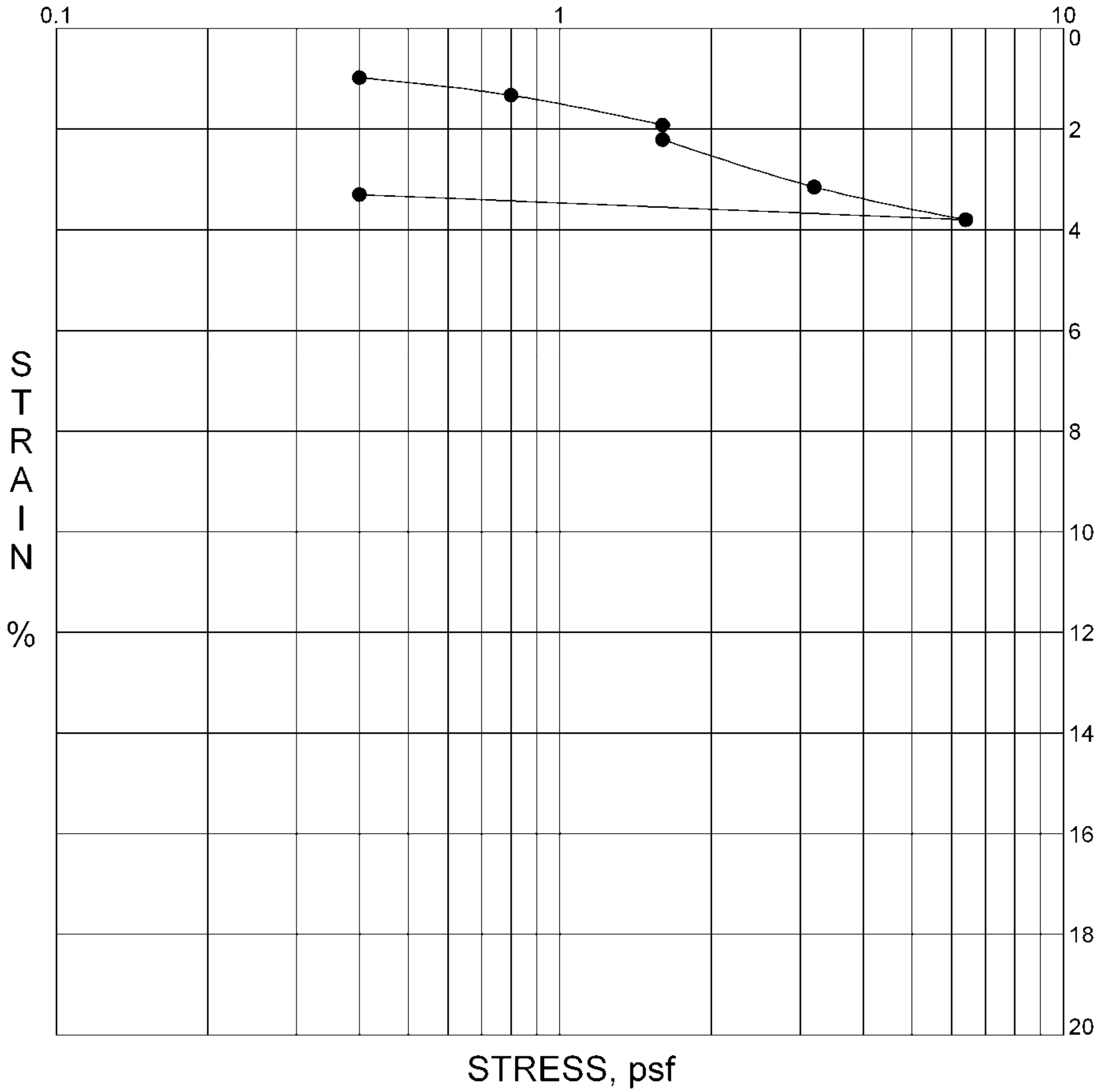
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 5.7	Dry Density: 114.9	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.15

PROJECT LOCATION: 6136 Manchester Road	PROJECT NO.: 6058
SAMPLE LOCATION: B-5 @ 20.0	DESCRIPTION: Qal



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 2.8	Dry Density: 112.9	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

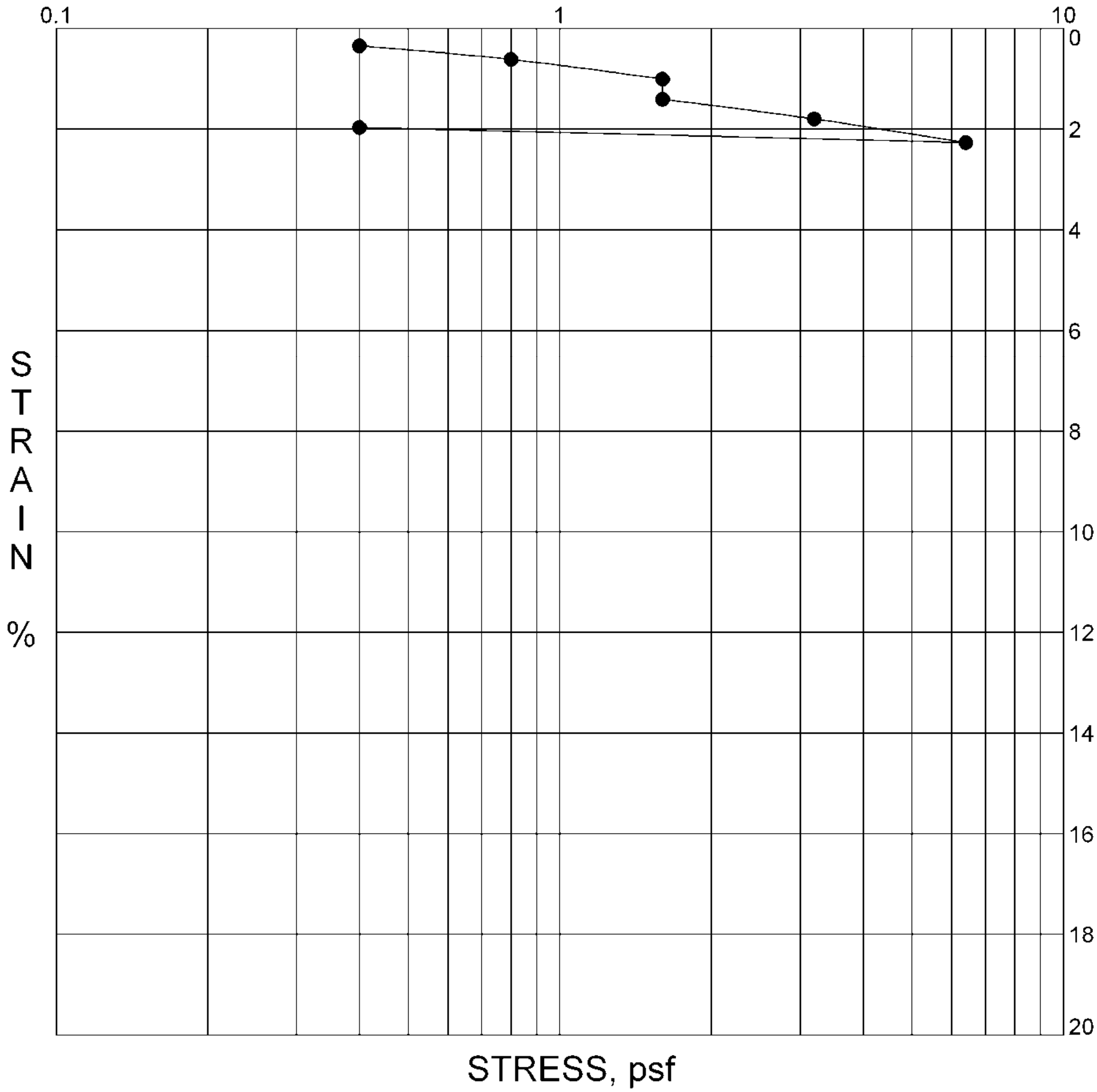
Figure C.16

PROJECT LOCATION: 6136 Manchester Road

PROJECT NO.: 6058

SAMPLE LOCATION: B-5 @ 30.0

DESCRIPTION: Qal



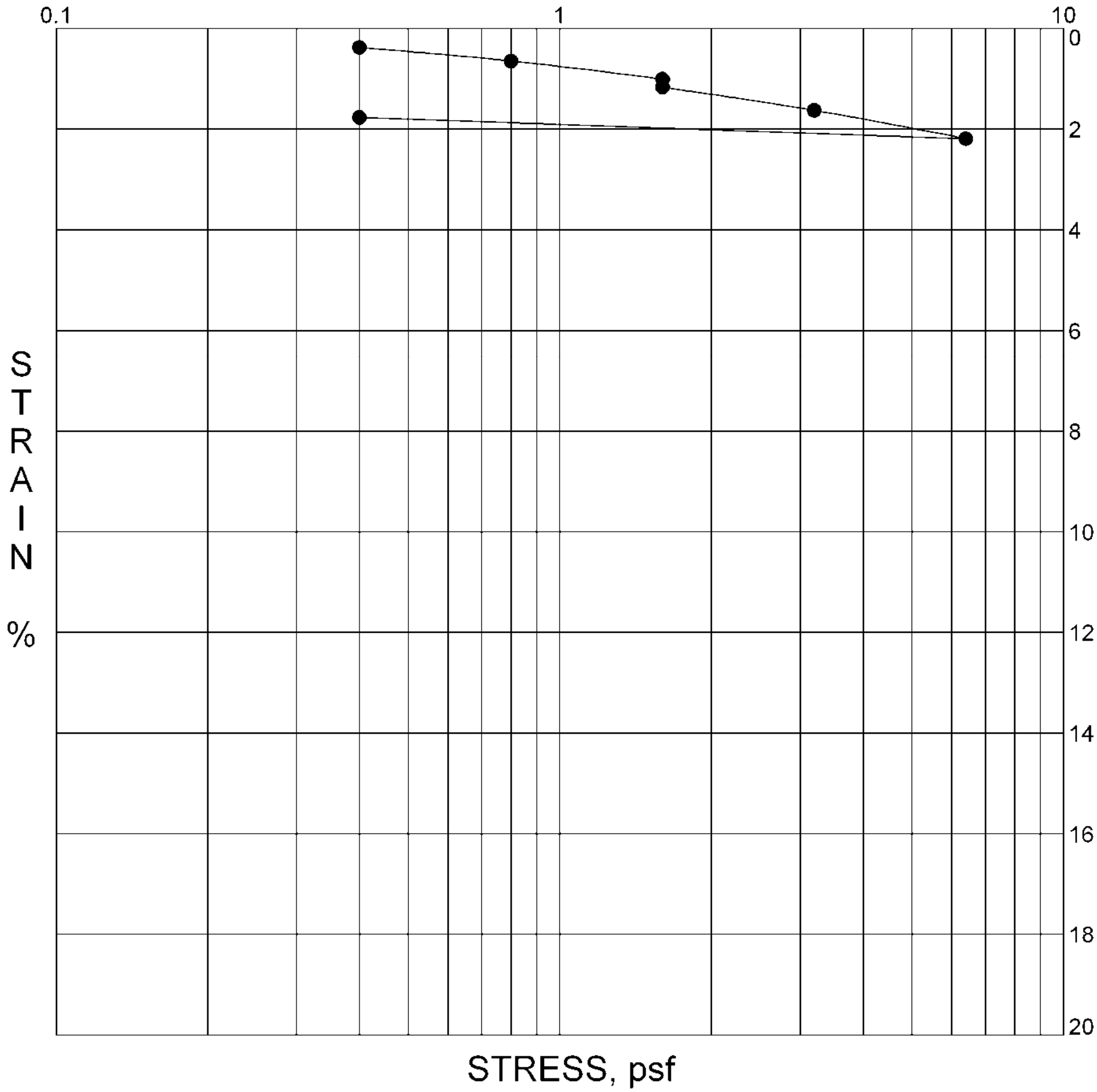
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 7.7	Dry Density: 113.2	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.17

PROJECT LOCATION: 6136 Manchester Road	PROJECT NO.: 6058
SAMPLE LOCATION: B-5 @ 40.0	DESCRIPTION: Qal



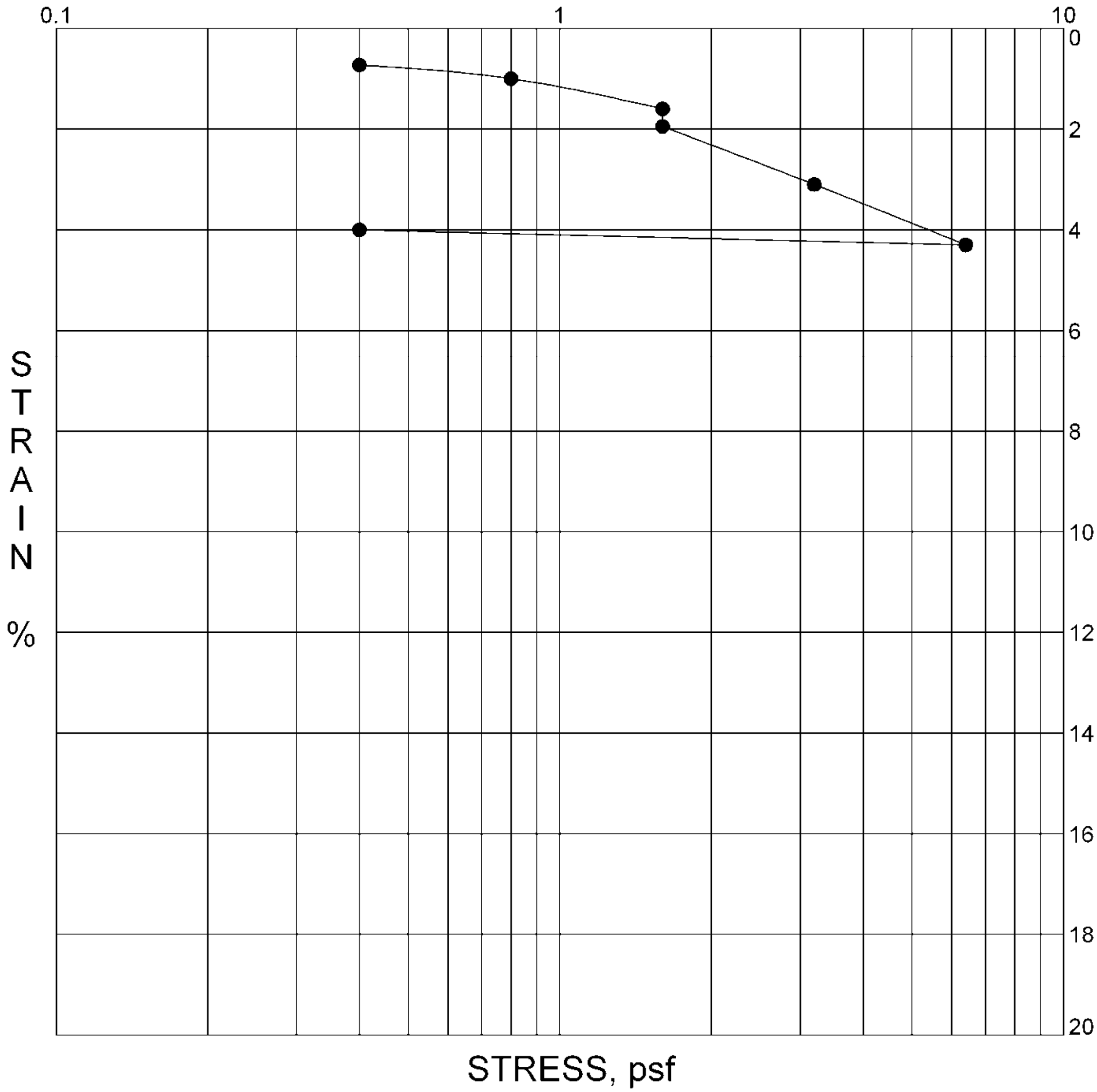
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 2.7	Dry Density: 116.3	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.18

PROJECT LOCATION: 6136 Manchester Road	PROJECT NO.: 6058
SAMPLE LOCATION: B-6 @ 15.0	DESCRIPTION: Qal



Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 1.9	Dry Density: 112.5	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

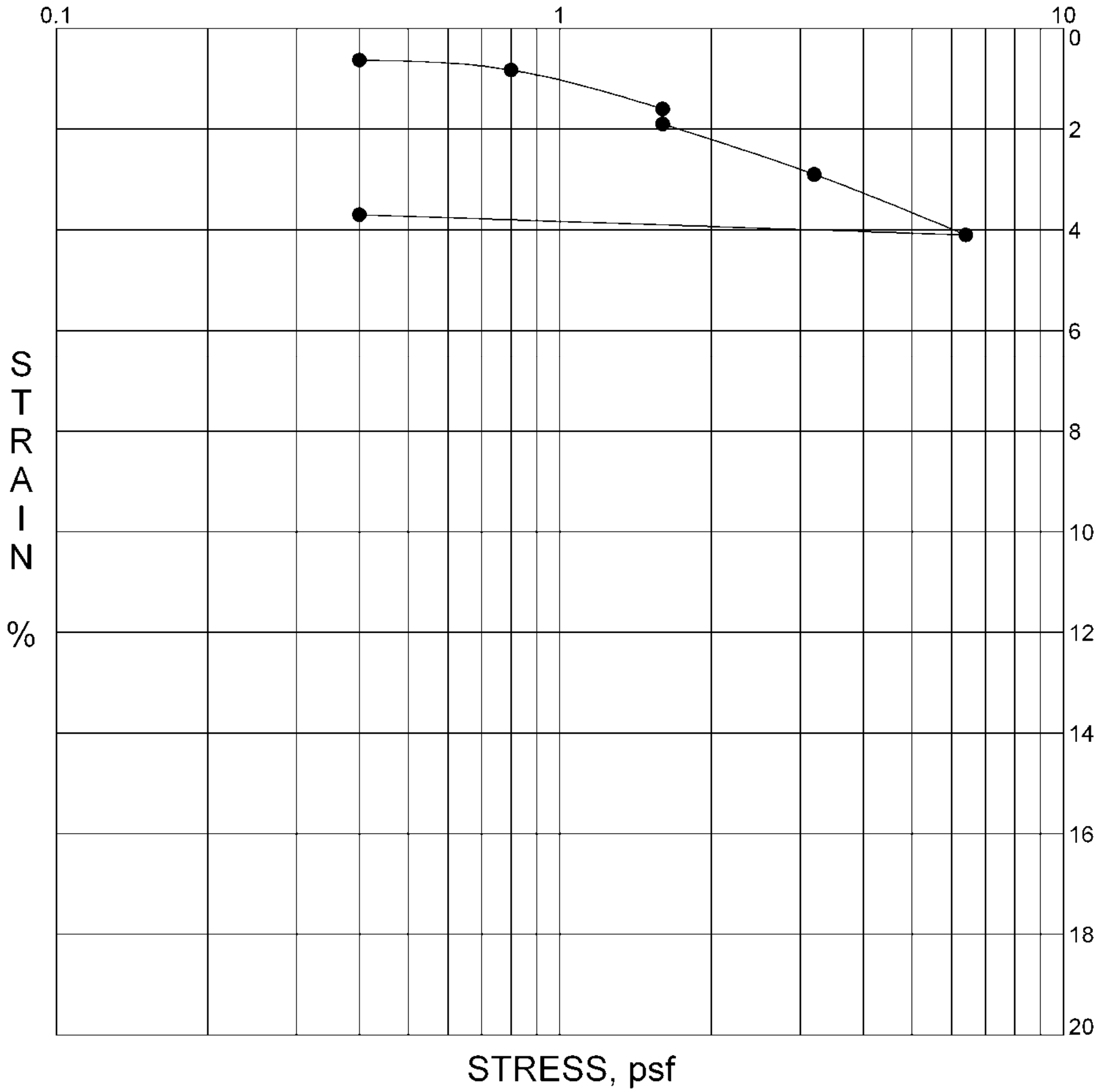
Figure C.19

PROJECT LOCATION: 6136 Manchester Road

PROJECT NO.: 6058

SAMPLE LOCATION: B-6 @ 25.0

DESCRIPTION: Qal



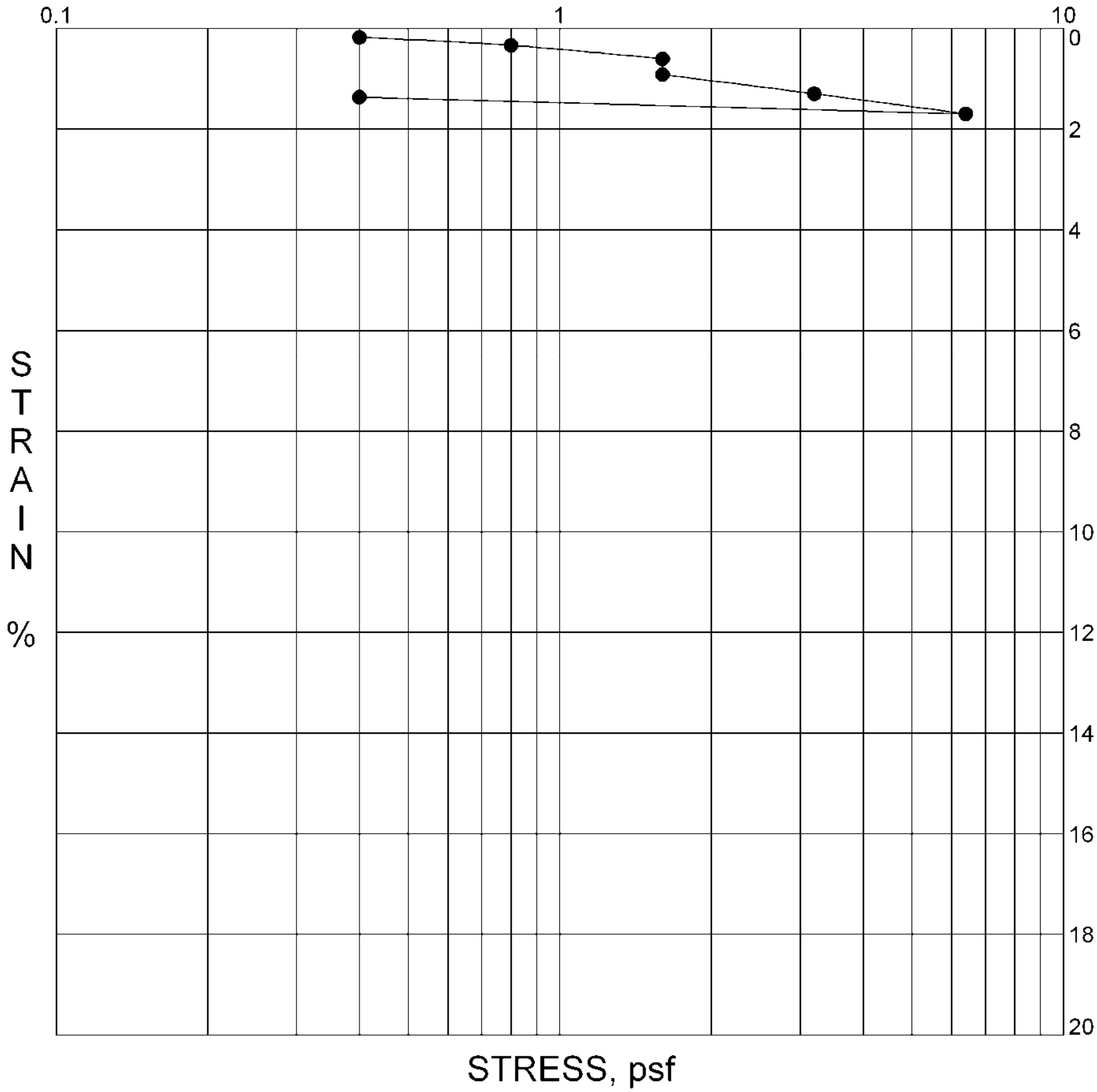
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 4.0	Dry Density: 120.4	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.20

PROJECT LOCATION: 6136 Manchester Road	PROJECT NO.: 6058
SAMPLE LOCATION: B-6 @ 35.0	DESCRIPTION: Qal



Test Results

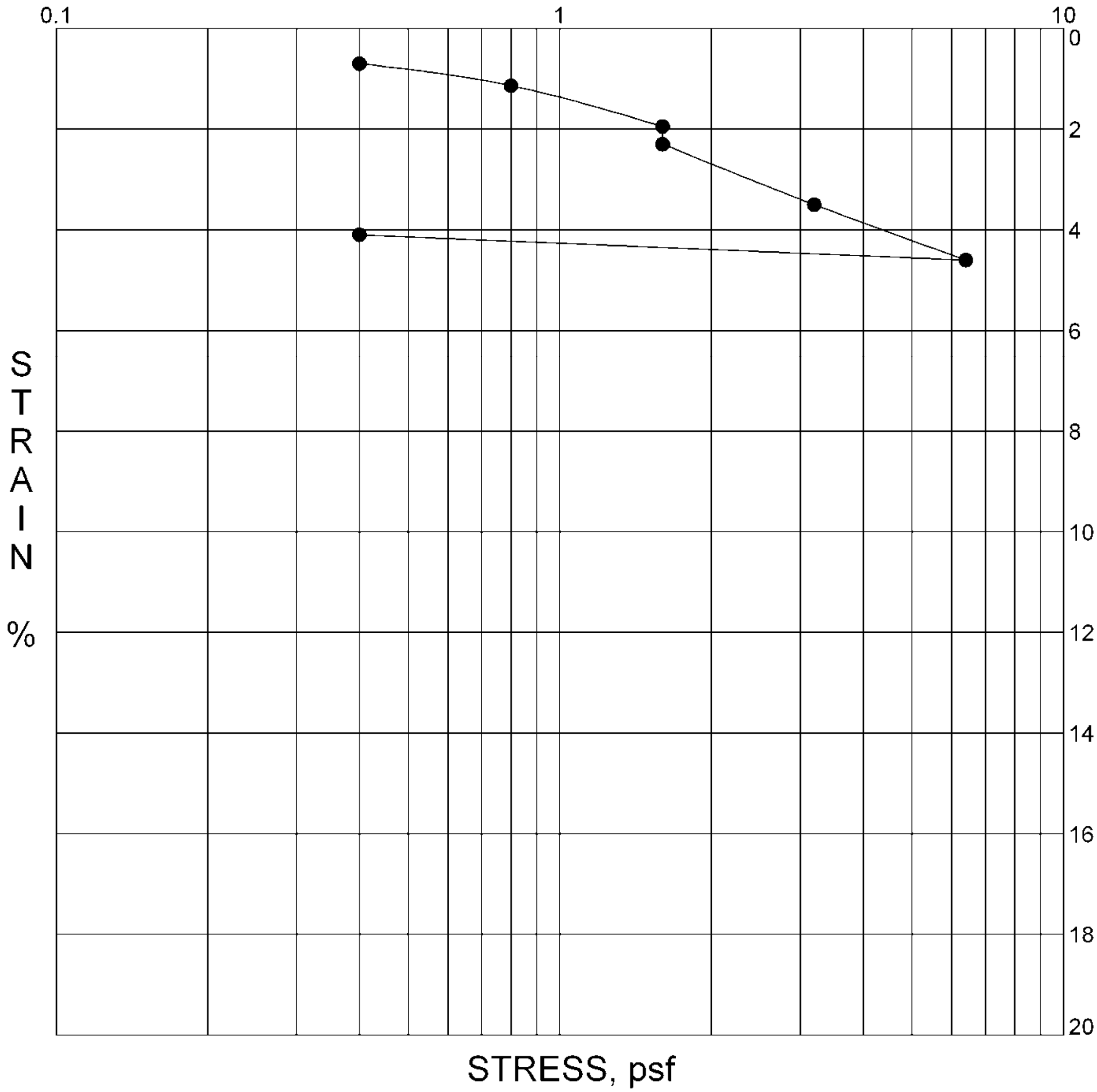
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 5.9	Dry Density: 111.2	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.21

PROJECT LOCATION: 6136 Manchester Road | PROJECT NO.: 6058

SAMPLE LOCATION: B-7 @ 15.0 | DESCRIPTION: Qal



Test Results

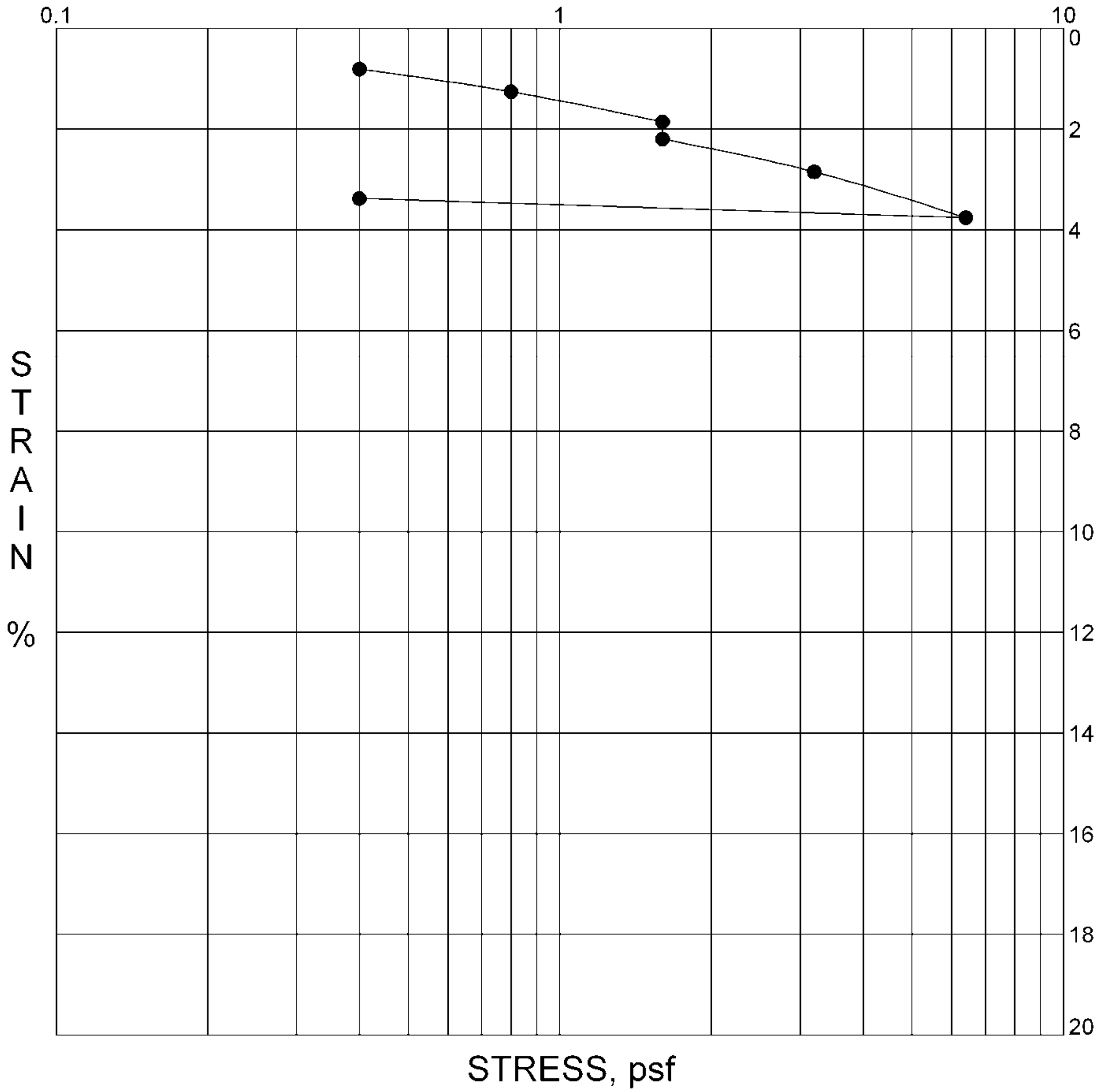
Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 3.6	Dry Density: 108.2	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.22

PROJECT LOCATION: 6136 Manchester Road | PROJECT NO.: 6058

SAMPLE LOCATION: B-7 @ 25.0 | DESCRIPTION: Qal



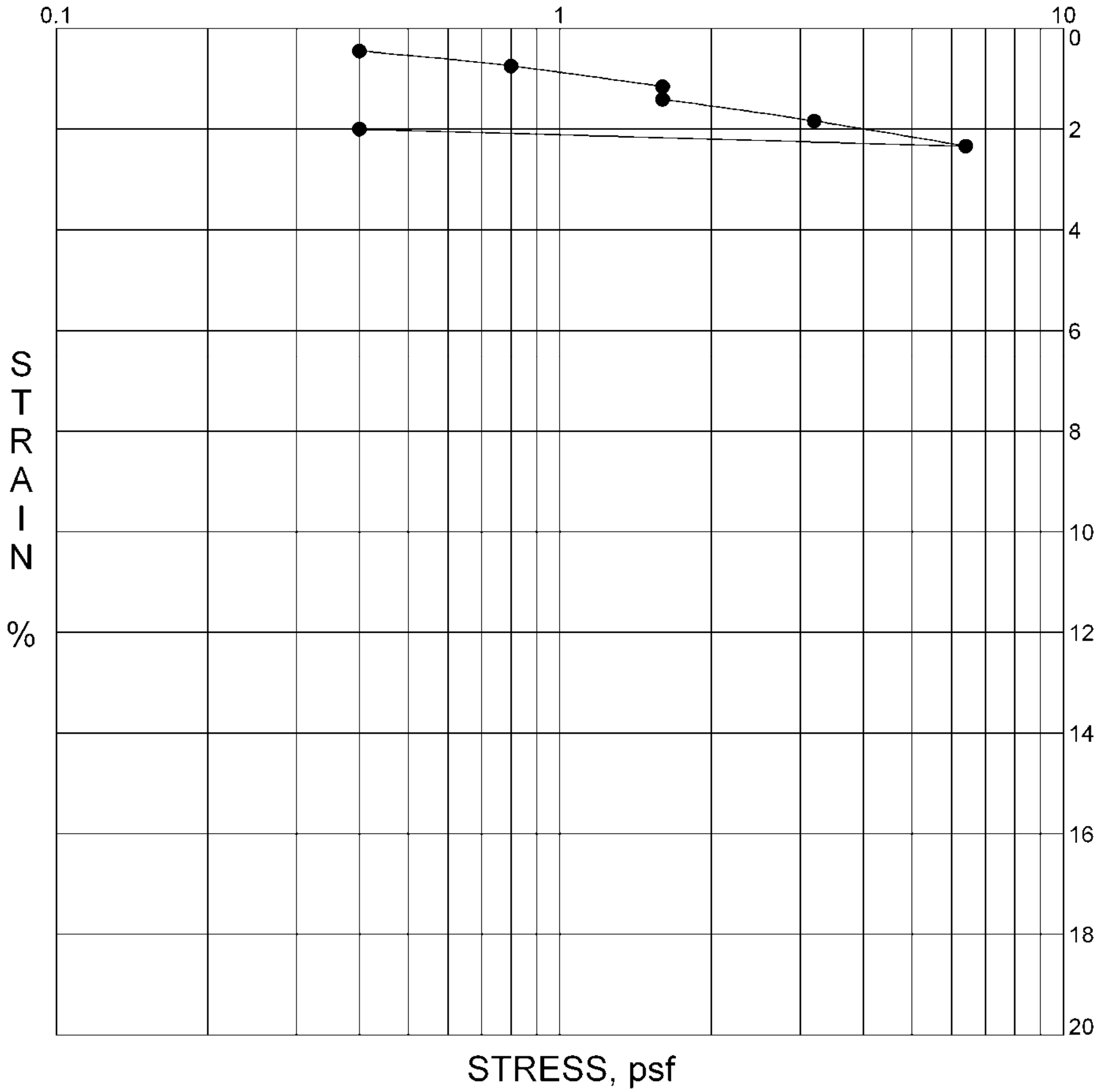
Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 6.4	Dry Density: 115.4	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.23

PROJECT LOCATION: 6136 Manchester Road	PROJECT NO.: 6058
SAMPLE LOCATION: B-7 @ 35.0	DESCRIPTION: Qal

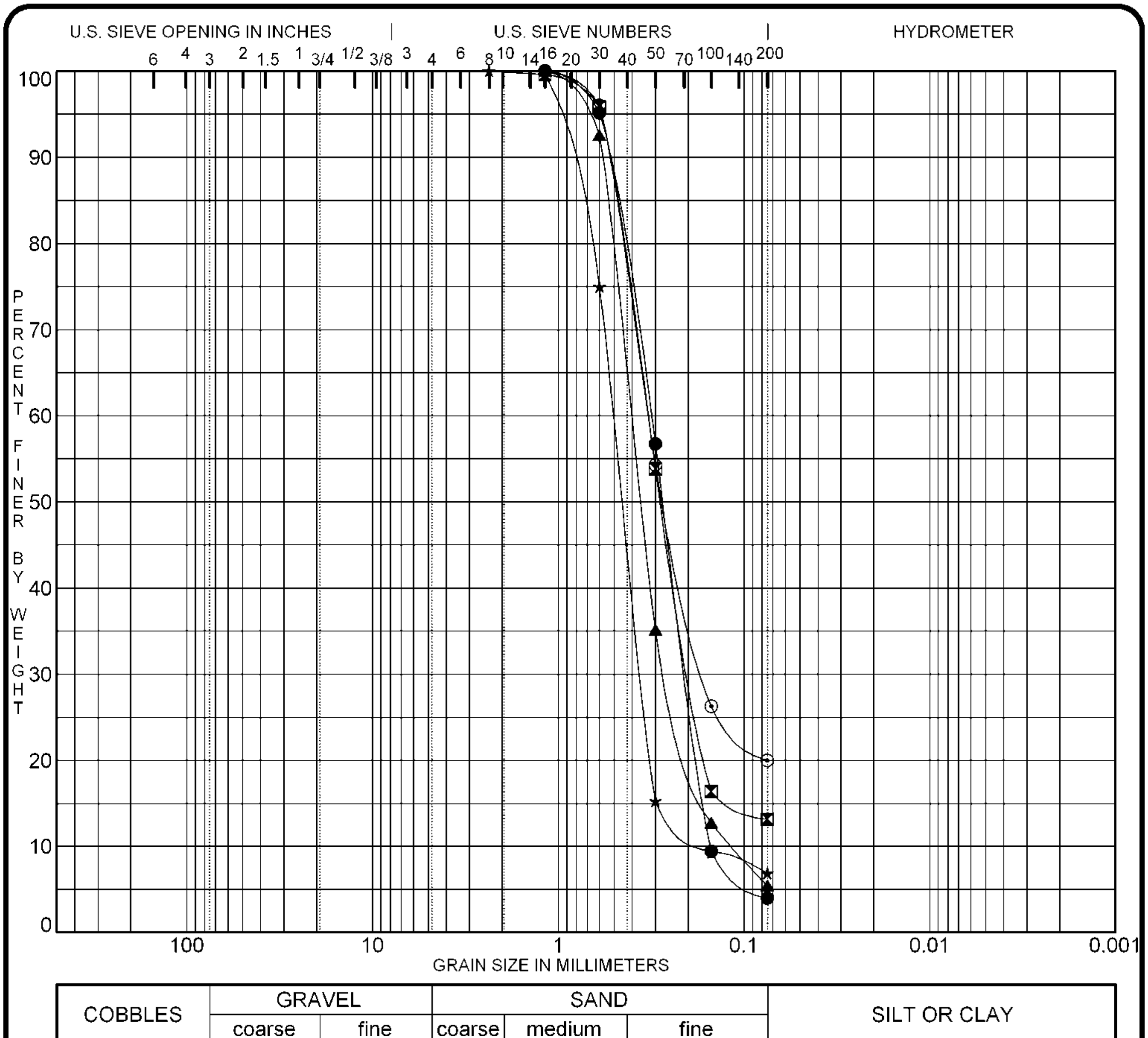


Test Results

Moisture Content (%)	Density (pcf)	Water Added At
Insitu: 2.8	Dry Density: 110.4	**1600 lbs.**

CONSOLIDATION TEST DIAGRAM

Figure C.24



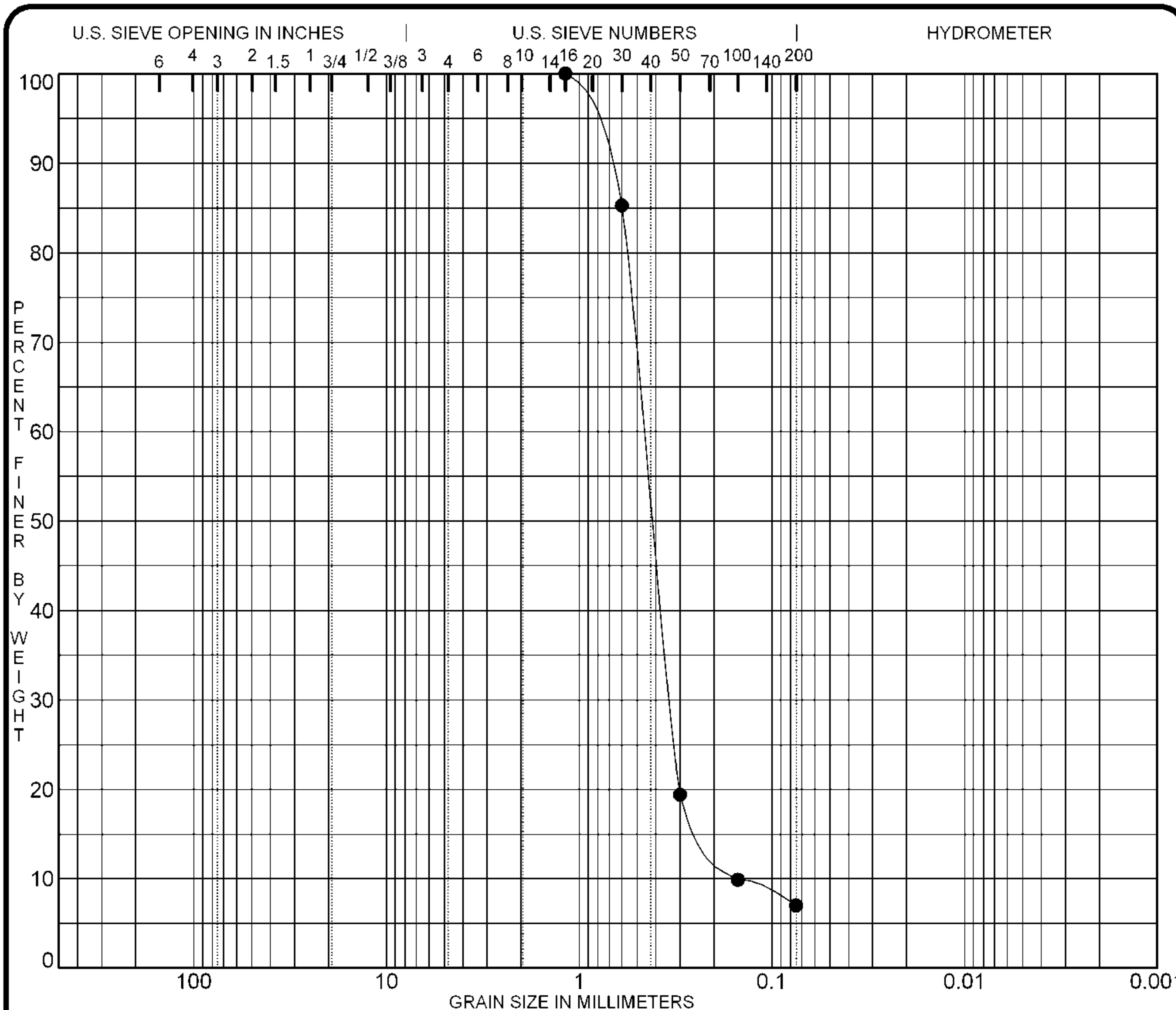
	Specimen Identification		USCS Classification	MC%	LL	PL	PI	Cc	Cu
●	**B-1**	**25.0**	**Qal**	**9**				**0.85**	**2.1**
⧗	**B-4**	**15.0**	**Qal**	**6**					
▲	**B-5**	**25.0**	**Qal**	**5**				**1.40**	**3.5**
★	**B-6**	**15.0**	**Qal**	**2**				**1.54**	**3.1**
⊙	**B-6**	**25.0**	**Qal**	**4**					

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-1**	**25.0**	**1.18**	**0.32**	**0.203**	**0.1513**	**0.0**	**96.0**	**4.0**	
⧗	**B-4**	**15.0**	**1.18**	**0.33**	**0.193**		**0.0**	**86.4**	**13.1**	
▲	**B-5**	**25.0**	**1.18**	**0.40**	**0.256**	**0.1159**	**0.0**	**94.6**	**5.4**	
★	**B-6**	**15.0**	**2.36**	**0.50**	**0.356**	**0.1629**	**0.0**	**93.1**	**6.9**	
⊙	**B-6**	**25.0**	**1.18**	**0.33**	**0.164**		**0.0**	**80.0**	**20.0**	

PROJECT **- 6136 Manchester Road**

GRADATION CURVES

GeoConcepts



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification	USCS Classification	MC%	LL	PL	PI	Cc	Cu
● B-7 15.0	Qal	4				1.61	3.0

Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● B-7 15.0	1.18	0.46	0.335	0.1516	0.0	93.0	7.0	

PROJECT - 6136 Manchester Road

GRADATION CURVES

GeoConcepts

APPENDIX III

ANALYSES

Lateral Design

Seismic Evaluation

Retaining Walls (15 Feet High with Level Backslope)

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:	Qal	WALL HEIGHT	15 feet
SHEAR DIAGRAM:	B-5@10	BACKSLOPE ANGLE:	0 degrees
COHESION:	200 psf	SURCHARGE:	0 pounds
PHI ANGLE:	32 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):	133.3 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 22.6 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	56 degrees
AREA OF TRIAL FAILURE WEDGE	72.6 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	9795.3 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	14.3 feet
DEPTH OF TENSION CRACK	3.1 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	8.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**4346.2 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**38.6 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**pcf**

Retaining Walls (25 Feet High with Level Backslope)

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:	Qal	WALL HEIGHT	25 feet
SHEAR DIAGRAM:	B-5@10	BACKSLOPE ANGLE:	0 degrees
COHESION:	200 psf	SURCHARGE:	0 pounds
PHI ANGLE:	32 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):	133.3 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 22.6 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	56 degrees
AREA OF TRIAL FAILURE WEDGE	208.2 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	28108.6 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	26.8 feet
DEPTH OF TENSION CRACK	2.8 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	15.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**14569.2 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**46.6 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**pcf**

Seismic Retaining Walls (25 Feet High with Level Backslope)

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:	Qal	WALL HEIGHT	25 feet
SHEAR DIAGRAM:	B-5@10	BACKSLOPE ANGLE:	0 degrees
COHESION:	200 psf	SURCHARGE:	0 pounds
PHI ANGLE:	32 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):	200.0 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 32.0 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	267.4 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	36093.4 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	29.0 feet
DEPTH OF TENSION CRACK	3.1 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	19.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**16365.5 pounds**

At Rest Pressure for Retaining Walls

AT REST PRESSURE CALCULATION

CALCULATION PARAMETERS

EARTH MATERIAL:	Qal	COHESION:	200	psf
SHEAR DIAGRAM:	B-5@10	PHI ANGLE:	32	degrees
		DENSITY:	135	pcf

CALCULATED RESULTS

AT REST PRESSURE	64	pcf

CONCLUSIONS:

THE CALCULATED PRESSURE DUE TO AT REST CONDITIONS ARE PRESENTED IN THE TABLE.

Hydrostatic Retaining Walls (15 Feet High with Level Backslope)

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:	Qal	WALL HEIGHT	15 feet
SHEAR DIAGRAM:	B-5@10	BACKSLOPE ANGLE:	0 degrees
COHESION:	200 psf	SURCHARGE:	0 pounds
PHI ANGLE:	32 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	75 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):	133.3 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 22.6 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	62.3 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	4671.2 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	11.0 feet
DEPTH OF TENSION CRACK	5.8 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	6.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**1553.6 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**13.8 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**pcf**

Hydrostatic Retaining Walls (25 Feet High with Level Backslope)

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:	Qal	WALL HEIGHT	25 feet
SHEAR DIAGRAM:	B-5@10	BACKSLOPE ANGLE:	0 degrees
COHESION:	200 psf	SURCHARGE:	0 pounds
PHI ANGLE:	32 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	75 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):	133.3 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 22.6 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	56 degrees
AREA OF TRIAL FAILURE WEDGE	199.7 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	14979.3 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	23.2 feet
DEPTH OF TENSION CRACK	5.7 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	13.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**6444.4 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**20.6 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**pcf**

Hydrostatic At Rest Pressure for Retaining Walls

AT REST PRESSURE CALCULATION

CALCULATION PARAMETERS

EARTH MATERIAL:	Qal	COHESION:	200	psf
SHEAR DIAGRAM:	B-5@10	PHI ANGLE:	32	degrees
		DENSITY:	75	pcf

CALCULATED RESULTS

AT REST PRESSURE	36	pcf

CONCLUSIONS:

THE CALCULATED PRESSURE DUE TO AT REST CONDITIONS ARE PRESENTED IN THE TABLE.

Maximum Vertical Cut Height

TEMPORARY EXCAVATION HEIGHT

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

CALCULATION PARAMETERS

EARTH MATERIAL:	Qal	WALL HEIGHT:	15 feet
SHEAR DIAGRAM:	B-5@10	BACKSLOPE ANGLE:	0 degrees
COHESION:	200 psf	SURCHARGE:	0 pounds
PHI ANGLE:	32 degrees	SURCHARGE TYPE:	U Uniform
DENSITY:	120 pcf	INITIAL FAILURE ANGLE:	20 degrees
SAFETY FACTOR:	1.25	FINAL FAILURE ANGLE:	70 degrees
WALL FRICTION:	0 degrees	INITIAL TENSION CRACK:	4 feet
CD (C/FS):	160.0 psf	FINAL TENSION CRACK:	30 feet
PHID = ATAN(TAN(PHI)/FS) =	26.6 degrees		

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	50 degrees
AREA OF TRIAL FAILURE WEDGE	18.5 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	2215.9 pounds
NUMBER OF TRIAL WEDGES ANALYZED	12393 trials
LENGTH OF FAILURE PLANE	6.2 feet
DEPTH OF TENSION CRACK	2.2 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	4.0 feet
CALCULATED HORIZONTAL THRUST	**-9.9 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**-0.4 pcf**
MAXIMUM HEIGHT OF TEMPORARY EXCAVATION	**7.0 feet**

Shoring Piles (15 Feet High with Level Backslope)

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:	Qal	RETAINED LENGTH	15 feet
SHEAR DIAGRAM:	B-5@10	BACKSLOPE ANGLE:	0 degrees
COHESION:	200 psf	SURCHARGE:	0 pounds
PHI ANGLE:	32 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):	160.0 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 26.6 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	65.8 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	7895.0 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	13.2 feet
DEPTH OF TENSION CRACK	3.8 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	7.0 feet
CALCULATED THRUST ON PILE	**2610.9 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**23.2 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**pcf**

Shoring Piles (25 Feet High with Level Backslope)

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:	Qal	RETAINED LENGTH	25 feet
SHEAR DIAGRAM:	B-5@10	BACKSLOPE ANGLE:	0 degrees
COHESION:	200 psf	SURCHARGE:	0 pounds
PHI ANGLE:	32 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):	160.0 psf	FINAL TENSION CRACK:	40 feet

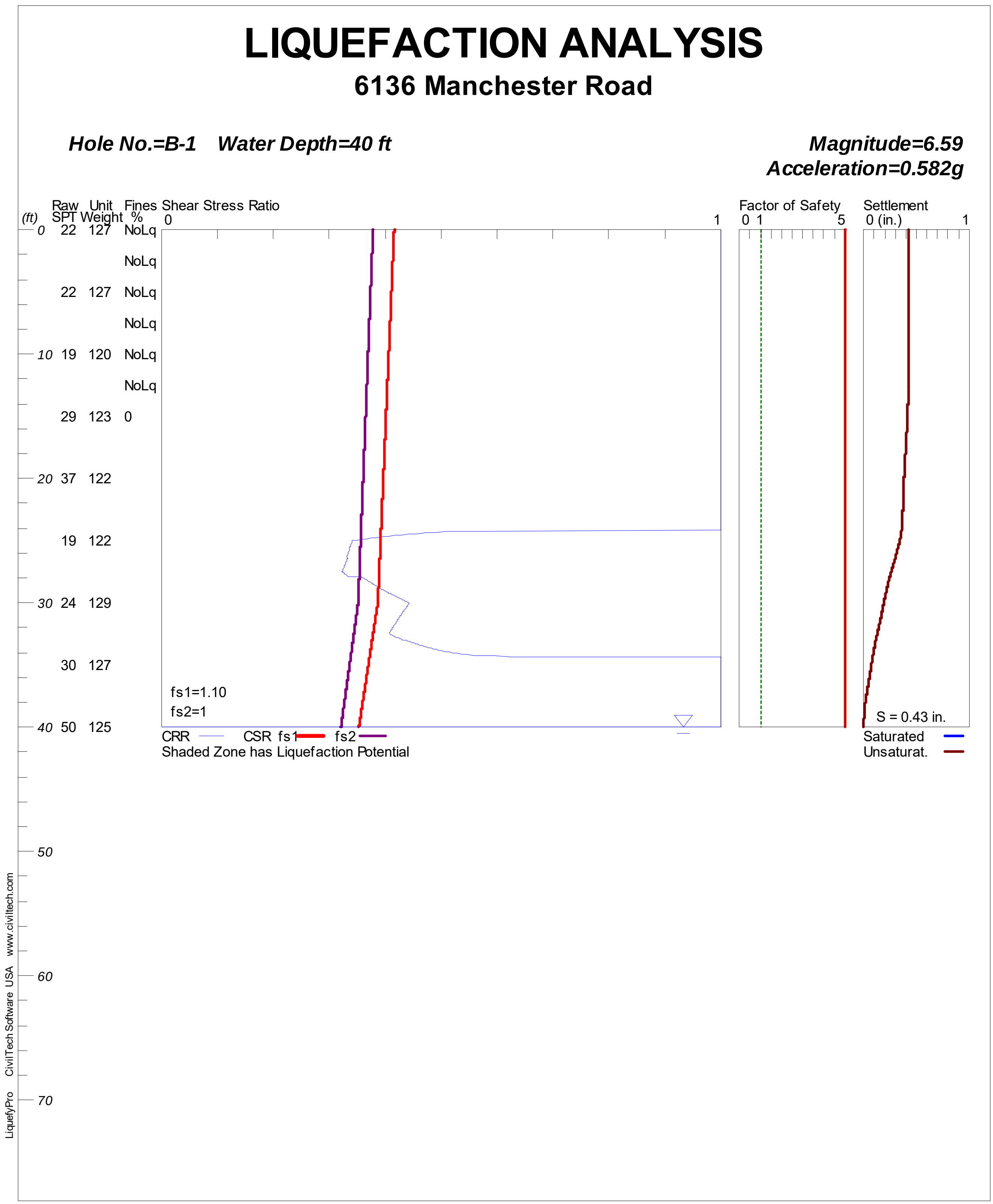
PHID = ATAN(TAN(PHI)/FS) = 26.6 degrees

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	189.8 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	22772.6 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	24.5 feet
DEPTH OF TENSION CRACK	4.2 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	13.0 feet
CALCULATED THRUST ON PILE	**9807.1 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**31.4 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**pcf**


LIQUEFACTION ANALYSIS
6136 Manchester Road
Hole No.=B-1 Water Depth=40 ft
Magnitude=6.59
Acceleration=0.582g
Raw SPT
Unit Weight
Fines %
(ft)
Shear Stress Ratio
0
1
Factor of Safety
0 1 5
Settlement
0 (in.) 1
0 22 127 NoLq
NoLq
22 127 NoLq
NoLq
10 19 120 NoLq
NoLq
29 123 0
20 37 122
19 122
30 24 129
30 127
40 50 125
50
60
70
fs1=1.10
fs2=1
S = 0.43 in.
CRR
CSR fs1
fs2
Shaded Zone has Liquefaction Potential
Saturated
Unsaturat.
LiquefyPro CivilTech Software USA www.civiltech.com

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LIQUEFACTION ANALYSIS CALCULATION DETAILS

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Input File Name: Z:\OUR DOCUMENTS\Liquefaction Analysis\6058-1 B-1.liq
Title: 6136 Manchester Road
Subtitle: 6058

Input Data:
Surface Elev.=
Hole No.=B-1
Depth of Hole=40.00 ft
Water Table during Earthquake= 40.00 ft
Water Table during In-Situ Testing= 40.00 ft
Max. Acceleration=0.58 g
Earthquake Magnitude=6.59
No-Liquefiable Soils: CL, OL are Non-Liq. Soil
1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1.1
Plot two CSR (fs1=User, fs2=1)
10. Average two input data between two Depths: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	Gamma pcf	Fines %
0.00	22.00	127.00	NoLiq
2.50	22.00	127.00	NoLiq
5.00	22.00	127.00	NoLiq
7.50	22.00	127.00	NoLiq
10.00	19.00	120.00	NoLiq
12.50	19.00	120.00	NoLiq
15.00	29.00	123.00	0.00
17.50	29.00	123.00	0.00
20.00	37.00	122.00	0.00
22.50	37.00	122.00	0.00
25.00	19.00	122.00	0.00
27.50	19.00	122.00	0.00
30.00	24.00	129.00	0.00
32.50	24.00	129.00	0.00
35.00	30.00	127.00	0.00
37.50	30.00	127.00	0.00
40.00	50.00	125.00	0.00

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

Peak Ground Acceleration (PGA), a_max = 0.58g

CSR Calculation:

Depth ft	gamma pcf	sigma atm	gamma' pcf	sigma' atm	rd	mZ g	a(z) g	CSR	x fs1	=CSRfs
0.00	127.00	0.000	127.00	0.000	1.00	0.000	0.582	0.38	1.10	0.42
2.00	127.00	0.120	127.00	0.120	1.00	0.000	0.582	0.38	1.10	0.41
4.00	127.00	0.240	127.00	0.240	0.99	0.000	0.582	0.37	1.10	0.41

6.00	127.00	0.360	127.00	0.360	0.99	0.000	0.582	0.37	1.10	0.41
8.00	125.60	0.480	125.60	0.480	0.98	0.000	0.582	0.37	1.10	0.41
10.00	120.00	0.596	120.00	0.596	0.98	0.000	0.582	0.37	1.10	0.41
12.00	120.00	0.710	120.00	0.710	0.97	0.000	0.582	0.37	1.10	0.40
14.00	121.80	0.824	121.80	0.824	0.97	0.000	0.582	0.37	1.10	0.40
16.00	123.00	0.939	123.00	0.939	0.96	0.000	0.582	0.36	1.10	0.40
18.00	122.80	1.056	122.80	1.056	0.96	0.000	0.582	0.36	1.10	0.40
20.00	122.00	1.171	122.00	1.171	0.95	0.000	0.582	0.36	1.10	0.40
22.00	122.00	1.287	122.00	1.287	0.95	0.000	0.582	0.36	1.10	0.39
24.00	122.00	1.402	122.00	1.402	0.94	0.000	0.582	0.36	1.10	0.39
26.00	122.00	1.517	122.00	1.517	0.94	0.000	0.582	0.36	1.10	0.39
28.00	123.40	1.633	123.40	1.633	0.93	0.000	0.582	0.35	1.10	0.39
30.00	129.00	1.752	129.00	1.752	0.93	0.000	0.582	0.35	1.10	0.39
32.00	129.00	1.874	129.00	1.874	0.91	0.000	0.582	0.35	1.10	0.38
34.00	127.80	1.995	127.80	1.995	0.90	0.000	0.582	0.34	1.10	0.37
36.00	127.00	2.116	127.00	2.116	0.88	0.000	0.582	0.33	1.10	0.37
38.00	126.60	2.236	126.60	2.236	0.86	0.000	0.582	0.33	1.10	0.36
40.00	125.00	2.354	125.00	2.354	0.85	0.000	0.582	0.32	1.10	0.35

CSR is based on water table at 40.00 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
0.00	22.00	1.50	0.75	0.000	1.70	42.08	NoLiq	7.20	49.28	2.00
2.00	22.00	1.50	0.75	0.120	1.70	42.08	NoLiq	7.20	49.28	2.00
4.00	22.00	1.50	0.75	0.240	1.70	42.08	NoLiq	7.20	49.28	2.00
6.00	22.00	1.50	0.75	0.360	1.67	41.24	NoLiq	7.20	48.44	2.00
8.00	21.40	1.50	0.75	0.480	1.44	34.75	NoLiq	7.20	41.95	2.00
10.00	19.00	1.50	0.85	0.596	1.30	31.38	NoLiq	7.20	38.58	2.00
12.00	19.00	1.50	0.85	0.710	1.19	28.76	NoLiq	7.20	35.96	2.00
14.00	25.00	1.50	0.85	0.824	1.10	35.12	40.40	7.20	42.32	2.00
16.00	29.00	1.50	0.95	0.939	1.03	42.64	0.00	0.00	42.64	2.00
18.00	30.60	1.50	0.95	1.056	0.97	42.44	0.00	0.00	42.44	2.00
20.00	37.00	1.50	0.95	1.171	0.92	48.72	0.00	0.00	48.72	2.00
22.00	37.00	1.50	0.95	1.287	0.88	46.48	0.00	0.00	46.48	2.00
24.00	26.20	1.50	0.95	1.402	0.84	31.53	0.00	0.00	31.53	2.00
26.00	19.00	1.50	0.95	1.517	0.81	21.98	0.00	0.00	21.98	0.24
28.00	20.00	1.50	1.00	1.633	0.78	23.48	0.00	0.00	23.48	0.26
30.00	24.00	1.50	1.00	1.752	0.76	27.20	0.00	0.00	27.20	0.32
32.00	24.00	1.50	1.00	1.874	0.73	26.30	0.00	0.00	26.30	0.31
34.00	27.60	1.50	1.00	1.995	0.71	29.31	0.00	0.00	29.31	0.39
36.00	30.00	1.50	1.00	2.116	0.69	30.94	0.00	0.00	30.94	2.00
38.00	34.00	1.50	1.00	2.236	0.67	34.11	0.00	0.00	34.11	2.00
40.00	50.00	1.50	1.00	2.354	0.65	48.88	0.00	0.00	48.88	2.00

CRR is based on water table at 40.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 6.59:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
0.00	0.00	2.00	1.00	2.00	1.39	2.00	0.42	5.00 ^
2.00	0.08	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
4.00	0.16	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
6.00	0.23	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
8.00	0.31	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
10.00	0.39	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
12.00	0.46	2.00	1.00	2.00	1.39	2.00	0.40	5.00 ^
14.00	0.54	2.00	1.00	2.00	1.39	2.78	0.40	5.00
16.00	0.61	2.00	1.00	2.00	1.39	2.78	0.40	5.00
18.00	0.69	2.00	1.00	2.00	1.39	2.78	0.40	5.00
20.00	0.76	2.00	1.00	2.00	1.39	2.78	0.40	5.00
22.00	0.84	2.00	1.00	2.00	1.39	2.78	0.39	5.00
24.00	0.91	2.00	1.00	2.00	1.39	2.78	0.39	5.00
26.00	0.99	0.24	1.00	0.24	1.39	0.33	0.39	5.00
28.00	1.06	0.26	1.00	0.26	1.39	0.36	0.39	5.00
30.00	1.14	0.32	0.98	0.32	1.39	0.44	0.39	5.00
32.00	1.22	0.31	0.97	0.30	1.39	0.41	0.38	5.00
34.00	1.30	0.39	0.96	0.37	1.39	0.52	0.37	5.00
36.00	1.38	2.00	0.95	1.90	1.39	2.64	0.37	5.00
38.00	1.45	2.00	0.94	1.88	1.39	2.61	0.36	5.00

40.00	1.53	2.00	0.93	1.86	1.39	2.59	0.35	5.00

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

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

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
0.00	-	-	-	49.28	NoLiq	0.00	49.28
2.00	-	-	-	49.28	NoLiq	0.00	49.28
4.00	-	-	-	49.28	NoLiq	0.00	49.28
6.00	-	-	-	48.44	NoLiq	0.00	48.44
8.00	-	-	-	41.95	NoLiq	0.00	41.95
10.00	-	-	-	38.58	NoLiq	0.00	38.58
12.00	-	-	-	35.96	NoLiq	0.00	35.96
14.00	-	-	-	42.32	40.40	0.00	42.32
16.00	-	-	-	42.64	0.00	0.00	42.64
18.00	-	-	-	42.44	0.00	0.00	42.44
20.00	-	-	-	48.72	0.00	0.00	48.72
22.00	-	-	-	46.48	0.00	0.00	46.48
24.00	-	-	-	31.53	0.00	0.00	31.53
26.00	-	-	-	21.98	0.00	0.00	21.98
28.00	-	-	-	23.48	0.00	0.00	23.48
30.00	-	-	-	27.20	0.00	0.00	27.20
32.00	-	-	-	26.30	0.00	0.00	26.30
34.00	-	-	-	29.31	0.00	0.00	29.31
36.00	-	-	-	30.94	0.00	0.00	30.94
38.00	-	-	-	34.11	0.00	0.00	34.11
40.00	-	-	-	48.88	0.00	0.00	48.88

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

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

Depth ft	CSRsf	/ MSF*	=CSRm	F.S.	Fines %	(N1)60s	Dr %	ec %	dsz in.	dsp in.	S in.
No Settlement of Saturated Sands											

Settlement of Saturated Sands=0.000 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

Depth ft	sigma' atm	sigC' atm	(N1)60s	CSRsf	Gmax atm	g*Ge/Gm	g_eff	ec7.5 %	Cec	ec %	dsz in.	dsp in.	S in.
39.95	2.35	1.53	48.51	0.35	2013.93	4.1E-4	0.0836	0.0264	0.80	0.0212	2.55E-4	0.000	0.000
38.00	2.24	1.45	34.11	0.36	1746.24	4.6E-4	0.1681	0.0770	0.80	0.0618	7.42E-4	0.017	0.017
36.00	2.12	1.38	30.94	0.37	1644.46	4.7E-4	0.1809	0.0982	0.80	0.0788	9.45E-4	0.038	0.056
34.00	2.00	1.30	29.31	0.37	1568.51	4.8E-4	0.1851	0.1091	0.80	0.0876	1.05E-3	0.038	0.093
32.00	1.87	1.22	26.30	0.38	1466.15	4.9E-4	0.1992	0.1369	0.80	0.1099	1.32E-3	0.049	0.143
30.00	1.75	1.14	27.20	0.39	1433.61	4.7E-4	0.1825	0.1197	0.80	0.0961	1.15E-3	0.049	0.192
28.00	1.63	1.06	23.48	0.39	1317.83	4.8E-4	0.1939	0.1550	0.80	0.1244	1.49E-3	0.053	0.245
26.00	1.52	0.99	21.98	0.39	1242.81	4.8E-4	0.1879	0.1636	0.80	0.1313	1.58E-3	0.068	0.313

24.00	1.40	0.91	31.53	0.39	1347.19	4.1E-4	0.1192	0.0627	0.80	0.0504	6.04E-4	0.048	0.361
22.00	1.29	0.84	46.48	0.39	1468.63	3.5E-4	0.0788	0.0249	0.80	0.0200	2.40E-4	0.013	0.374
20.00	1.17	0.76	48.72	0.40	1423.36	3.3E-4	0.0693	0.0219	0.80	0.0176	2.11E-4	0.009	0.383
18.00	1.06	0.69	42.44	0.40	1290.59	3.3E-4	0.1171	0.0370	0.80	0.0297	3.57E-4	0.013	0.397
16.00	0.94	0.61	42.64	0.40	1219.35	3.1E-4	0.0964	0.0305	0.80	0.0245	2.94E-4	0.013	0.410
14.00	0.82	0.54	42.32	0.40	1138.85	2.9E-4	0.0798	0.0252	0.80	0.0203	2.43E-4	0.011	0.420
12.00	0.71	0.46	35.96	0.40	1001.23	2.9E-4	0.0761	0.0314	0.80	0.0252	0.00E0	0.008	0.428
10.00	0.60	0.39	38.58	0.41	939.45	2.6E-4	0.0568	0.0198	0.80	0.0159	0.00E0	0.000	0.428
8.00	0.48	0.31	41.95	0.41	866.86	2.3E-4	0.0501	0.0159	0.80	0.0127	0.00E0	0.000	0.428
6.00	0.36	0.23	48.44	0.41	787.71	1.9E-4	0.0361	0.0114	0.80	0.0092	0.00E0	0.000	0.428
4.00	0.24	0.16	49.28	0.41	646.81	1.5E-4	0.0272	0.0086	0.80	0.0069	0.00E0	0.000	0.428
2.00	0.12	0.08	49.28	0.41	457.38	1.1E-4	0.0226	0.0072	0.80	0.0057	0.00E0	0.000	0.428
0.00	0.00	0.00	49.28	0.42	4.17	1.0E-6	0.0010	0.0003	0.80	0.0003	0.00E0	0.000	0.428

Settlement of Unsaturated Sands=0.428 in.
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Unsaturated Sands=0.428 in.
Differential Settlement=0.214 to 0.283 in.

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

1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)

SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT) [atm (tsf)]
fs	Friction from CPT testing [atm (tsf)]
Rf	Ratio of fs/qc (%)
gamma	Total unit weight of soil
gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size
Dr	Relative Density
sigma	Total vertical stress [atm]
sigma'	Effective vertical stress [atm]
sigC'	Effective confining pressure [atm]
rd	Acceleration reduction coefficient by Seed
a_max.	Peak Ground Acceleration (PGA) in ground surface
mZ	Linear acceleration reduction coefficient X depth
a_min.	Minimum acceleration under linear reduction, mZ
CRRv	CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksig	Overburden stress correction factor for CRR7.5
CRRm	After magnitude scaling correction CRRm=CRRv * MSF
MSF	Magnitude scaling factor from M=7.5 to user input M
CSR	Cyclic stress ratio induced by earthquake
CSRfs	CSRfs=CSR*fs1 (Default fs1=1)
fs1	First CSR curve in graphic defined in #9 of Advanced page
fs2	2nd CSR curve in graphic defined in #9 of Advanced page
F.S.	Calculated factor of safety against liquefaction F.S.=CRRm/CSRsf
Cebs	Energy Ratio, Borehole Dia., and Sampling Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60	Fines correction of SPT

(N1)60f	(N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq	Overburden stress correction factor
qc1	CPT after Overburden stress correction
dqc1	Fines correction of CPT
qc1f	CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n	CPT after normalization in Robertson's method
Kc	Fine correction factor in Robertson's Method
qc1f	CPT after Fines correction in Robertson's Method
Ic	Soil type index in Suzuki's and Robertson's Methods
(N1)60s	(N1)60 after settlement fines corrections
CSRm	After magnitude scaling correction for Settlement calculation CSRm=CSRsf / MSF*
CSRfs	Cyclic stress ratio induced by earthquake with user inputed fs
MSF*	Scaling factor from CSR, MSF*=1, based on Item 2 of Page C.
ec	Volumetric strain for saturated sands
dz	Calculation segment, dz=0.050 ft
dsz	Settlement in each segment, dz
dp	User defined print interval
dsp	Settlement in each print interval, dp
Gmax	Shear Modulus at low strain
g_eff	gamma_eff, Effective shear Strain
g*Ge/Gm	gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5	Volumetric Strain for magnitude=7.5
Cec	Magnitude correction factor for any magnitude
ec	Volumetric strain for unsaturated sands, ec=Cec * ec7.5
NoLiq	No-Liquefy Soils

References:

1. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.

SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.

2. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING AND SEISMIC SITE RESPONSE EVALUATION, Paper No. SPL-2, PROCEEDINGS: Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA, March 2001.

3. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING: A UNIFIED AND CONSISTENT FRAMEWORK, Earthquake Engineering Research Center, Report No. EERC 2003-06 by R.B Seed and etc. April 2003.

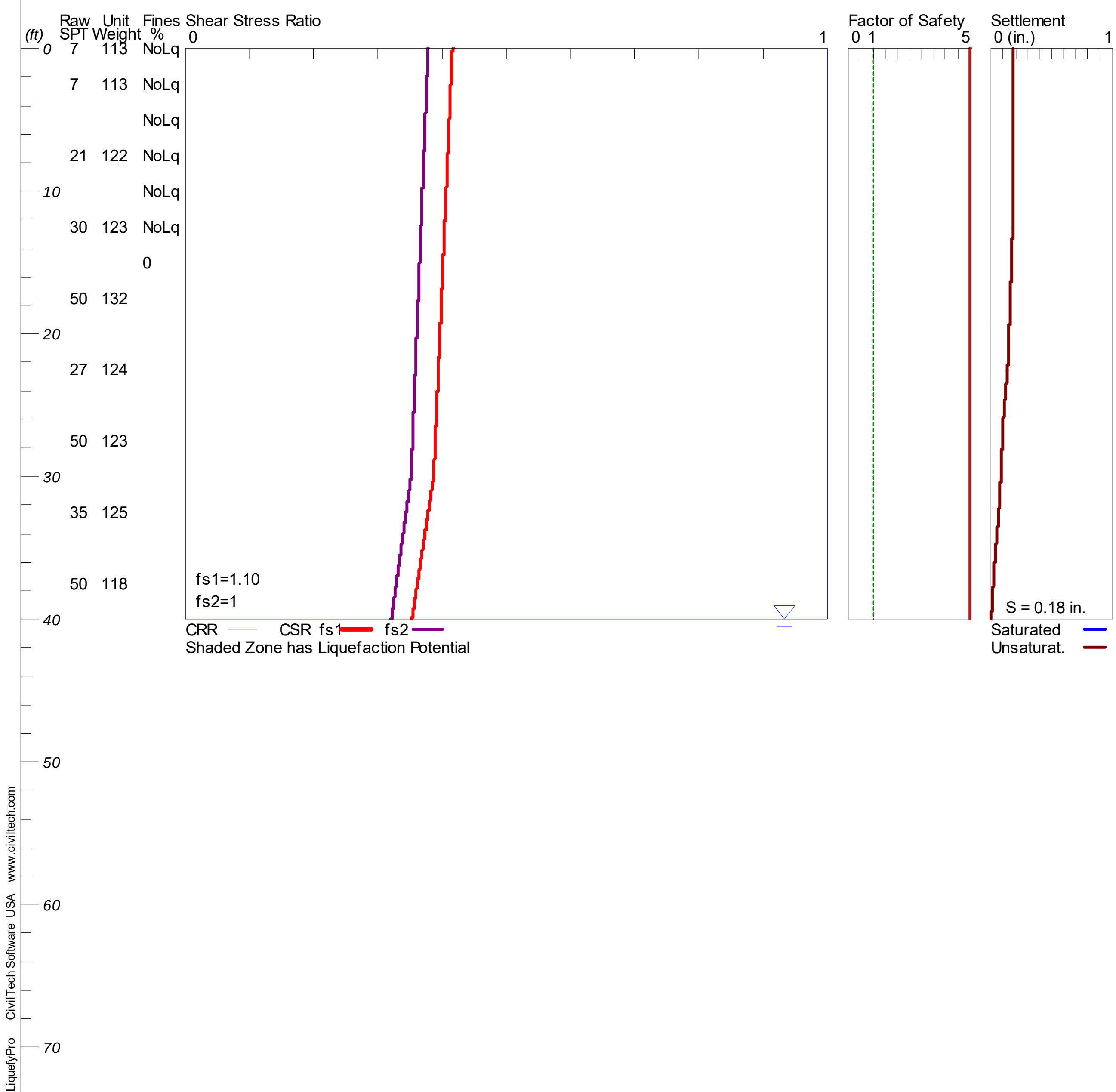
Note: Print Interval you selected does not show complete results. To get complete results, you should select 'Segment' in Print Interval (Item 12, Page C).

LIQUEFACTION ANALYSIS

6136 Manchester Road

Hole No.=B-2 Water Depth=40 ft

Magnitude=6.59
Acceleration=0.582g



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

*******************************************************************************************************
       Font: Courier New, Regular, Size 8 is recommended for this report.
       Licensed to ,  2/3/2022       2:26:21 PM

       Input File Name: Z:\OUR DOCUMENTS\Liquefaction Analysis\6058-1 B-2.liq
       Title:  6136 Manchester Road
       Subtitle:  6058

 Input Data:
       Surface Elev.=
       Hole No.=B-2
       Depth of Hole=40.00 ft
       Water Table during Earthquake= 40.00 ft
       Water Table during In-Situ Testing= 40.00 ft
       Max. Acceleration=0.58 g
       Earthquake Magnitude=6.59
       No-Liquefiable Soils:   CL, OL are Non-Liq. Soil
       1. SPT or BPT Calculation.
       2. Settlement Analysis Method: Ishihara / Yoshimine
       3. Fines Correction for Liquefaction: Stark/Olson et al.*
       4. Fine Correction for Settlement: During Liquefaction*
       5. Settlement Calculation in: All zones*
       6. Hammer Energy Ratio,                                   Ce = 1.25
       7. Borehole Diameter,                                         Cb= 1
       8. Sampling Method,                                         Cs= 1.2
       9. User request factor of safety (apply to CSR) ,   User= 1.1
          Plot two CSR (fs1=User, fs2=1)
       10. Average two input data between two Depths: Yes*
       * Recommended Options

       In-Situ Test Data:
```

Depth ft	SPT	Gamma pcf	Fines %
0.00	7.00	113.00	NoLiq
2.50	7.00	113.00	NoLiq
5.00	7.00	113.00	NoLiq
7.50	21.00	122.00	NoLiq
10.00	21.00	122.00	NoLiq
12.50	30.00	123.00	NoLiq
15.00	30.00	123.00	0.00
17.50	50.00	132.00	0.00
20.00	50.00	132.00	0.00
22.50	27.00	124.00	0.00
25.00	27.00	124.00	0.00
27.50	50.00	123.00	0.00
30.00	50.00	123.00	0.00
32.50	35.00	125.00	0.00
35.00	35.00	125.00	0.00
37.50	50.00	118.00	0.00
40.00	50.00	118.00	0.00

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

       Peak Ground Acceleration (PGA), a_max = 0.58g

       CSR Calculation:
```

Depth ft	gamma pcf	sigma atm	gamma' pcf	sigma' atm	rd	mZ g	a(z) g	CSR	x fs1	=CSRfs
0.00	113.00	0.000	113.00	0.000	1.00	0.000	0.582	0.38	1.10	0.42
2.00	113.00	0.107	113.00	0.107	1.00	0.000	0.582	0.38	1.10	0.41
4.00	113.00	0.214	113.00	0.214	0.99	0.000	0.582	0.37	1.10	0.41

6.00	116.60	0.321	116.60	0.321	0.99	0.000	0.582	0.37	1.10	0.41
8.00	122.00	0.435	122.00	0.435	0.98	0.000	0.582	0.37	1.10	0.41
10.00	122.00	0.550	122.00	0.550	0.98	0.000	0.582	0.37	1.10	0.41
12.00	122.80	0.665	122.80	0.665	0.97	0.000	0.582	0.37	1.10	0.40
14.00	123.00	0.782	123.00	0.782	0.97	0.000	0.582	0.37	1.10	0.40
16.00	126.60	0.899	126.60	0.899	0.96	0.000	0.582	0.36	1.10	0.40
18.00	132.00	1.022	132.00	1.022	0.96	0.000	0.582	0.36	1.10	0.40
20.00	132.00	1.146	132.00	1.146	0.95	0.000	0.582	0.36	1.10	0.40
22.00	125.60	1.268	125.60	1.268	0.95	0.000	0.582	0.36	1.10	0.39
24.00	124.00	1.386	124.00	1.386	0.94	0.000	0.582	0.36	1.10	0.39
26.00	123.60	1.503	123.60	1.503	0.94	0.000	0.582	0.36	1.10	0.39
28.00	123.00	1.619	123.00	1.619	0.93	0.000	0.582	0.35	1.10	0.39
30.00	123.00	1.735	123.00	1.735	0.93	0.000	0.582	0.35	1.10	0.39
32.00	124.60	1.852	124.60	1.852	0.91	0.000	0.582	0.35	1.10	0.38
34.00	125.00	1.970	125.00	1.970	0.90	0.000	0.582	0.34	1.10	0.37
36.00	122.20	2.088	122.20	2.088	0.88	0.000	0.582	0.33	1.10	0.37
38.00	118.00	2.201	118.00	2.201	0.86	0.000	0.582	0.33	1.10	0.36
40.00	118.00	2.312	118.00	2.312	0.85	0.000	0.582	0.32	1.10	0.35

CSR is based on water table at 40.00 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
0.00	7.00	1.50	0.75	0.000	1.70	13.39	NoLiq	7.20	20.59	0.22
2.00	7.00	1.50	0.75	0.107	1.70	13.39	NoLiq	7.20	20.59	0.22
4.00	7.00	1.50	0.75	0.214	1.70	13.39	NoLiq	7.20	20.59	0.22
6.00	12.60	1.50	0.75	0.321	1.70	24.10	NoLiq	7.20	31.30	2.00
8.00	21.00	1.50	0.75	0.435	1.52	35.84	NoLiq	7.20	43.04	2.00
10.00	21.00	1.50	0.85	0.550	1.35	36.11	NoLiq	7.20	43.31	2.00
12.00	28.20	1.50	0.85	0.665	1.23	44.07	NoLiq	7.20	51.27	2.00
14.00	30.00	1.50	0.85	0.782	1.13	43.26	40.40	7.20	50.46	2.00
16.00	38.00	1.50	0.95	0.899	1.05	57.12	0.00	0.00	57.12	2.00
18.00	50.00	1.50	0.95	1.022	0.99	70.49	0.00	0.00	70.49	2.00
20.00	50.00	1.50	0.95	1.146	0.93	66.55	0.00	0.00	66.55	2.00
22.00	31.60	1.50	0.95	1.268	0.89	39.99	0.00	0.00	39.99	2.00
24.00	27.00	1.50	0.95	1.386	0.85	32.69	0.00	0.00	32.69	2.00
26.00	36.20	1.50	0.95	1.503	0.82	42.08	0.00	0.00	42.08	2.00
28.00	50.00	1.50	1.00	1.619	0.79	58.94	0.00	0.00	58.94	2.00
30.00	50.00	1.50	1.00	1.735	0.76	56.93	0.00	0.00	56.93	2.00
32.00	38.00	1.50	1.00	1.852	0.73	41.88	0.00	0.00	41.88	2.00
34.00	35.00	1.50	1.00	1.970	0.71	37.40	0.00	0.00	37.40	2.00
36.00	41.00	1.50	1.00	2.088	0.69	42.56	0.00	0.00	42.56	2.00
38.00	50.00	1.50	1.00	2.201	0.67	50.55	0.00	0.00	50.55	2.00
40.00	50.00	1.50	1.00	2.312	0.66	49.32	0.00	0.00	49.32	2.00

CRR is based on water table at 40.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 6.59:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
0.00	0.00	0.22	1.00	0.22	1.39	2.00	0.42	5.00 ^
2.00	0.07	0.22	1.00	0.22	1.39	2.00	0.41	5.00 ^
4.00	0.14	0.22	1.00	0.22	1.39	2.00	0.41	5.00 ^
6.00	0.21	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
8.00	0.28	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
10.00	0.36	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
12.00	0.43	2.00	1.00	2.00	1.39	2.00	0.40	5.00 ^
14.00	0.51	2.00	1.00	2.00	1.39	2.78	0.40	5.00
16.00	0.58	2.00	1.00	2.00	1.39	2.78	0.40	5.00
18.00	0.66	2.00	1.00	2.00	1.39	2.78	0.40	5.00
20.00	0.75	2.00	1.00	2.00	1.39	2.78	0.40	5.00
22.00	0.82	2.00	1.00	2.00	1.39	2.78	0.39	5.00
24.00	0.90	2.00	1.00	2.00	1.39	2.78	0.39	5.00
26.00	0.98	2.00	1.00	2.00	1.39	2.78	0.39	5.00
28.00	1.05	2.00	1.00	2.00	1.39	2.78	0.39	5.00
30.00	1.13	2.00	0.99	1.97	1.39	2.75	0.39	5.00
32.00	1.20	2.00	0.97	1.95	1.39	2.71	0.38	5.00
34.00	1.28	2.00	0.96	1.93	1.39	2.68	0.37	5.00
36.00	1.36	2.00	0.95	1.90	1.39	2.65	0.37	5.00
38.00	1.43	2.00	0.94	1.88	1.39	2.62	0.36	5.00

40.00	1.50	2.00	0.93	1.86	1.39	2.60	0.35	5.00

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

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

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
0.00	-	-	-	20.59	NoLiq	0.00	20.59
2.00	-	-	-	20.59	NoLiq	0.00	20.59
4.00	-	-	-	20.59	NoLiq	0.00	20.59
6.00	-	-	-	31.30	NoLiq	0.00	31.30
8.00	-	-	-	43.04	NoLiq	0.00	43.04
10.00	-	-	-	43.31	NoLiq	0.00	43.31
12.00	-	-	-	51.27	NoLiq	0.00	51.27
14.00	-	-	-	50.46	40.40	0.00	50.46
16.00	-	-	-	57.12	0.00	0.00	57.12
18.00	-	-	-	70.49	0.00	0.00	70.49
20.00	-	-	-	66.55	0.00	0.00	66.55
22.00	-	-	-	39.99	0.00	0.00	39.99
24.00	-	-	-	32.69	0.00	0.00	32.69
26.00	-	-	-	42.08	0.00	0.00	42.08
28.00	-	-	-	58.94	0.00	0.00	58.94
30.00	-	-	-	56.93	0.00	0.00	56.93
32.00	-	-	-	41.88	0.00	0.00	41.88
34.00	-	-	-	37.40	0.00	0.00	37.40
36.00	-	-	-	42.56	0.00	0.00	42.56
38.00	-	-	-	50.55	0.00	0.00	50.55
40.00	-	-	-	49.32	0.00	0.00	49.32

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

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

Depth ft	CSRsf	/ MSF*	=CSRm	F.S.	Fines %	(N1)60s	Dr %	ec %	dsz in.	dsp in.	S in.
No Settlement of Saturated Sands											

Settlement of Saturated Sands=0.000 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

Depth ft	sigma' atm	sigC' atm	(N1)60s	CSRsf	Gmax atm	g*Ge/Gm	g_eff	ec7.5 %	Cec	ec %	dsz in.	dsp in.	S in.
39.95	2.31	1.50	49.35	0.35	2007.30	4.1E-4	0.0811	0.0257	0.80	0.0206	2.47E-4	0.000	0.000
38.00	2.20	1.43	50.55	0.36	1975.28	4.0E-4	0.1133	0.0358	0.80	0.0287	3.45E-4	0.014	0.014
36.00	2.09	1.36	42.56	0.37	1816.70	4.2E-4	0.1295	0.0409	0.80	0.0329	3.94E-4	0.014	0.028
34.00	1.97	1.28	37.40	0.37	1690.51	4.4E-4	0.1419	0.0535	0.80	0.0429	5.15E-4	0.020	0.048
32.00	1.85	1.20	41.88	0.38	1702.02	4.1E-4	0.1232	0.0389	0.80	0.0313	3.75E-4	0.019	0.067
30.00	1.74	1.13	56.93	0.39	1824.74	3.7E-4	0.0912	0.0289	0.80	0.0232	2.78E-4	0.013	0.079
28.00	1.62	1.05	58.94	0.39	1783.02	3.5E-4	0.0827	0.0262	0.80	0.0210	2.52E-4	0.011	0.090
26.00	1.50	0.98	42.08	0.39	1535.38	3.8E-4	0.1004	0.0317	0.80	0.0255	3.06E-4	0.011	0.101

24.00	1.39	0.90	32.69	0.39	1355.38	4.0E-4	0.1137	0.0563	0.80	0.0452	5.42E-4	0.020 0.121
22.00	1.27	0.82	39.99	0.39	1386.73	3.6E-4	0.0871	0.0275	0.80	0.0221	2.65E-4	0.018 0.139
20.00	1.15	0.75	66.55	0.40	1562.16	2.9E-4	0.0798	0.0252	0.80	0.0203	2.43E-4	0.009 0.148
18.00	1.02	0.66	70.49	0.40	1503.27	2.7E-4	0.0647	0.0205	0.80	0.0164	1.97E-4	0.009 0.157
16.00	0.90	0.58	57.12	0.40	1314.63	2.7E-4	0.0667	0.0211	0.80	0.0169	2.03E-4	0.008 0.164
14.00	0.78	0.51	50.46	0.40	1176.48	2.7E-4	0.0625	0.0198	0.80	0.0159	1.90E-4	0.009 0.173
12.00	0.67	0.43	51.27	0.40	1091.30	2.5E-4	0.0509	0.0161	0.80	0.0129	0.00E0	0.005 0.178
10.00	0.55	0.36	43.31	0.41	937.71	2.4E-4	0.0470	0.0148	0.80	0.0119	0.00E0	0.000 0.178
8.00	0.43	0.28	43.04	0.41	831.88	2.1E-4	0.0433	0.0137	0.80	0.0110	0.00E0	0.000 0.178
6.00	0.32	0.21	31.30	0.41	643.23	2.0E-4	0.0402	0.0214	0.80	0.0172	0.00E0	0.000 0.178
4.00	0.21	0.14	20.59	0.41	456.25	1.9E-4	0.0510	0.0484	0.80	0.0388	0.00E0	0.000 0.178
2.00	0.11	0.07	20.59	0.41	322.62	1.4E-4	0.0256	0.0243	0.80	0.0195	0.00E0	0.000 0.178
0.00	0.00	0.00	20.59	0.42	3.12	1.3E-6	0.0010	0.0010	0.80	0.0008	0.00E0	0.000 0.178

Settlement of Unsaturated Sands=0.178 in.
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Unsaturated Sands=0.178 in.
Differential Settlement=0.089 to 0.117 in.

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

1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)

SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT) [atm (tsf)]
fs	Friction from CPT testing [atm (tsf)]
Rf	Ratio of fs/qc (%)
gamma	Total unit weight of soil
gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size
Dr	Relative Density
sigma	Total vertical stress [atm]
sigma'	Effective vertical stress [atm]
sigC'	Effective confining pressure [atm]
rd	Acceleration reduction coefficient by Seed
a_max.	Peak Ground Acceleration (PGA) in ground surface
mZ	Linear acceleration reduction coefficient X depth
a_min.	Minimum acceleration under linear reduction, mZ
CRRv	CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksig	Overburden stress correction factor for CRR7.5
CRRm	After magnitude scaling correction CRRm=CRRv * MSF
MSF	Magnitude scaling factor from M=7.5 to user input M
CSR	Cyclic stress ratio induced by earthquake
CSRfs	CSRfs=CSR*fs1 (Default fs1=1)
fs1	First CSR curve in graphic defined in #9 of Advanced page
fs2	2nd CSR curve in graphic defined in #9 of Advanced page
F.S.	Calculated factor of safety against liquefaction F.S.=CRRm/CSRsf
Cebs	Energy Ratio, Borehole Dia., and Sampling Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60	Fines correction of SPT

(N1)60f (N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq Overburden stress correction factor
qc1 CPT after Overburden stress correction
dqc1 Fines correction of CPT
qc1f CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n CPT after normalization in Robertson's method
Kc Fine correction factor in Robertson's Method
qc1f CPT after Fines correction in Robertson's Method
Ic Soil type index in Suzuki's and Robertson's Methods
(N1)60s (N1)60 after settlement fines corrections
CSRm After magnitude scaling correction for Settlement calculation CSRm=CSRsf / MSF*
CSRfs Cyclic stress ratio induced by earthquake with user inputed fs
MSF* Scaling factor from CSR, MSF*=1, based on Item 2 of Page C.
ec Volumetric strain for saturated sands
dz Calculation segment, dz=0.050 ft
dsz Settlement in each segment, dz
dp User defined print interval
dsp Settlement in each print interval, dp
Gmax Shear Modulus at low strain
g_eff gamma_eff, Effective shear Strain
g*Ge/Gm gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5 Volumetric Strain for magnitude=7.5
Cec Magnitude correction factor for any magnitude
ec Volumetric strain for unsaturated sands, ec=Cec * ec7.5
NoLiq No-Liquefy Soils

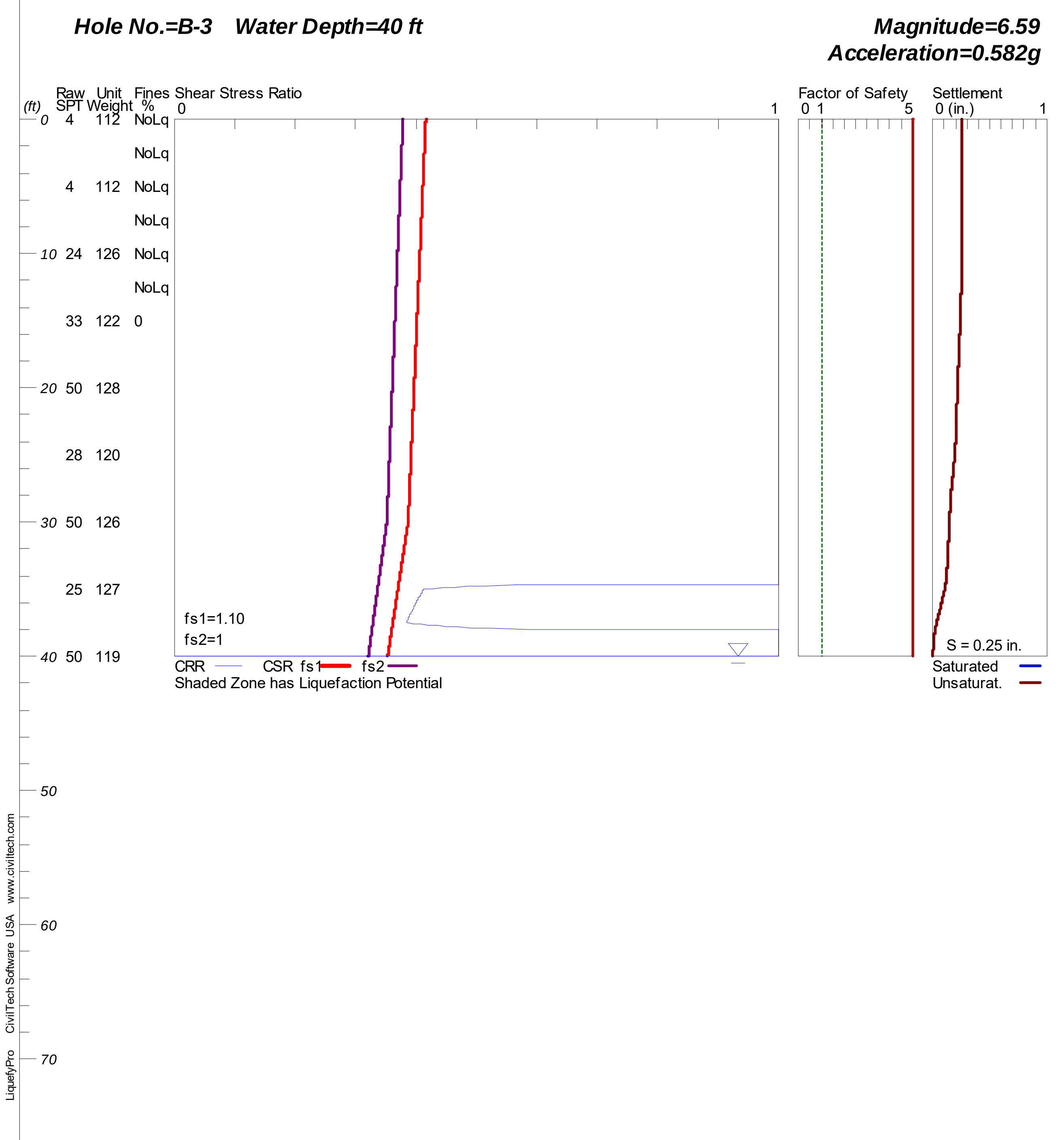
References:

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

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

LIQUEFACTION ANALYSIS

6136 Manchester Road



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LIQUEFACTION ANALYSIS CALCULATION DETAILS

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Input File Name: Z:\OUR DOCUMENTS\Liquefaction Analysis\6058-1 B-3.liq
Title: 6136 Manchester Road
Subtitle: 6058

Input Data:
Surface Elev.=
Hole No.=B-3
Depth of Hole=40.00 ft
Water Table during Earthquake= 40.00 ft
Water Table during In-Situ Testing= 40.00 ft
Max. Acceleration=0.58 g
Earthquake Magnitude=6.59
No-Liquefiable Soils: CL, OL are Non-Liq. Soil
1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1.1
Plot two CSR (fs1=User, fs2=1)
10. Average two input data between two Depths: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	Gamma pcf	Fines %
0.00	4.00	112.00	NoLiq
2.50	4.00	112.00	NoLiq
5.00	4.00	112.00	NoLiq
7.50	4.00	112.00	NoLiq
10.00	24.00	126.00	NoLiq
12.50	24.00	126.00	NoLiq
15.00	33.00	122.00	0.00
17.50	33.00	122.00	0.00
20.00	50.00	128.00	0.00
22.50	50.00	128.00	0.00
25.00	28.00	120.00	0.00
27.50	28.00	120.00	0.00
30.00	50.00	126.00	0.00
32.50	50.00	126.00	0.00
35.00	25.00	127.00	0.00
37.50	25.00	127.00	0.00
40.00	50.00	119.00	0.00

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

Peak Ground Acceleration (PGA), a_max = 0.58g

CSR Calculation:

Depth ft	gamma pcf	sigma atm	gamma' pcf	sigma' atm	rd	mZ g	a(z) g	CSR	x fs1	=CSRfs
0.00	112.00	0.000	112.00	0.000	1.00	0.000	0.582	0.38	1.10	0.42
2.00	112.00	0.106	112.00	0.106	1.00	0.000	0.582	0.38	1.10	0.41
4.00	112.00	0.212	112.00	0.212	0.99	0.000	0.582	0.37	1.10	0.41

6.00	112.00	0.318	112.00	0.318	0.99	0.000	0.582	0.37	1.10	0.41
8.00	114.80	0.424	114.80	0.424	0.98	0.000	0.582	0.37	1.10	0.41
10.00	126.00	0.537	126.00	0.537	0.98	0.000	0.582	0.37	1.10	0.41
12.00	126.00	0.656	126.00	0.656	0.97	0.000	0.582	0.37	1.10	0.40
14.00	123.60	0.775	123.60	0.775	0.97	0.000	0.582	0.37	1.10	0.40
16.00	122.00	0.890	122.00	0.890	0.96	0.000	0.582	0.36	1.10	0.40
18.00	123.20	1.006	123.20	1.006	0.96	0.000	0.582	0.36	1.10	0.40
20.00	128.00	1.124	128.00	1.124	0.95	0.000	0.582	0.36	1.10	0.40
22.00	128.00	1.245	128.00	1.245	0.95	0.000	0.582	0.36	1.10	0.39
24.00	123.20	1.365	123.20	1.365	0.94	0.000	0.582	0.36	1.10	0.39
26.00	120.00	1.479	120.00	1.479	0.94	0.000	0.582	0.36	1.10	0.39
28.00	121.20	1.593	121.20	1.593	0.93	0.000	0.582	0.35	1.10	0.39
30.00	126.00	1.709	126.00	1.709	0.93	0.000	0.582	0.35	1.10	0.39
32.00	126.00	1.828	126.00	1.828	0.91	0.000	0.582	0.35	1.10	0.38
34.00	126.60	1.948	126.60	1.948	0.90	0.000	0.582	0.34	1.10	0.37
36.00	127.00	2.068	127.00	2.068	0.88	0.000	0.582	0.33	1.10	0.37
38.00	125.40	2.187	125.40	2.187	0.86	0.000	0.582	0.33	1.10	0.36
40.00	119.00	2.303	119.00	2.303	0.85	0.000	0.582	0.32	1.10	0.35

CSR is based on water table at 40.00 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
0.00	4.00	1.50	0.75	0.000	1.70	7.65	NoLiq	7.20	14.85	0.16
2.00	4.00	1.50	0.75	0.106	1.70	7.65	NoLiq	7.20	14.85	0.16
4.00	4.00	1.50	0.75	0.212	1.70	7.65	NoLiq	7.20	14.85	0.16
6.00	4.00	1.50	0.75	0.318	1.70	7.65	NoLiq	7.20	14.85	0.16
8.00	8.00	1.50	0.75	0.424	1.54	13.83	NoLiq	7.20	21.03	0.23
10.00	24.00	1.50	0.85	0.537	1.36	41.74	NoLiq	7.20	48.94	2.00
12.00	24.00	1.50	0.85	0.656	1.23	37.77	NoLiq	7.20	44.97	2.00
14.00	29.40	1.50	0.85	0.775	1.14	42.59	40.40	7.20	49.79	2.00
16.00	33.00	1.50	0.95	0.890	1.06	49.84	0.00	0.00	49.84	2.00
18.00	36.40	1.50	0.95	1.006	1.00	51.72	0.00	0.00	51.72	2.00
20.00	50.00	1.50	0.95	1.124	0.94	67.19	0.00	0.00	67.19	2.00
22.00	50.00	1.50	0.95	1.245	0.90	63.84	0.00	0.00	63.84	2.00
24.00	36.80	1.50	0.95	1.365	0.86	44.89	0.00	0.00	44.89	2.00
26.00	28.00	1.50	0.95	1.479	0.82	32.81	0.00	0.00	32.81	2.00
28.00	32.40	1.50	1.00	1.593	0.79	38.51	0.00	0.00	38.51	2.00
30.00	50.00	1.50	1.00	1.709	0.76	57.36	0.00	0.00	57.36	2.00
32.00	50.00	1.50	1.00	1.828	0.74	55.47	0.00	0.00	55.47	2.00
34.00	35.00	1.50	1.00	1.948	0.72	37.62	0.00	0.00	37.62	2.00
36.00	25.00	1.50	1.00	2.068	0.70	26.08	0.00	0.00	26.08	0.30
38.00	30.00	1.50	1.00	2.187	0.68	30.42	0.00	0.00	30.42	2.00
40.00	50.00	1.50	1.00	2.303	0.66	49.42	0.00	0.00	49.42	2.00

CRR is based on water table at 40.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 6.59:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
0.00	0.00	0.16	1.00	0.16	1.39	2.00	0.42	5.00 ^
2.00	0.07	0.16	1.00	0.16	1.39	2.00	0.41	5.00 ^
4.00	0.14	0.16	1.00	0.16	1.39	2.00	0.41	5.00 ^
6.00	0.21	0.16	1.00	0.16	1.39	2.00	0.41	5.00 ^
8.00	0.28	0.23	1.00	0.23	1.39	2.00	0.41	5.00 ^
10.00	0.35	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
12.00	0.43	2.00	1.00	2.00	1.39	2.00	0.40	5.00 ^
14.00	0.50	2.00	1.00	2.00	1.39	2.78	0.40	5.00
16.00	0.58	2.00	1.00	2.00	1.39	2.78	0.40	5.00
18.00	0.65	2.00	1.00	2.00	1.39	2.78	0.40	5.00
20.00	0.73	2.00	1.00	2.00	1.39	2.78	0.40	5.00
22.00	0.81	2.00	1.00	2.00	1.39	2.78	0.39	5.00
24.00	0.89	2.00	1.00	2.00	1.39	2.78	0.39	5.00
26.00	0.96	2.00	1.00	2.00	1.39	2.78	0.39	5.00
28.00	1.04	2.00	1.00	2.00	1.39	2.79	0.39	5.00
30.00	1.11	2.00	0.99	1.98	1.39	2.75	0.39	5.00
32.00	1.19	2.00	0.98	1.95	1.39	2.72	0.38	5.00
34.00	1.27	2.00	0.97	1.93	1.39	2.69	0.37	5.00
36.00	1.34	0.30	0.95	0.29	1.39	0.40	0.37	5.00
38.00	1.42	2.00	0.94	1.89	1.39	2.63	0.36	5.00

40.00	1.50	2.00	0.93	1.87	1.39	2.60	0.35	5.00

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

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

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
0.00	-	-	-	14.85	NoLiq	0.00	14.85
2.00	-	-	-	14.85	NoLiq	0.00	14.85
4.00	-	-	-	14.85	NoLiq	0.00	14.85
6.00	-	-	-	14.85	NoLiq	0.00	14.85
8.00	-	-	-	21.03	NoLiq	0.00	21.03
10.00	-	-	-	48.94	NoLiq	0.00	48.94
12.00	-	-	-	44.97	NoLiq	0.00	44.97
14.00	-	-	-	49.79	40.40	0.00	49.79
16.00	-	-	-	49.84	0.00	0.00	49.84
18.00	-	-	-	51.72	0.00	0.00	51.72
20.00	-	-	-	67.19	0.00	0.00	67.19
22.00	-	-	-	63.84	0.00	0.00	63.84
24.00	-	-	-	44.89	0.00	0.00	44.89
26.00	-	-	-	32.81	0.00	0.00	32.81
28.00	-	-	-	38.51	0.00	0.00	38.51
30.00	-	-	-	57.36	0.00	0.00	57.36
32.00	-	-	-	55.47	0.00	0.00	55.47
34.00	-	-	-	37.62	0.00	0.00	37.62
36.00	-	-	-	26.08	0.00	0.00	26.08
38.00	-	-	-	30.42	0.00	0.00	30.42
40.00	-	-	-	49.42	0.00	0.00	49.42

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

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

Depth ft	CSRsf	/ MSF*	=CSRm	F.S.	Fines %	(N1)60s	Dr %	ec %	dsz in.	dsp in.	S in.
No Settlement of Saturated Sands											

Settlement of Saturated Sands=0.000 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

Depth ft	sigma' atm	sigC' atm	(N1)60s	CSRsf	Gmax atm	g*Ge/Gm	g_eff	ec7.5 %	Cec	ec %	dsz in.	dsp in.	S in.
39.95	2.30	1.50	48.95	0.35	1997.80	4.1E-4	0.1176	0.0372	0.80	0.0299	3.58E-4	0.000	0.000
38.00	2.19	1.42	30.42	0.36	1662.82	4.7E-4	0.1830	0.1020	0.80	0.0818	9.82E-4	0.021	0.021
36.00	2.07	1.34	26.08	0.37	1535.79	4.9E-4	0.2099	0.1459	0.80	0.1171	1.41E-3	0.056	0.078
34.00	1.95	1.27	37.62	0.37	1684.02	4.3E-4	0.1388	0.0515	0.80	0.0414	4.96E-4	0.044	0.122
32.00	1.83	1.19	55.47	0.38	1856.80	3.7E-4	0.0951	0.0301	0.80	0.0241	2.90E-4	0.013	0.135
30.00	1.71	1.11	57.36	0.39	1815.54	3.6E-4	0.0891	0.0282	0.80	0.0226	2.71E-4	0.011	0.146
28.00	1.59	1.04	38.51	0.39	1534.66	4.0E-4	0.1152	0.0404	0.80	0.0324	3.89E-4	0.012	0.159
26.00	1.48	0.96	32.81	0.39	1402.10	4.1E-4	0.1220	0.0600	0.80	0.0482	5.78E-4	0.024	0.183

24.00	1.36	0.89	44.89	0.39	1495.08	3.6E-4	0.0857	0.0271	0.80	0.0218	2.61E-4	0.018	0.201
22.00	1.25	0.81	63.84	0.39	1605.98	3.1E-4	0.0605	0.0191	0.80	0.0154	1.84E-4	0.008	0.209
20.00	1.12	0.73	67.19	0.40	1552.18	2.9E-4	0.0768	0.0243	0.80	0.0195	2.34E-4	0.008	0.217
18.00	1.01	0.65	51.72	0.40	1345.49	3.0E-4	0.0859	0.0272	0.80	0.0218	2.62E-4	0.010	0.227
16.00	0.89	0.58	49.84	0.40	1250.39	2.9E-4	0.0751	0.0237	0.80	0.0191	2.29E-4	0.010	0.237
14.00	0.77	0.50	49.79	0.40	1165.96	2.7E-4	0.0625	0.0198	0.80	0.0159	1.90E-4	0.008	0.245
12.00	0.66	0.43	44.97	0.40	1037.50	2.6E-4	0.0557	0.0176	0.80	0.0141	0.00E0	0.005	0.250
10.00	0.54	0.35	48.94	0.41	965.55	2.3E-4	0.0502	0.0159	0.80	0.0127	0.00E0	0.000	0.250
8.00	0.42	0.28	21.03	0.41	647.12	2.7E-4	0.1145	0.1056	0.80	0.0848	0.00E0	0.000	0.250
6.00	0.32	0.21	14.85	0.41	498.96	2.6E-4	0.0971	0.1406	0.80	0.1129	0.00E0	0.000	0.250
4.00	0.21	0.14	14.85	0.41	407.41	2.1E-4	0.1646	0.2385	0.80	0.1914	0.00E0	0.000	0.250
2.00	0.11	0.07	14.85	0.41	288.09	1.5E-4	0.0301	0.0435	0.80	0.0349	0.00E0	0.000	0.250
0.00	0.00	0.00	14.85	0.42	2.80	1.5E-6	0.0010	0.0015	0.80	0.0012	0.00E0	0.000	0.250

Settlement of Unsaturated Sands=0.250 in.
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Unsaturated Sands=0.250 in.
Differential Settlement=0.125 to 0.165 in.

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

1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)

SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT) [atm (tsf)]
fs	Friction from CPT testing [atm (tsf)]
Rf	Ratio of fs/qc (%)
gamma	Total unit weight of soil
gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size
Dr	Relative Density
sigma	Total vertical stress [atm]
sigma'	Effective vertical stress [atm]
sigC'	Effective confining pressure [atm]
rd	Acceleration reduction coefficient by Seed
a_max.	Peak Ground Acceleration (PGA) in ground surface
mZ	Linear acceleration reduction coefficient X depth
a_min.	Minimum acceleration under linear reduction, mZ
CRRv	CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksig	Overburden stress correction factor for CRR7.5
CRRm	After magnitude scaling correction CRRm=CRRv * MSF
MSF	Magnitude scaling factor from M=7.5 to user input M
CSR	Cyclic stress ratio induced by earthquake
CSRfs	CSRfs=CSR*fs1 (Default fs1=1)
fs1	First CSR curve in graphic defined in #9 of Advanced page
fs2	2nd CSR curve in graphic defined in #9 of Advanced page
F.S.	Calculated factor of safety against liquefaction F.S.=CRRm/CSRsf
Cebs	Energy Ratio, Borehole Dia., and Sampling Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60	Fines correction of SPT

(N1)60f	(N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq	Overburden stress correction factor
qc1	CPT after Overburden stress correction
dqc1	Fines correction of CPT
qc1f	CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n	CPT after normalization in Robertson's method
Kc	Fine correction factor in Robertson's Method
qc1f	CPT after Fines correction in Robertson's Method
Ic	Soil type index in Suzuki's and Robertson's Methods
(N1)60s	(N1)60 after settlement fines corrections
CSRm	After magnitude scaling correction for Settlement calculation CSRm=CSRsf / MSF*
CSRfs	Cyclic stress ratio induced by earthquake with user inputed fs
MSF*	Scaling factor from CSR, MSF*=1, based on Item 2 of Page C.
ec	Volumetric strain for saturated sands
dz	Calculation segment, dz=0.050 ft
dsz	Settlement in each segment, dz
dp	User defined print interval
dsp	Settlement in each print interval, dp
Gmax	Shear Modulus at low strain
g_eff	gamma_eff, Effective shear Strain
g*Ge/Gm	gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5	Volumetric Strain for magnitude=7.5
Cec	Magnitude correction factor for any magnitude
ec	Volumetric strain for unsaturated sands, ec=Cec * ec7.5
NoLiq	No-Liquefy Soils

References:

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

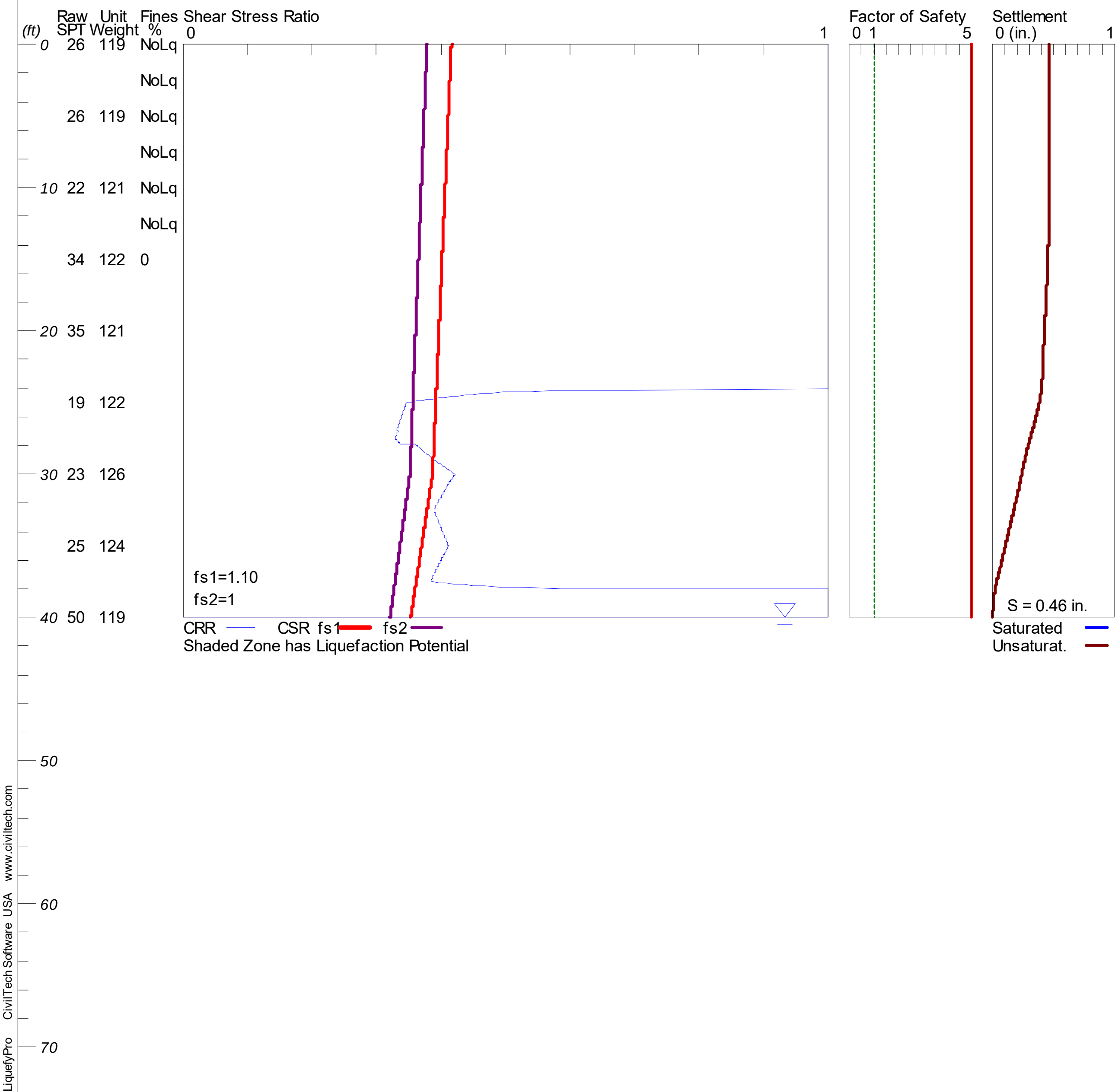
Note: Print Interval you selected does not show complete results. To get complete results, you should select 'Segment' in Print Interval (Item 12, Page C).

LIQUEFACTION ANALYSIS

6136 Manchester Road

Hole No.=B-4 Water Depth=40 ft

Magnitude=6.59
Acceleration=0.582g



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LIQUEFACTION ANALYSIS CALCULATION DETAILS
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Input File Name: Z:\OUR DOCUMENTS\Liquefaction Analysis\6058-1 B-4.liq
Title: 6136 Manchester Road
Subtitle: 6058

Input Data:
Surface Elev.=
Hole No.=B-4
Depth of Hole=40.00 ft
Water Table during Earthquake= 40.00 ft
Water Table during In-Situ Testing= 40.00 ft
Max. Acceleration=0.58 g
Earthquake Magnitude=6.59
No-Liquefiable Soils: CL, OL are Non-Liq. Soil
1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1.1
Plot two CSR (fs1=User, fs2=1)
10. Average two input data between two Depths: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	Gamma pcf	Fines %
0.00	26.00	119.00	NoLiq
2.50	26.00	119.00	NoLiq
5.00	26.00	119.00	NoLiq
7.50	26.00	119.00	NoLiq
10.00	22.00	121.00	NoLiq
12.50	22.00	121.00	NoLiq
15.00	34.00	122.00	0.00
17.50	34.00	122.00	0.00
20.00	35.00	121.00	0.00
22.50	35.00	121.00	0.00
25.00	19.00	122.00	0.00
27.50	19.00	122.00	0.00
30.00	23.00	126.00	0.00
32.50	23.00	126.00	0.00
35.00	25.00	124.00	0.00
37.50	25.00	124.00	0.00
40.00	50.00	119.00	0.00

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

Peak Ground Acceleration (PGA), a_max = 0.58g

CSR Calculation:

Depth ft	gamma pcf	sigma atm	gamma' pcf	sigma' atm	rd	mZ g	a(z) g	CSR	x fs1	=CSRfs
0.00	119.00	0.000	119.00	0.000	1.00	0.000	0.582	0.38	1.10	0.42
2.00	119.00	0.112	119.00	0.112	1.00	0.000	0.582	0.38	1.10	0.41
4.00	119.00	0.225	119.00	0.225	0.99	0.000	0.582	0.37	1.10	0.41

6.00	119.00	0.337	119.00	0.337	0.99	0.000	0.582	0.37	1.10	0.41
8.00	119.40	0.450	119.40	0.450	0.98	0.000	0.582	0.37	1.10	0.41
10.00	121.00	0.563	121.00	0.563	0.98	0.000	0.582	0.37	1.10	0.41
12.00	121.00	0.678	121.00	0.678	0.97	0.000	0.582	0.37	1.10	0.40
14.00	121.60	0.792	121.60	0.792	0.97	0.000	0.582	0.37	1.10	0.40
16.00	122.00	0.908	122.00	0.908	0.96	0.000	0.582	0.36	1.10	0.40
18.00	121.80	1.023	121.80	1.023	0.96	0.000	0.582	0.36	1.10	0.40
20.00	121.00	1.138	121.00	1.138	0.95	0.000	0.582	0.36	1.10	0.40
22.00	121.00	1.252	121.00	1.252	0.95	0.000	0.582	0.36	1.10	0.39
24.00	121.60	1.367	121.60	1.367	0.94	0.000	0.582	0.36	1.10	0.39
26.00	122.00	1.482	122.00	1.482	0.94	0.000	0.582	0.36	1.10	0.39
28.00	122.80	1.597	122.80	1.597	0.93	0.000	0.582	0.35	1.10	0.39
30.00	126.00	1.715	126.00	1.715	0.93	0.000	0.582	0.35	1.10	0.39
32.00	126.00	1.834	126.00	1.834	0.91	0.000	0.582	0.35	1.10	0.38
34.00	124.80	1.952	124.80	1.952	0.90	0.000	0.582	0.34	1.10	0.37
36.00	124.00	2.070	124.00	2.070	0.88	0.000	0.582	0.33	1.10	0.37
38.00	123.00	2.187	123.00	2.187	0.86	0.000	0.582	0.33	1.10	0.36
40.00	119.00	2.301	119.00	2.301	0.85	0.000	0.582	0.32	1.10	0.35

CSR is based on water table at 40.00 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
0.00	26.00	1.50	0.75	0.000	1.70	49.73	NoLiq	7.20	56.93	2.00
2.00	26.00	1.50	0.75	0.112	1.70	49.73	NoLiq	7.20	56.93	2.00
4.00	26.00	1.50	0.75	0.225	1.70	49.73	NoLiq	7.20	56.93	2.00
6.00	26.00	1.50	0.75	0.337	1.70	49.73	NoLiq	7.20	56.93	2.00
8.00	25.20	1.50	0.75	0.450	1.49	42.27	NoLiq	7.20	49.47	2.00
10.00	22.00	1.50	0.85	0.563	1.33	37.37	NoLiq	7.20	44.57	2.00
12.00	22.00	1.50	0.85	0.678	1.21	34.07	NoLiq	7.20	41.27	2.00
14.00	29.20	1.50	0.85	0.792	1.12	41.82	40.40	7.20	49.02	2.00
16.00	34.00	1.50	0.95	0.908	1.05	50.86	0.00	0.00	50.86	2.00
18.00	34.20	1.50	0.95	1.023	0.99	48.19	0.00	0.00	48.19	2.00
20.00	35.00	1.50	0.95	1.138	0.94	46.76	0.00	0.00	46.76	2.00
22.00	35.00	1.50	0.95	1.252	0.89	44.57	0.00	0.00	44.57	2.00
24.00	25.40	1.50	0.95	1.367	0.86	30.96	0.00	0.00	30.96	2.00
26.00	19.00	1.50	0.95	1.482	0.82	22.24	0.00	0.00	22.24	0.24
28.00	19.80	1.50	1.00	1.597	0.79	23.50	0.00	0.00	23.50	0.26
30.00	23.00	1.50	1.00	1.715	0.76	26.35	0.00	0.00	26.35	0.31
32.00	23.00	1.50	1.00	1.834	0.74	25.48	0.00	0.00	25.48	0.29
34.00	24.20	1.50	1.00	1.952	0.72	25.98	0.00	0.00	25.98	0.30
36.00	25.00	1.50	1.00	2.070	0.70	26.07	0.00	0.00	26.07	0.30
38.00	30.00	1.50	1.00	2.187	0.68	30.43	0.00	0.00	30.43	2.00
40.00	50.00	1.50	1.00	2.301	0.66	49.44	0.00	0.00	49.44	2.00

CRR is based on water table at 40.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 6.59:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
0.00	0.00	2.00	1.00	2.00	1.39	2.00	0.42	5.00 ^
2.00	0.07	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
4.00	0.15	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
6.00	0.22	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
8.00	0.29	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
10.00	0.37	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
12.00	0.44	2.00	1.00	2.00	1.39	2.00	0.40	5.00 ^
14.00	0.52	2.00	1.00	2.00	1.39	2.78	0.40	5.00
16.00	0.59	2.00	1.00	2.00	1.39	2.78	0.40	5.00
18.00	0.66	2.00	1.00	2.00	1.39	2.78	0.40	5.00
20.00	0.74	2.00	1.00	2.00	1.39	2.78	0.40	5.00
22.00	0.81	2.00	1.00	2.00	1.39	2.78	0.39	5.00
24.00	0.89	2.00	1.00	2.00	1.39	2.78	0.39	5.00
26.00	0.96	0.24	1.00	0.24	1.39	0.34	0.39	5.00
28.00	1.04	0.26	1.00	0.26	1.39	0.36	0.39	5.00
30.00	1.11	0.31	0.99	0.30	1.39	0.42	0.39	5.00
32.00	1.19	0.29	0.98	0.28	1.39	0.40	0.38	5.00
34.00	1.27	0.30	0.97	0.29	1.39	0.40	0.37	5.00
36.00	1.35	0.30	0.95	0.29	1.39	0.40	0.37	5.00
38.00	1.42	2.00	0.94	1.89	1.39	2.63	0.36	5.00

40.00	1.50	2.00	0.93	1.87	1.39	2.60	0.35	5.00

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

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

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
0.00	-	-	-	56.93	NoLiq	0.00	56.93
2.00	-	-	-	56.93	NoLiq	0.00	56.93
4.00	-	-	-	56.93	NoLiq	0.00	56.93
6.00	-	-	-	56.93	NoLiq	0.00	56.93
8.00	-	-	-	49.47	NoLiq	0.00	49.47
10.00	-	-	-	44.57	NoLiq	0.00	44.57
12.00	-	-	-	41.27	NoLiq	0.00	41.27
14.00	-	-	-	49.02	40.40	0.00	49.02
16.00	-	-	-	50.86	0.00	0.00	50.86
18.00	-	-	-	48.19	0.00	0.00	48.19
20.00	-	-	-	46.76	0.00	0.00	46.76
22.00	-	-	-	44.57	0.00	0.00	44.57
24.00	-	-	-	30.96	0.00	0.00	30.96
26.00	-	-	-	22.24	0.00	0.00	22.24
28.00	-	-	-	23.50	0.00	0.00	23.50
30.00	-	-	-	26.35	0.00	0.00	26.35
32.00	-	-	-	25.48	0.00	0.00	25.48
34.00	-	-	-	25.98	0.00	0.00	25.98
36.00	-	-	-	26.07	0.00	0.00	26.07
38.00	-	-	-	30.43	0.00	0.00	30.43
40.00	-	-	-	49.44	0.00	0.00	49.44

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

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

Depth ft	CSRsf	/ MSF*	=CSRm	F.S.	Fines %	(N1)60s	Dr %	ec %	dsz in.	dsp in.	S in.

No Settlement of Saturated Sands

Settlement of Saturated Sands=0.000 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

Depth ft	sigma' atm	sigC' atm	(N1)60s	CSRsf	Gmax atm	g*Ge/Gm	g_eff	ec7.5 %	Cec	ec %	dsz in.	dsp in.	S in.
39.95	2.30	1.49	48.97	0.35	1997.31	4.1E-4	0.1175	0.0371	0.80	0.0298	3.58E-4	0.000	0.000
38.00	2.19	1.42	30.43	0.36	1662.68	4.7E-4	0.1829	0.1019	0.80	0.0818	9.81E-4	0.021	0.021
36.00	2.07	1.35	26.07	0.37	1536.34	4.9E-4	0.2105	0.1464	0.80	0.1175	1.41E-3	0.056	0.078
34.00	1.95	1.27	25.98	0.37	1490.48	4.9E-4	0.2037	0.1423	0.80	0.1142	1.37E-3	0.055	0.132
32.00	1.83	1.19	25.48	0.38	1435.12	4.9E-4	0.1991	0.1428	0.80	0.1146	1.37E-3	0.056	0.188
30.00	1.71	1.11	26.35	0.39	1403.34	4.7E-4	0.1824	0.1250	0.80	0.1003	1.20E-3	0.051	0.240
28.00	1.60	1.04	23.50	0.39	1303.81	4.8E-4	0.1869	0.1492	0.80	0.1197	1.44E-3	0.053	0.292
26.00	1.48	0.96	22.24	0.39	1233.02	4.7E-4	0.1786	0.1532	0.80	0.1230	1.48E-3	0.064	0.356

24.00	1.37	0.89	30.96	0.39	1322.01	4.1E-4	0.1171	0.0635	0.80	0.0509	6.11E-4	0.046	0.402
22.00	1.25	0.81	44.57	0.39	1428.62	3.5E-4	0.0789	0.0249	0.80	0.0200	2.40E-4	0.013	0.416
20.00	1.14	0.74	46.76	0.40	1383.71	3.3E-4	0.1171	0.0370	0.80	0.0297	3.57E-4	0.010	0.426
18.00	1.02	0.66	48.19	0.40	1325.26	3.1E-4	0.0954	0.0302	0.80	0.0242	2.91E-4	0.013	0.438
16.00	0.91	0.59	50.86	0.40	1270.97	2.9E-4	0.0757	0.0239	0.80	0.0192	2.31E-4	0.010	0.449
14.00	0.79	0.52	49.02	0.40	1173.15	2.7E-4	0.0654	0.0207	0.80	0.0166	1.99E-4	0.008	0.457
12.00	0.68	0.44	41.27	0.40	1024.57	2.7E-4	0.0626	0.0198	0.80	0.0159	0.00E0	0.006	0.463
10.00	0.56	0.37	44.57	0.41	958.38	2.4E-4	0.0472	0.0149	0.80	0.0120	0.00E0	0.000	0.463
8.00	0.45	0.29	49.47	0.41	886.63	2.1E-4	0.0410	0.0130	0.80	0.0104	0.00E0	0.000	0.463
6.00	0.34	0.22	56.93	0.41	804.57	1.7E-4	0.0319	0.0101	0.80	0.0081	0.00E0	0.000	0.463
4.00	0.22	0.15	56.93	0.41	656.93	1.4E-4	0.0267	0.0084	0.80	0.0068	0.00E0	0.000	0.463
2.00	0.11	0.07	56.93	0.41	464.53	1.0E-4	0.0200	0.0063	0.80	0.0051	0.00E0	0.000	0.463
0.00	0.00	0.00	56.93	0.42	4.38	9.5E-7	0.0010	0.0003	0.80	0.0003	0.00E0	0.000	0.463

Settlement of Unsaturated Sands=0.463 in.
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Unsaturated Sands=0.463 in.
Differential Settlement=0.232 to 0.306 in.

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

1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)

SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT) [atm (tsf)]
fs	Friction from CPT testing [atm (tsf)]
Rf	Ratio of fs/qc (%)
gamma	Total unit weight of soil
gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size
Dr	Relative Density
sigma	Total vertical stress [atm]
sigma'	Effective vertical stress [atm]
sigC'	Effective confining pressure [atm]
rd	Acceleration reduction coefficient by Seed
a_max.	Peak Ground Acceleration (PGA) in ground surface
mZ	Linear acceleration reduction coefficient X depth
a_min.	Minimum acceleration under linear reduction, mZ
CRRv	CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksig	Overburden stress correction factor for CRR7.5
CRRm	After magnitude scaling correction CRRm=CRRv * MSF
MSF	Magnitude scaling factor from M=7.5 to user input M
CSR	Cyclic stress ratio induced by earthquake
CSRfs	CSRfs=CSR*fs1 (Default fs1=1)
fs1	First CSR curve in graphic defined in #9 of Advanced page
fs2	2nd CSR curve in graphic defined in #9 of Advanced page
F.S.	Calculated factor of safety against liquefaction F.S.=CRRm/CSRsf
Cebs	Energy Ratio, Borehole Dia., and Sampling Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60	Fines correction of SPT

(N1)60f	(N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq	Overburden stress correction factor
qc1	CPT after Overburden stress correction
dqc1	Fines correction of CPT
qc1f	CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n	CPT after normalization in Robertson's method
Kc	Fine correction factor in Robertson's Method
qc1f	CPT after Fines correction in Robertson's Method
Ic	Soil type index in Suzuki's and Robertson's Methods
(N1)60s	(N1)60 after settlement fines corrections
CSRm	After magnitude scaling correction for Settlement calculation CSRm=CSRsf / MSF*
CSRfs	Cyclic stress ratio induced by earthquake with user inputed fs
MSF*	Scaling factor from CSR, MSF*=1, based on Item 2 of Page C.
ec	Volumetric strain for saturated sands
dz	Calculation segment, dz=0.050 ft
dsz	Settlement in each segment, dz
dp	User defined print interval
dsp	Settlement in each print interval, dp
Gmax	Shear Modulus at low strain
g_eff	gamma_eff, Effective shear Strain
g*Ge/Gm	gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5	Volumetric Strain for magnitude=7.5
Cec	Magnitude correction factor for any magnitude
ec	Volumetric strain for unsaturated sands, ec=Cec * ec7.5
NoLiq	No-Liquefy Soils

References:

1. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.

SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.

2. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING AND SEISMIC SITE RESPONSE EVALUATION, Paper No. SPL-2, PROCEEDINGS: Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA, March 2001.

3. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING: A UNIFIED AND CONSISTENT FRAMEWORK, Earthquake Engineering Research Center, Report No. EERC 2003-06 by R.B Seed and etc. April 2003.

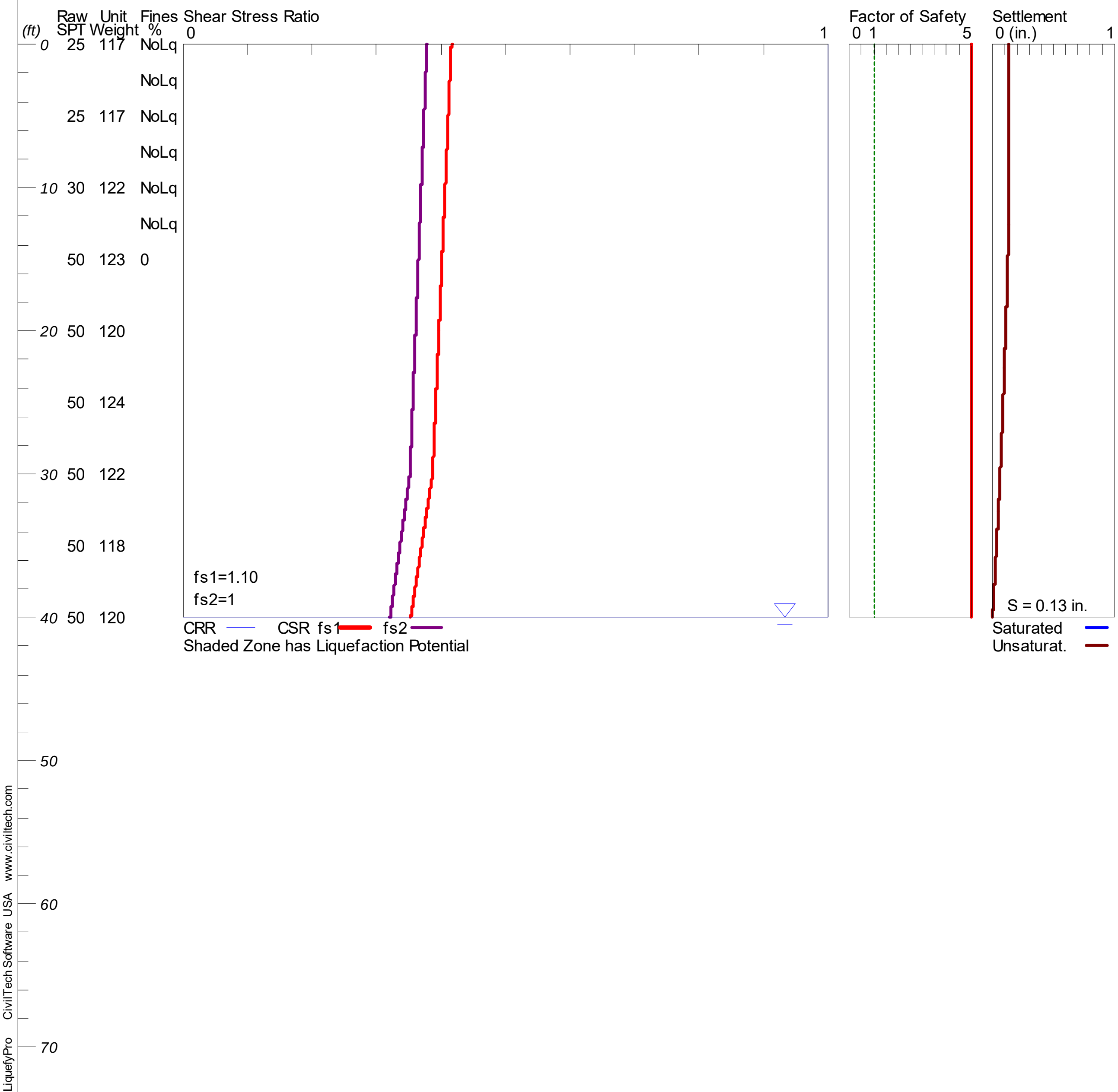
Note: Print Interval you selected does not show complete results. To get complete results, you should select 'Segment' in Print Interval (Item 12, Page C).

LIQUEFACTION ANALYSIS

6136 Manchester Road

Hole No.=B-5   Water Depth=40 ft

Magnitude=6.59
Acceleration=0.582g



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LIQUEFACTION ANALYSIS CALCULATION DETAILS

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Input File Name: Z:\OUR DOCUMENTS\Liquefaction Analysis\6058-1 B-5.liq
Title: 6136 Manchester Road
Subtitle: 6058

Input Data:
Surface Elev.=
Hole No.=B-5
Depth of Hole=40.00 ft
Water Table during Earthquake= 40.00 ft
Water Table during In-Situ Testing= 40.00 ft
Max. Acceleration=0.58 g
Earthquake Magnitude=6.59
No-Liquefiable Soils: CL, OL are Non-Liq. Soil
1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1.1
Plot two CSR (fs1=User, fs2=1)
10. Average two input data between two Depths: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	Gamma pcf	Fines %
0.00	25.00	117.00	NoLiq
2.50	25.00	117.00	NoLiq
5.00	25.00	117.00	NoLiq
7.50	25.00	117.00	NoLiq
10.00	30.00	122.00	NoLiq
12.50	30.00	122.00	NoLiq
15.00	50.00	123.00	0.00
17.50	50.00	123.00	0.00
20.00	50.00	120.00	0.00
22.50	50.00	120.00	0.00
25.00	50.00	124.00	0.00
27.50	50.00	124.00	0.00
30.00	50.00	122.00	0.00
32.50	50.00	122.00	0.00
35.00	50.00	118.00	0.00
37.50	50.00	118.00	0.00
40.00	50.00	120.00	0.00

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

Peak Ground Acceleration (PGA), a_max = 0.58g

CSR Calculation:

Depth ft	gamma pcf	sigma atm	gamma' pcf	sigma' atm	rd	mZ g	a(z) g	CSR	x fs1	=CSRfs
0.00	117.00	0.000	117.00	0.000	1.00	0.000	0.582	0.38	1.10	0.42
2.00	117.00	0.111	117.00	0.111	1.00	0.000	0.582	0.38	1.10	0.41
4.00	117.00	0.221	117.00	0.221	0.99	0.000	0.582	0.37	1.10	0.41

6.00	117.00	0.332	117.00	0.332	0.99	0.000	0.582	0.37	1.10	0.41
8.00	118.00	0.442	118.00	0.442	0.98	0.000	0.582	0.37	1.10	0.41
10.00	122.00	0.556	122.00	0.556	0.98	0.000	0.582	0.37	1.10	0.41
12.00	122.00	0.671	122.00	0.671	0.97	0.000	0.582	0.37	1.10	0.40
14.00	122.60	0.787	122.60	0.787	0.97	0.000	0.582	0.37	1.10	0.40
16.00	123.00	0.903	123.00	0.903	0.96	0.000	0.582	0.36	1.10	0.40
18.00	122.40	1.019	122.40	1.019	0.96	0.000	0.582	0.36	1.10	0.40
20.00	120.00	1.133	120.00	1.133	0.95	0.000	0.582	0.36	1.10	0.40
22.00	120.00	1.247	120.00	1.247	0.95	0.000	0.582	0.36	1.10	0.39
24.00	122.40	1.361	122.40	1.361	0.94	0.000	0.582	0.36	1.10	0.39
26.00	124.00	1.478	124.00	1.478	0.94	0.000	0.582	0.36	1.10	0.39
28.00	123.60	1.595	123.60	1.595	0.93	0.000	0.582	0.35	1.10	0.39
30.00	122.00	1.711	122.00	1.711	0.93	0.000	0.582	0.35	1.10	0.39
32.00	122.00	1.826	122.00	1.826	0.91	0.000	0.582	0.35	1.10	0.38
34.00	119.60	1.941	119.60	1.941	0.90	0.000	0.582	0.34	1.10	0.37
36.00	118.00	2.053	118.00	2.053	0.88	0.000	0.582	0.33	1.10	0.37
38.00	118.40	2.164	118.40	2.164	0.86	0.000	0.582	0.33	1.10	0.36
40.00	120.00	2.277	120.00	2.277	0.85	0.000	0.582	0.32	1.10	0.35

CSR is based on water table at 40.00 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
0.00	25.00	1.50	0.75	0.000	1.70	47.81	NoLiq	7.20	55.01	2.00
2.00	25.00	1.50	0.75	0.111	1.70	47.81	NoLiq	7.20	55.01	2.00
4.00	25.00	1.50	0.75	0.221	1.70	47.81	NoLiq	7.20	55.01	2.00
6.00	25.00	1.50	0.75	0.332	1.70	47.81	NoLiq	7.20	55.01	2.00
8.00	26.00	1.50	0.75	0.442	1.50	43.98	NoLiq	7.20	51.18	2.00
10.00	30.00	1.50	0.85	0.556	1.34	51.31	NoLiq	7.20	58.51	2.00
12.00	30.00	1.50	0.85	0.671	1.22	46.69	NoLiq	7.20	53.89	2.00
14.00	42.00	1.50	0.85	0.787	1.13	60.38	40.40	7.20	67.58	2.00
16.00	50.00	1.50	0.95	0.903	1.05	74.99	0.00	0.00	74.99	2.00
18.00	50.00	1.50	0.95	1.019	0.99	70.59	0.00	0.00	70.59	2.00
20.00	50.00	1.50	0.95	1.133	0.94	66.92	0.00	0.00	66.92	2.00
22.00	50.00	1.50	0.95	1.247	0.90	63.81	0.00	0.00	63.81	2.00
24.00	50.00	1.50	0.95	1.361	0.86	61.07	0.00	0.00	61.07	2.00
26.00	50.00	1.50	0.95	1.478	0.82	58.61	0.00	0.00	58.61	2.00
28.00	50.00	1.50	1.00	1.595	0.79	59.38	0.00	0.00	59.38	2.00
30.00	50.00	1.50	1.00	1.711	0.76	57.33	0.00	0.00	57.33	2.00
32.00	50.00	1.50	1.00	1.826	0.74	55.50	0.00	0.00	55.50	2.00
34.00	50.00	1.50	1.00	1.941	0.72	53.83	0.00	0.00	53.83	2.00
36.00	50.00	1.50	1.00	2.053	0.70	52.35	0.00	0.00	52.35	2.00
38.00	50.00	1.50	1.00	2.164	0.68	50.98	0.00	0.00	50.98	2.00
40.00	50.00	1.50	1.00	2.277	0.66	49.70	0.00	0.00	49.70	2.00

CRR is based on water table at 40.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 6.59:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
0.00	0.00	2.00	1.00	2.00	1.39	2.00	0.42	5.00 ^
2.00	0.07	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
4.00	0.14	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
6.00	0.22	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
8.00	0.29	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
10.00	0.36	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
12.00	0.44	2.00	1.00	2.00	1.39	2.00	0.40	5.00 ^
14.00	0.51	2.00	1.00	2.00	1.39	2.78	0.40	5.00
16.00	0.59	2.00	1.00	2.00	1.39	2.78	0.40	5.00
18.00	0.66	2.00	1.00	2.00	1.39	2.78	0.40	5.00
20.00	0.74	2.00	1.00	2.00	1.39	2.78	0.40	5.00
22.00	0.81	2.00	1.00	2.00	1.39	2.78	0.39	5.00
24.00	0.88	2.00	1.00	2.00	1.39	2.78	0.39	5.00
26.00	0.96	2.00	1.00	2.00	1.39	2.78	0.39	5.00
28.00	1.04	2.00	1.00	2.00	1.39	2.79	0.39	5.00
30.00	1.11	2.00	0.99	1.98	1.39	2.75	0.39	5.00
32.00	1.19	2.00	0.98	1.95	1.39	2.72	0.38	5.00
34.00	1.26	2.00	0.97	1.93	1.39	2.69	0.37	5.00
36.00	1.33	2.00	0.96	1.91	1.39	2.66	0.37	5.00
38.00	1.41	2.00	0.95	1.89	1.39	2.63	0.36	5.00

40.00	1.48	2.00	0.94	1.87	1.39	2.60	0.35	5.00

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

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

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
0.00	-	-	-	55.01	NoLiq	0.00	55.01
2.00	-	-	-	55.01	NoLiq	0.00	55.01
4.00	-	-	-	55.01	NoLiq	0.00	55.01
6.00	-	-	-	55.01	NoLiq	0.00	55.01
8.00	-	-	-	51.18	NoLiq	0.00	51.18
10.00	-	-	-	58.51	NoLiq	0.00	58.51
12.00	-	-	-	53.89	NoLiq	0.00	53.89
14.00	-	-	-	67.58	40.40	0.00	67.58
16.00	-	-	-	74.99	0.00	0.00	74.99
18.00	-	-	-	70.59	0.00	0.00	70.59
20.00	-	-	-	66.92	0.00	0.00	66.92
22.00	-	-	-	63.81	0.00	0.00	63.81
24.00	-	-	-	61.07	0.00	0.00	61.07
26.00	-	-	-	58.61	0.00	0.00	58.61
28.00	-	-	-	59.38	0.00	0.00	59.38
30.00	-	-	-	57.33	0.00	0.00	57.33
32.00	-	-	-	55.50	0.00	0.00	55.50
34.00	-	-	-	53.83	0.00	0.00	53.83
36.00	-	-	-	52.35	0.00	0.00	52.35
38.00	-	-	-	50.98	0.00	0.00	50.98
40.00	-	-	-	49.70	0.00	0.00	49.70

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

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

Depth ft	CSRsf	/ MSF*	=CSRm	F.S.	Fines %	(N1)60s	Dr %	ec %	dsz in.	dsp in.	S in.
No Settlement of Saturated Sands											

Settlement of Saturated Sands=0.000 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

Depth ft	sigma' atm	sigC' atm	(N1)60s	CSRsf	Gmax atm	g*Ge/Gm	g_eff	ec7.5 %	Cec	ec %	dsz in.	dsp in.	S in.
39.95	2.27	1.48	49.73	0.35	1996.97	4.0E-4	0.1142	0.0361	0.80	0.0290	3.48E-4	0.000	0.000
38.00	2.16	1.41	50.98	0.36	1964.28	4.0E-4	0.1100	0.0348	0.80	0.0279	3.35E-4	0.013	0.014
36.00	2.05	1.33	52.35	0.37	1929.92	3.9E-4	0.1054	0.0333	0.80	0.0267	3.21E-4	0.013	0.027
34.00	1.94	1.26	53.83	0.37	1894.17	3.8E-4	0.1004	0.0318	0.80	0.0255	3.06E-4	0.013	0.039
32.00	1.83	1.19	55.50	0.38	1856.15	3.7E-4	0.0949	0.0300	0.80	0.0241	2.89E-4	0.012	0.051
30.00	1.71	1.11	57.33	0.39	1816.22	3.6E-4	0.0892	0.0282	0.80	0.0226	2.72E-4	0.011	0.062
28.00	1.60	1.04	59.38	0.39	1774.17	3.5E-4	0.0809	0.0256	0.80	0.0205	2.46E-4	0.010	0.073
26.00	1.48	0.96	58.61	0.39	1700.31	3.4E-4	0.0757	0.0239	0.80	0.0192	2.31E-4	0.010	0.082

24.00	1.36	0.88	61.07	0.39	1654.27	3.2E-4	0.0678	0.0215	0.80	0.0172	2.07E-4	0.009	0.091
22.00	1.25	0.81	63.81	0.39	1606.61	3.1E-4	0.0606	0.0192	0.80	0.0154	1.85E-4	0.008	0.099
20.00	1.13	0.74	66.92	0.40	1556.32	2.9E-4	0.0780	0.0247	0.80	0.0198	2.38E-4	0.008	0.107
18.00	1.02	0.66	70.59	0.40	1501.98	2.7E-4	0.0644	0.0204	0.80	0.0163	1.96E-4	0.009	0.115
16.00	0.90	0.59	74.99	0.40	1442.54	2.5E-4	0.0529	0.0167	0.80	0.0134	1.61E-4	0.007	0.122
14.00	0.79	0.51	67.58	0.40	1300.69	2.4E-4	0.0493	0.0156	0.80	0.0125	1.50E-4	0.006	0.128
12.00	0.67	0.44	53.89	0.40	1114.19	2.4E-4	0.0494	0.0156	0.80	0.0125	0.00E0	0.004	0.133
10.00	0.56	0.36	58.51	0.41	1042.09	2.2E-4	0.0385	0.0122	0.80	0.0098	0.00E0	0.000	0.133
8.00	0.44	0.29	51.18	0.41	889.21	2.0E-4	0.0397	0.0126	0.80	0.0101	0.00E0	0.000	0.133
6.00	0.33	0.22	55.01	0.41	788.76	1.7E-4	0.0320	0.0101	0.80	0.0081	0.00E0	0.000	0.133
4.00	0.22	0.14	55.01	0.41	644.02	1.4E-4	0.0268	0.0085	0.80	0.0068	0.00E0	0.000	0.133
2.00	0.11	0.07	55.01	0.41	455.40	1.0E-4	0.0201	0.0064	0.80	0.0051	0.00E0	0.000	0.133
0.00	0.00	0.00	55.01	0.42	4.33	9.6E-7	0.0010	0.0003	0.80	0.0003	0.00E0	0.000	0.133

Settlement of Unsaturated Sands=0.133 in.
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Unsaturated Sands=0.133 in.
Differential Settlement=0.066 to 0.088 in.

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

1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)

SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT) [atm (tsf)]
fs	Friction from CPT testing [atm (tsf)]
Rf	Ratio of fs/qc (%)
gamma	Total unit weight of soil
gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size
Dr	Relative Density
sigma	Total vertical stress [atm]
sigma'	Effective vertical stress [atm]
sigC'	Effective confining pressure [atm]
rd	Acceleration reduction coefficient by Seed
a_max.	Peak Ground Acceleration (PGA) in ground surface
mZ	Linear acceleration reduction coefficient X depth
a_min.	Minimum acceleration under linear reduction, mZ
CRRv	CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksig	Overburden stress correction factor for CRR7.5
CRRm	After magnitude scaling correction CRRm=CRRv * MSF
MSF	Magnitude scaling factor from M=7.5 to user input M
CSR	Cyclic stress ratio induced by earthquake
CSRfs	CSRfs=CSR*fs1 (Default fs1=1)
fs1	First CSR curve in graphic defined in #9 of Advanced page
fs2	2nd CSR curve in graphic defined in #9 of Advanced page
F.S.	Calculated factor of safety against liquefaction F.S.=CRRm/CSRsf
Cebs	Energy Ratio, Borehole Dia., and Sampling Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60	Fines correction of SPT

(N1)60f	(N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq	Overburden stress correction factor
qc1	CPT after Overburden stress correction
dqc1	Fines correction of CPT
qc1f	CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n	CPT after normalization in Robertson's method
Kc	Fine correction factor in Robertson's Method
qc1f	CPT after Fines correction in Robertson's Method
Ic	Soil type index in Suzuki's and Robertson's Methods
(N1)60s	(N1)60 after settlement fines corrections
CSRm	After magnitude scaling correction for Settlement calculation CSRm=CSRsf / MSF*
CSRfs	Cyclic stress ratio induced by earthquake with user inputed fs
MSF*	Scaling factor from CSR, MSF*=1, based on Item 2 of Page C.
ec	Volumetric strain for saturated sands
dz	Calculation segment, dz=0.050 ft
dsz	Settlement in each segment, dz
dp	User defined print interval
dsp	Settlement in each print interval, dp
Gmax	Shear Modulus at low strain
g_eff	gamma_eff, Effective shear Strain
g*Ge/Gm	gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5	Volumetric Strain for magnitude=7.5
Cec	Magnitude correction factor for any magnitude
ec	Volumetric strain for unsaturated sands, ec=Cec * ec7.5
NoLiq	No-Liquefy Soils

References:

1. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.
SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.

2. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING AND SEISMIC SITE RESPONSE EVALUATION, Paper No. SPL-2, PROCEEDINGS: Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA, March 2001.

3. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING: A UNIFIED AND CONSISTENT FRAMEWORK, Earthquake Engineering Research Center, Report No. EERC 2003-06 by R.B Seed and etc. April 2003.

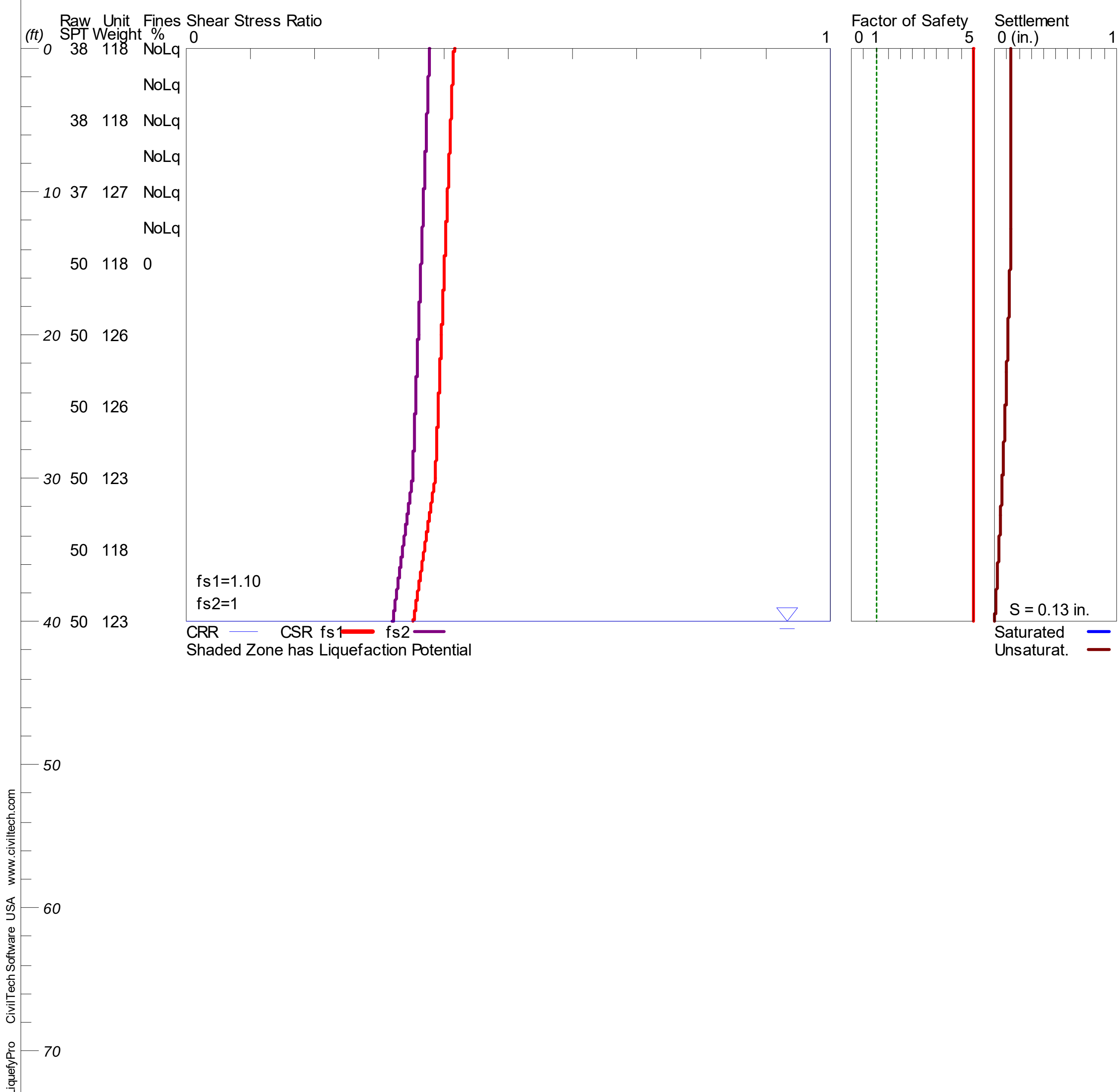
Note: Print Interval you selected does not show complete results. To get complete results, you should select 'Segment' in Print Interval (Item 12, Page C).

LIQUEFACTION ANALYSIS

6136 Manchester Road

Hole No.=B-6 Water Depth=40 ft

Magnitude=6.59
Acceleration=0.582g



****
LIQUEFACTION ANALYSIS CALCULATION DETAILS
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Font: Courier New, Regular, Size 8 is recommended for this report.
Licensed to , 2/3/2022 2:28:33 PM

Input File Name: Z:\OUR DOCUMENTS\Liquefaction Analysis\6058-1 B-6.liq
Title: 6136 Manchester Road
Subtitle: 6058

Input Data:
Surface Elev.=
Hole No.=B-6
Depth of Hole=40.00 ft
Water Table during Earthquake= 40.00 ft
Water Table during In-Situ Testing= 40.00 ft
Max. Acceleration=0.58 g
Earthquake Magnitude=6.59
No-Liquefiable Soils: CL, OL are Non-Liq. Soil
1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1.1
Plot two CSR (fs1=User, fs2=1)
10. Average two input data between two Depths: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	Gamma pcf	Fines %
0.00	38.00	118.00	NoLiq
2.50	38.00	118.00	NoLiq
5.00	38.00	118.00	NoLiq
7.50	38.00	118.00	NoLiq
10.00	37.00	127.00	NoLiq
12.50	37.00	127.00	NoLiq
15.00	50.00	118.00	0.00
17.50	50.00	118.00	0.00
20.00	50.00	126.00	0.00
22.50	50.00	126.00	0.00
25.00	50.00	126.00	0.00
27.50	50.00	126.00	0.00
30.00	50.00	123.00	0.00
32.50	50.00	123.00	0.00
35.00	50.00	118.00	0.00
37.50	50.00	118.00	0.00
40.00	50.00	123.00	0.00

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

Peak Ground Acceleration (PGA), a_max = 0.58g

CSR Calculation:

Depth ft	gamma pcf	sigma atm	gamma' pcf	sigma' atm	rd	mZ g	a(z) g	CSR	x fs1	=CSRfs
0.00	118.00	0.000	118.00	0.000	1.00	0.000	0.582	0.38	1.10	0.42
2.00	118.00	0.112	118.00	0.112	1.00	0.000	0.582	0.38	1.10	0.41
4.00	118.00	0.223	118.00	0.223	0.99	0.000	0.582	0.37	1.10	0.41

6.00	118.00	0.335	118.00	0.335	0.99	0.000	0.582	0.37	1.10	0.41
8.00	119.80	0.446	119.80	0.446	0.98	0.000	0.582	0.37	1.10	0.41
10.00	127.00	0.563	127.00	0.563	0.98	0.000	0.582	0.37	1.10	0.41
12.00	127.00	0.683	127.00	0.683	0.97	0.000	0.582	0.37	1.10	0.40
14.00	121.60	0.801	121.60	0.801	0.97	0.000	0.582	0.37	1.10	0.40
16.00	118.00	0.913	118.00	0.913	0.96	0.000	0.582	0.36	1.10	0.40
18.00	119.60	1.025	119.60	1.025	0.96	0.000	0.582	0.36	1.10	0.40
20.00	126.00	1.141	126.00	1.141	0.95	0.000	0.582	0.36	1.10	0.40
22.00	126.00	1.260	126.00	1.260	0.95	0.000	0.582	0.36	1.10	0.39
24.00	126.00	1.379	126.00	1.379	0.94	0.000	0.582	0.36	1.10	0.39
26.00	126.00	1.498	126.00	1.498	0.94	0.000	0.582	0.36	1.10	0.39
28.00	125.40	1.617	125.40	1.617	0.93	0.000	0.582	0.35	1.10	0.39
30.00	123.00	1.735	123.00	1.735	0.93	0.000	0.582	0.35	1.10	0.39
32.00	123.00	1.851	123.00	1.851	0.91	0.000	0.582	0.35	1.10	0.38
34.00	120.00	1.966	120.00	1.966	0.90	0.000	0.582	0.34	1.10	0.37
36.00	118.00	2.078	118.00	2.078	0.88	0.000	0.582	0.33	1.10	0.37
38.00	119.00	2.190	119.00	2.190	0.86	0.000	0.582	0.33	1.10	0.36
40.00	123.00	2.304	123.00	2.304	0.85	0.000	0.582	0.32	1.10	0.35

CSR is based on water table at 40.00 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
0.00	38.00	1.50	0.75	0.000	1.70	72.68	NoLiq	7.20	79.88	2.00
2.00	38.00	1.50	0.75	0.112	1.70	72.68	NoLiq	7.20	79.88	2.00
4.00	38.00	1.50	0.75	0.223	1.70	72.68	NoLiq	7.20	79.88	2.00
6.00	38.00	1.50	0.75	0.335	1.70	72.68	NoLiq	7.20	79.88	2.00
8.00	37.80	1.50	0.75	0.446	1.50	63.66	NoLiq	7.20	70.86	2.00
10.00	37.00	1.50	0.85	0.563	1.33	62.88	NoLiq	7.20	70.08	2.00
12.00	37.00	1.50	0.85	0.683	1.21	57.09	NoLiq	7.20	64.29	2.00
14.00	44.80	1.50	0.85	0.801	1.12	63.82	40.40	7.20	71.02	2.00
16.00	50.00	1.50	0.95	0.913	1.05	74.55	0.00	0.00	74.55	2.00
18.00	50.00	1.50	0.95	1.025	0.99	70.37	0.00	0.00	70.37	2.00
20.00	50.00	1.50	0.95	1.141	0.94	66.70	0.00	0.00	66.70	2.00
22.00	50.00	1.50	0.95	1.260	0.89	63.47	0.00	0.00	63.47	2.00
24.00	50.00	1.50	0.95	1.379	0.85	60.67	0.00	0.00	60.67	2.00
26.00	50.00	1.50	0.95	1.498	0.82	58.21	0.00	0.00	58.21	2.00
28.00	50.00	1.50	1.00	1.617	0.79	58.97	0.00	0.00	58.97	2.00
30.00	50.00	1.50	1.00	1.735	0.76	56.94	0.00	0.00	56.94	2.00
32.00	50.00	1.50	1.00	1.851	0.74	55.13	0.00	0.00	55.13	2.00
34.00	50.00	1.50	1.00	1.966	0.71	53.49	0.00	0.00	53.49	2.00
36.00	50.00	1.50	1.00	2.078	0.69	52.02	0.00	0.00	52.02	2.00
38.00	50.00	1.50	1.00	2.190	0.68	50.68	0.00	0.00	50.68	2.00
40.00	50.00	1.50	1.00	2.304	0.66	49.41	0.00	0.00	49.41	2.00

CRR is based on water table at 40.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 6.59:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
0.00	0.00	2.00	1.00	2.00	1.39	2.00	0.42	5.00 ^
2.00	0.07	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
4.00	0.14	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
6.00	0.22	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
8.00	0.29	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
10.00	0.37	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
12.00	0.44	2.00	1.00	2.00	1.39	2.00	0.40	5.00 ^
14.00	0.52	2.00	1.00	2.00	1.39	2.78	0.40	5.00
16.00	0.59	2.00	1.00	2.00	1.39	2.78	0.40	5.00
18.00	0.67	2.00	1.00	2.00	1.39	2.78	0.40	5.00
20.00	0.74	2.00	1.00	2.00	1.39	2.78	0.40	5.00
22.00	0.82	2.00	1.00	2.00	1.39	2.78	0.39	5.00
24.00	0.90	2.00	1.00	2.00	1.39	2.78	0.39	5.00
26.00	0.97	2.00	1.00	2.00	1.39	2.78	0.39	5.00
28.00	1.05	2.00	1.00	2.00	1.39	2.78	0.39	5.00
30.00	1.13	2.00	0.99	1.97	1.39	2.75	0.39	5.00
32.00	1.20	2.00	0.97	1.95	1.39	2.71	0.38	5.00
34.00	1.28	2.00	0.96	1.93	1.39	2.68	0.37	5.00
36.00	1.35	2.00	0.95	1.91	1.39	2.65	0.37	5.00
38.00	1.42	2.00	0.94	1.89	1.39	2.63	0.36	5.00

40.00	1.50	2.00	0.93	1.87	1.39	2.60	0.35	5.00

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

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

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
0.00	-	-	-	79.88	NoLiq	0.00	79.88
2.00	-	-	-	79.88	NoLiq	0.00	79.88
4.00	-	-	-	79.88	NoLiq	0.00	79.88
6.00	-	-	-	79.88	NoLiq	0.00	79.88
8.00	-	-	-	70.86	NoLiq	0.00	70.86
10.00	-	-	-	70.08	NoLiq	0.00	70.08
12.00	-	-	-	64.29	NoLiq	0.00	64.29
14.00	-	-	-	71.02	40.40	0.00	71.02
16.00	-	-	-	74.55	0.00	0.00	74.55
18.00	-	-	-	70.37	0.00	0.00	70.37
20.00	-	-	-	66.70	0.00	0.00	66.70
22.00	-	-	-	63.47	0.00	0.00	63.47
24.00	-	-	-	60.67	0.00	0.00	60.67
26.00	-	-	-	58.21	0.00	0.00	58.21
28.00	-	-	-	58.97	0.00	0.00	58.97
30.00	-	-	-	56.94	0.00	0.00	56.94
32.00	-	-	-	55.13	0.00	0.00	55.13
34.00	-	-	-	53.49	0.00	0.00	53.49
36.00	-	-	-	52.02	0.00	0.00	52.02
38.00	-	-	-	50.68	0.00	0.00	50.68
40.00	-	-	-	49.41	0.00	0.00	49.41

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

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

Depth ft	CSRsf	/ MSF*	=CSRm	F.S.	Fines %	(N1)60s	Dr %	ec %	dsz in.	dsp in.	S in.

No Settlement of Saturated Sands

Settlement of Saturated Sands=0.000 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

Depth ft	sigma' atm	sigC' atm	(N1)60s	CSRsf	Gmax atm	g*Ge/Gm	g_eff	ec7.5 %	Cec	ec %	dsz in.	dsp in.	S in.
39.95	2.30	1.50	49.44	0.35	2004.86	4.1E-4	0.1167	0.0369	0.80	0.0296	3.55E-4	0.000	0.000
38.00	2.19	1.42	50.68	0.36	1971.96	4.0E-4	0.1123	0.0355	0.80	0.0285	3.42E-4	0.014	0.014
36.00	2.08	1.35	52.02	0.37	1937.85	3.9E-4	0.1076	0.0340	0.80	0.0273	3.28E-4	0.013	0.027
34.00	1.97	1.28	53.49	0.37	1902.37	3.9E-4	0.1026	0.0324	0.80	0.0260	3.13E-4	0.013	0.040
32.00	1.85	1.20	55.13	0.38	1864.44	3.8E-4	0.0970	0.0307	0.80	0.0246	2.96E-4	0.012	0.052
30.00	1.73	1.13	56.94	0.39	1824.55	3.7E-4	0.0912	0.0288	0.80	0.0231	2.78E-4	0.011	0.064
28.00	1.62	1.05	58.97	0.39	1782.40	3.5E-4	0.0826	0.0261	0.80	0.0210	2.52E-4	0.011	0.074
26.00	1.50	0.97	58.21	0.39	1708.11	3.4E-4	0.0773	0.0244	0.80	0.0196	2.35E-4	0.010	0.084

24.00	1.38	0.90	60.67	0.39	1661.58	3.3E-4	0.0692	0.0219	0.80	0.0176	2.11E-4	0.009	0.093
22.00	1.26	0.82	63.47	0.39	1612.30	3.1E-4	0.0615	0.0194	0.80	0.0156	1.87E-4	0.008	0.101
20.00	1.14	0.74	66.70	0.40	1559.80	2.9E-4	0.0791	0.0250	0.80	0.0201	2.41E-4	0.007	0.109
18.00	1.03	0.67	70.37	0.40	1505.03	2.7E-4	0.0651	0.0206	0.80	0.0165	1.98E-4	0.009	0.117
16.00	0.91	0.59	74.55	0.40	1448.22	2.5E-4	0.0539	0.0171	0.80	0.0137	1.64E-4	0.007	0.124
14.00	0.80	0.52	71.02	0.40	1334.47	2.4E-4	0.0485	0.0153	0.80	0.0123	1.48E-4	0.006	0.131
12.00	0.68	0.44	64.29	0.40	1191.91	2.3E-4	0.0441	0.0140	0.80	0.0112	0.00E0	0.004	0.135
10.00	0.56	0.37	70.08	0.41	1113.64	2.1E-4	0.0348	0.0110	0.80	0.0088	0.00E0	0.000	0.135
8.00	0.45	0.29	70.86	0.41	995.30	1.8E-4	0.0349	0.0110	0.80	0.0088	0.00E0	0.000	0.135
6.00	0.33	0.22	79.88	0.41	896.85	1.5E-4	0.0272	0.0086	0.80	0.0069	0.00E0	0.000	0.135
4.00	0.22	0.14	79.88	0.41	732.28	1.3E-4	0.0232	0.0073	0.80	0.0059	0.00E0	0.000	0.135
2.00	0.11	0.07	79.88	0.41	517.81	8.9E-5	0.0166	0.0053	0.80	0.0042	0.00E0	0.000	0.135
0.00	0.00	0.00	79.88	0.42	4.90	8.5E-7	0.0010	0.0003	0.80	0.0003	0.00E0	0.000	0.135

Settlement of Unsaturated Sands=0.135 in.
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Unsaturated Sands=0.135 in.
Differential Settlement=0.067 to 0.089 in.

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

1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)

SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT) [atm (tsf)]
fs	Friction from CPT testing [atm (tsf)]
Rf	Ratio of fs/qc (%)
gamma	Total unit weight of soil
gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size
Dr	Relative Density
sigma	Total vertical stress [atm]
sigma'	Effective vertical stress [atm]
sigC'	Effective confining pressure [atm]
rd	Acceleration reduction coefficient by Seed
a_max.	Peak Ground Acceleration (PGA) in ground surface
mZ	Linear acceleration reduction coefficient X depth
a_min.	Minimum acceleration under linear reduction, mZ
CRRv	CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksig	Overburden stress correction factor for CRR7.5
CRRm	After magnitude scaling correction CRRm=CRRv * MSF
MSF	Magnitude scaling factor from M=7.5 to user input M
CSR	Cyclic stress ratio induced by earthquake
CSRfs	CSRfs=CSR*fs1 (Default fs1=1)
fs1	First CSR curve in graphic defined in #9 of Advanced page
fs2	2nd CSR curve in graphic defined in #9 of Advanced page
F.S.	Calculated factor of safety against liquefaction F.S.=CRRm/CSRsf
Cebs	Energy Ratio, Borehole Dia., and Sampling Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60	Fines correction of SPT

(N1)60f	(N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq	Overburden stress correction factor
qc1	CPT after Overburden stress correction
dqc1	Fines correction of CPT
qc1f	CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n	CPT after normalization in Robertson's method
Kc	Fine correction factor in Robertson's Method
qc1f	CPT after Fines correction in Robertson's Method
Ic	Soil type index in Suzuki's and Robertson's Methods
(N1)60s	(N1)60 after settlement fines corrections
CSRm	After magnitude scaling correction for Settlement calculation CSRm=CSRsf / MSF*
CSRfs	Cyclic stress ratio induced by earthquake with user inputed fs
MSF*	Scaling factor from CSR, MSF*=1, based on Item 2 of Page C.
ec	Volumetric strain for saturated sands
dz	Calculation segment, dz=0.050 ft
dsz	Settlement in each segment, dz
dp	User defined print interval
dsp	Settlement in each print interval, dp
Gmax	Shear Modulus at low strain
g_eff	gamma_eff, Effective shear Strain
g*Ge/Gm	gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5	Volumetric Strain for magnitude=7.5
Cec	Magnitude correction factor for any magnitude
ec	Volumetric strain for unsaturated sands, ec=Cec * ec7.5
NoLiq	No-Liquefy Soils

References:

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

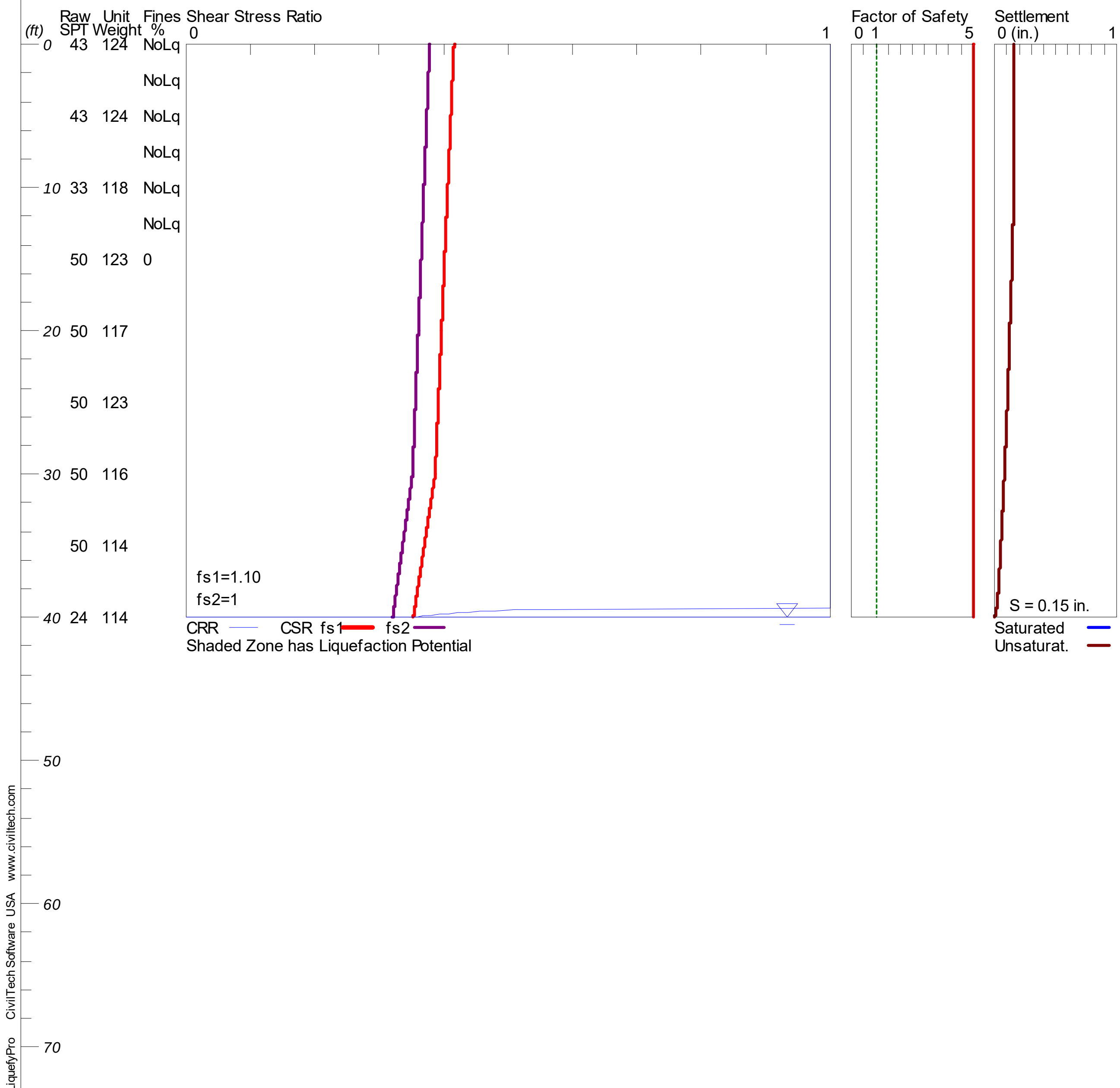
Note: Print Interval you selected does not show complete results. To get complete results, you should select 'Segment' in Print Interval (Item 12, Page C).

LIQUEFACTION ANALYSIS

6136 Manchester Road

Hole No.=B-7 Water Depth=40 ft

Magnitude=6.59
Acceleration=0.582g



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LIQUEFACTION ANALYSIS CALCULATION DETAILS
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Font: Courier New, Regular, Size 8 is recommended for this report.
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Input File Name: Z:\OUR DOCUMENTS\Liquefaction Analysis\6058-1 B-7.liq
Title: 6136 Manchester Road
Subtitle: 6058

Input Data:
Surface Elev.=
Hole No.=B-7
Depth of Hole=40.00 ft
Water Table during Earthquake= 40.00 ft
Water Table during In-Situ Testing= 40.00 ft
Max. Acceleration=0.58 g
Earthquake Magnitude=6.59
No-Liquefiable Soils: CL, OL are Non-Liq. Soil
1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Stark/Olson et al.*
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.25
7. Borehole Diameter, Cb= 1
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1.1
Plot two CSR (fs1=User, fs2=1)
10. Average two input data between two Depths: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	Gamma pcf	Fines %
0.00	43.00	124.00	NoLiq
2.50	43.00	124.00	NoLiq
5.00	43.00	124.00	NoLiq
7.50	43.00	124.00	NoLiq
10.00	33.00	118.00	NoLiq
12.50	33.00	118.00	NoLiq
15.00	50.00	123.00	0.00
17.50	50.00	123.00	0.00
20.00	50.00	117.00	0.00
22.50	50.00	117.00	0.00
25.00	50.00	123.00	0.00
27.50	50.00	123.00	0.00
30.00	50.00	116.00	0.00
32.50	50.00	116.00	0.00
35.00	50.00	114.00	0.00
37.50	50.00	114.00	0.00
40.00	24.00	114.00	0.00

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

Peak Ground Acceleration (PGA), a_max = 0.58g

CSR Calculation:

Depth ft	gamma pcf	sigma atm	gamma' pcf	sigma' atm	rd	mZ g	a(z) g	CSR	x fs1	=CSRfs
0.00	124.00	0.000	124.00	0.000	1.00	0.000	0.582	0.38	1.10	0.42
2.00	124.00	0.117	124.00	0.117	1.00	0.000	0.582	0.38	1.10	0.41
4.00	124.00	0.234	124.00	0.234	0.99	0.000	0.582	0.37	1.10	0.41

6.00	124.00	0.352	124.00	0.352	0.99	0.000	0.582	0.37	1.10	0.41
8.00	122.80	0.469	122.80	0.469	0.98	0.000	0.582	0.37	1.10	0.41
10.00	118.00	0.582	118.00	0.582	0.98	0.000	0.582	0.37	1.10	0.41
12.00	118.00	0.694	118.00	0.694	0.97	0.000	0.582	0.37	1.10	0.40
14.00	121.00	0.807	121.00	0.807	0.97	0.000	0.582	0.37	1.10	0.40
16.00	123.00	0.922	123.00	0.922	0.96	0.000	0.582	0.36	1.10	0.40
18.00	121.80	1.038	121.80	1.038	0.96	0.000	0.582	0.36	1.10	0.40
20.00	117.00	1.151	117.00	1.151	0.95	0.000	0.582	0.36	1.10	0.40
22.00	117.00	1.262	117.00	1.262	0.95	0.000	0.582	0.36	1.10	0.39
24.00	120.60	1.374	120.60	1.374	0.94	0.000	0.582	0.36	1.10	0.39
26.00	123.00	1.489	123.00	1.489	0.94	0.000	0.582	0.36	1.10	0.39
28.00	121.60	1.605	121.60	1.605	0.93	0.000	0.582	0.35	1.10	0.39
30.00	116.00	1.718	116.00	1.718	0.93	0.000	0.582	0.35	1.10	0.39
32.00	116.00	1.827	116.00	1.827	0.91	0.000	0.582	0.35	1.10	0.38
34.00	114.80	1.937	114.80	1.937	0.90	0.000	0.582	0.34	1.10	0.37
36.00	114.00	2.045	114.00	2.045	0.88	0.000	0.582	0.33	1.10	0.37
38.00	114.00	2.152	114.00	2.152	0.86	0.000	0.582	0.33	1.10	0.36
40.00	114.00	2.260	114.00	2.260	0.85	0.000	0.582	0.32	1.10	0.35

CSR is based on water table at 40.00 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
0.00	43.00	1.50	0.75	0.000	1.70	82.24	NoLiq	7.20	89.44	2.00
2.00	43.00	1.50	0.75	0.117	1.70	82.24	NoLiq	7.20	89.44	2.00
4.00	43.00	1.50	0.75	0.234	1.70	82.24	NoLiq	7.20	89.44	2.00
6.00	43.00	1.50	0.75	0.352	1.69	81.58	NoLiq	7.20	88.78	2.00
8.00	41.00	1.50	0.75	0.469	1.46	67.38	NoLiq	7.20	74.58	2.00
10.00	33.00	1.50	0.85	0.582	1.31	55.13	NoLiq	7.20	62.33	2.00
12.00	33.00	1.50	0.85	0.694	1.20	50.51	NoLiq	7.20	57.71	2.00
14.00	43.20	1.50	0.85	0.807	1.11	61.33	40.40	7.20	68.53	2.00
16.00	50.00	1.50	0.95	0.922	1.04	74.19	0.00	0.00	74.19	2.00
18.00	50.00	1.50	0.95	1.038	0.98	69.92	0.00	0.00	69.92	2.00
20.00	50.00	1.50	0.95	1.151	0.93	66.40	0.00	0.00	66.40	2.00
22.00	50.00	1.50	0.95	1.262	0.89	63.43	0.00	0.00	63.43	2.00
24.00	50.00	1.50	0.95	1.374	0.85	60.79	0.00	0.00	60.79	2.00
26.00	50.00	1.50	0.95	1.489	0.82	58.38	0.00	0.00	58.38	2.00
28.00	50.00	1.50	1.00	1.605	0.79	59.19	0.00	0.00	59.19	2.00
30.00	50.00	1.50	1.00	1.718	0.76	57.22	0.00	0.00	57.22	2.00
32.00	50.00	1.50	1.00	1.827	0.74	55.48	0.00	0.00	55.48	2.00
34.00	50.00	1.50	1.00	1.937	0.72	53.89	0.00	0.00	53.89	2.00
36.00	50.00	1.50	1.00	2.045	0.70	52.45	0.00	0.00	52.45	2.00
38.00	44.80	1.50	1.00	2.152	0.68	45.81	0.00	0.00	45.81	2.00
40.00	24.00	1.50	1.00	2.260	0.67	23.95	0.00	0.00	23.95	0.27

CRR is based on water table at 40.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 6.59:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
0.00	0.00	2.00	1.00	2.00	1.39	2.00	0.42	5.00 ^
2.00	0.08	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
4.00	0.15	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
6.00	0.23	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
8.00	0.30	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
10.00	0.38	2.00	1.00	2.00	1.39	2.00	0.41	5.00 ^
12.00	0.45	2.00	1.00	2.00	1.39	2.00	0.40	5.00 ^
14.00	0.52	2.00	1.00	2.00	1.39	2.78	0.40	5.00
16.00	0.60	2.00	1.00	2.00	1.39	2.78	0.40	5.00
18.00	0.67	2.00	1.00	2.00	1.39	2.78	0.40	5.00
20.00	0.75	2.00	1.00	2.00	1.39	2.78	0.40	5.00
22.00	0.82	2.00	1.00	2.00	1.39	2.78	0.39	5.00
24.00	0.89	2.00	1.00	2.00	1.39	2.78	0.39	5.00
26.00	0.97	2.00	1.00	2.00	1.39	2.78	0.39	5.00
28.00	1.04	2.00	1.00	2.00	1.39	2.78	0.39	5.00
30.00	1.12	2.00	0.99	1.98	1.39	2.75	0.39	5.00
32.00	1.19	2.00	0.98	1.95	1.39	2.72	0.38	5.00
34.00	1.26	2.00	0.97	1.93	1.39	2.69	0.37	5.00
36.00	1.33	2.00	0.96	1.91	1.39	2.66	0.37	5.00
38.00	1.40	2.00	0.95	1.89	1.39	2.64	0.36	5.00

40.00	1.47	0.27	0.94	0.25	1.39	0.35	0.35	5.00

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

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

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
0.00	-	-	-	89.44	NoLiq	0.00	89.44
2.00	-	-	-	89.44	NoLiq	0.00	89.44
4.00	-	-	-	89.44	NoLiq	0.00	89.44
6.00	-	-	-	88.78	NoLiq	0.00	88.78
8.00	-	-	-	74.58	NoLiq	0.00	74.58
10.00	-	-	-	62.33	NoLiq	0.00	62.33
12.00	-	-	-	57.71	NoLiq	0.00	57.71
14.00	-	-	-	68.53	40.40	0.00	68.53
16.00	-	-	-	74.19	0.00	0.00	74.19
18.00	-	-	-	69.92	0.00	0.00	69.92
20.00	-	-	-	66.40	0.00	0.00	66.40
22.00	-	-	-	63.43	0.00	0.00	63.43
24.00	-	-	-	60.79	0.00	0.00	60.79
26.00	-	-	-	58.38	0.00	0.00	58.38
28.00	-	-	-	59.19	0.00	0.00	59.19
30.00	-	-	-	57.22	0.00	0.00	57.22
32.00	-	-	-	55.48	0.00	0.00	55.48
34.00	-	-	-	53.89	0.00	0.00	53.89
36.00	-	-	-	52.45	0.00	0.00	52.45
38.00	-	-	-	45.81	0.00	0.00	45.81
40.00	-	-	-	23.95	0.00	0.00	23.95

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

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

Depth ft	CSRsf	/ MSF*	=CSRm	F.S.	Fines %	(N1)60s	Dr %	ec %	dsz in.	dsp in.	S in.

No Settlement of Saturated Sands

Settlement of Saturated Sands=0.000 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

Depth ft	sigma' atm	sigC' atm	(N1)60s	CSRsf	Gmax atm	g*Ge/Gm	g_eff	ec7.5 %	Cec	ec %	dsz in.	dsp in.	S in.
39.95	2.26	1.47	24.48	0.35	1571.33	5.1E-4	0.2310	0.1747	0.80	0.1402	1.68E-3	0.002	0.002
38.00	2.15	1.40	45.81	0.36	1890.26	4.1E-4	0.1200	0.0379	0.80	0.0305	3.65E-4	0.029	0.031
36.00	2.04	1.33	52.45	0.37	1927.32	3.9E-4	0.1047	0.0331	0.80	0.0266	3.19E-4	0.013	0.044
34.00	1.94	1.26	53.89	0.37	1892.77	3.8E-4	0.1000	0.0316	0.80	0.0254	3.05E-4	0.012	0.056
32.00	1.83	1.19	55.48	0.38	1856.48	3.7E-4	0.0950	0.0300	0.80	0.0241	2.89E-4	0.012	0.068
30.00	1.72	1.12	57.22	0.39	1818.57	3.7E-4	0.0898	0.0284	0.80	0.0228	2.73E-4	0.011	0.080
28.00	1.61	1.04	59.19	0.39	1778.01	3.5E-4	0.0817	0.0258	0.80	0.0207	2.49E-4	0.010	0.090
26.00	1.49	0.97	58.38	0.39	1704.69	3.4E-4	0.0766	0.0242	0.80	0.0194	2.33E-4	0.010	0.100

24.00	1.37	0.89	60.79	0.39	1659.35	3.3E-4	0.0687	0.0217	0.80	0.0174	2.09E-4	0.009	0.109
22.00	1.26	0.82	63.43	0.39	1613.03	3.1E-4	0.0616	0.0195	0.80	0.0156	1.88E-4	0.008	0.117
20.00	1.15	0.75	66.40	0.40	1564.44	2.9E-4	0.0805	0.0255	0.80	0.0204	2.45E-4	0.007	0.124
18.00	1.04	0.67	69.92	0.40	1511.51	2.7E-4	0.0667	0.0211	0.80	0.0169	2.03E-4	0.009	0.133
16.00	0.92	0.60	74.19	0.40	1452.90	2.5E-4	0.0548	0.0173	0.80	0.0139	1.67E-4	0.007	0.140
14.00	0.81	0.52	68.53	0.40	1323.24	2.5E-4	0.0502	0.0159	0.80	0.0127	1.53E-4	0.006	0.146
12.00	0.69	0.45	57.71	0.40	1159.15	2.4E-4	0.0487	0.0154	0.80	0.0124	0.00E0	0.004	0.151
10.00	0.58	0.38	62.33	0.41	1089.55	2.2E-4	0.0386	0.0122	0.80	0.0098	0.00E0	0.000	0.151
8.00	0.47	0.30	74.58	0.41	1037.46	1.8E-4	0.0352	0.0111	0.80	0.0089	0.00E0	0.000	0.151
6.00	0.35	0.23	88.78	0.41	952.32	1.5E-4	0.0269	0.0085	0.80	0.0068	0.00E0	0.000	0.151
4.00	0.23	0.15	89.44	0.41	779.47	1.2E-4	0.0208	0.0066	0.80	0.0053	0.00E0	0.000	0.151
2.00	0.12	0.08	89.44	0.41	551.18	8.8E-5	0.0163	0.0051	0.80	0.0041	0.00E0	0.000	0.151
0.00	0.00	0.00	89.44	0.42	5.09	8.2E-7	0.0010	0.0003	0.80	0.0003	0.00E0	0.000	0.151

Settlement of Unsaturated Sands=0.151 in.
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=2.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Unsaturated Sands=0.151 in.
Differential Settlement=0.075 to 0.100 in.

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

1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)

SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT) [atm (tsf)]
fs	Friction from CPT testing [atm (tsf)]
Rf	Ratio of fs/qc (%)
gamma	Total unit weight of soil
gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size
Dr	Relative Density
sigma	Total vertical stress [atm]
sigma'	Effective vertical stress [atm]
sigC'	Effective confining pressure [atm]
rd	Acceleration reduction coefficient by Seed
a_max.	Peak Ground Acceleration (PGA) in ground surface
mZ	Linear acceleration reduction coefficient X depth
a_min.	Minimum acceleration under linear reduction, mZ
CRRv	CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksig	Overburden stress correction factor for CRR7.5
CRRm	After magnitude scaling correction CRRm=CRRv * MSF
MSF	Magnitude scaling factor from M=7.5 to user input M
CSR	Cyclic stress ratio induced by earthquake
CSRfs	CSRfs=CSR*fs1 (Default fs1=1)
fs1	First CSR curve in graphic defined in #9 of Advanced page
fs2	2nd CSR curve in graphic defined in #9 of Advanced page
F.S.	Calculated factor of safety against liquefaction F.S.=CRRm/CSRsf
Cebs	Energy Ratio, Borehole Dia., and Sampling Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60	Fines correction of SPT

(N1)60f	(N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq	Overburden stress correction factor
qc1	CPT after Overburden stress correction
dqc1	Fines correction of CPT
qc1f	CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n	CPT after normalization in Robertson's method
Kc	Fine correction factor in Robertson's Method
qc1f	CPT after Fines correction in Robertson's Method
Ic	Soil type index in Suzuki's and Robertson's Methods
(N1)60s	(N1)60 after settlement fines corrections
CSRm	After magnitude scaling correction for Settlement calculation CSRm=CSRsf / MSF*
CSRfs	Cyclic stress ratio induced by earthquake with user inputed fs
MSF*	Scaling factor from CSR, MSF*=1, based on Item 2 of Page C.
ec	Volumetric strain for saturated sands
dz	Calculation segment, dz=0.050 ft
dsz	Settlement in each segment, dz
dp	User defined print interval
dsp	Settlement in each print interval, dp
Gmax	Shear Modulus at low strain
g_eff	gamma_eff, Effective shear Strain
g*Ge/Gm	gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5	Volumetric Strain for magnitude=7.5
Cec	Magnitude correction factor for any magnitude
ec	Volumetric strain for unsaturated sands, ec=Cec * ec7.5
NoLiq	No-Liquefy Soils

References:

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

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

6136 Manchester Avenue

6136 W Manchester Ave, Los Angeles, CA 90045, USA

Latitude, Longitude: 33.959365, -118.3931243

Date	11/4/2021, 3:30:19 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.85	MCE_R ground motion. (for 0.2 second period)
S_1	0.65	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.85	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.233	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.792	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.872	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.85	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	2.035	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.464	Factored deterministic acceleration value. (0.2 second)
S1RT	0.65	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.72	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.834	Factored deterministic acceleration value. (1.0 second)
PGAd	0.998	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.909	Mapped value of the risk coefficient at short periods

Type	Value	Description
C_{R1}	0.903	Mapped value of the risk coefficient at a period of 1 s

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

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

Input

Edition

Dynamic: Conterminous U.S. 2014 (u...

Latitude

Decimal degrees

33.959365

Longitude

Decimal degrees, negative values for western longitudes

-118.3931243

Site Class

259 m/s (Site class D)

Spectral Period

Peak Ground Acceleration

Time Horizon

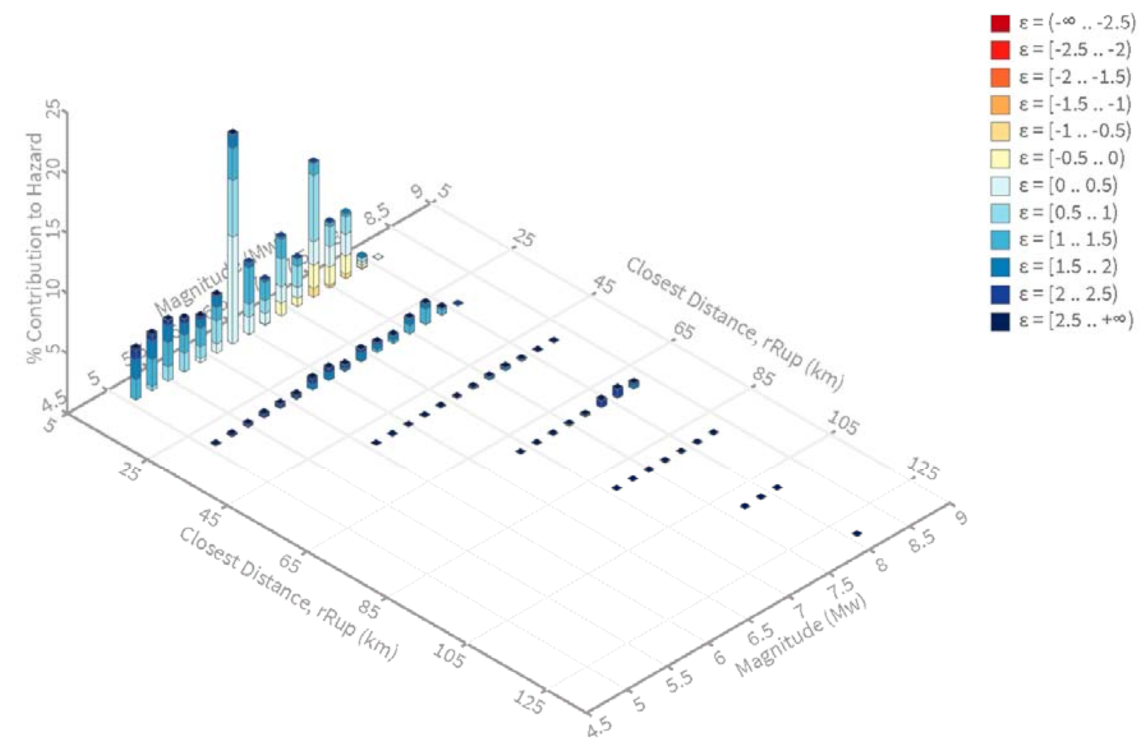
Return period in years

475

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 475 yrs
Exceedance rate: 0.0021052632 yr^{-1}
PGA ground motion: 0.48793801 g

Recovered targets

Return period: 507.02555 yrs
Exceedance rate: 0.0019722872 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.11 %

Mean (over all sources)

m: 6.59
r: 12.66 km
ε_0: 0.94 σ

Mode (largest m-r bin)

m: 6.34
r: 6.61 km
ε_0: 0.66 σ
Contribution: 17.41 %

Mode (largest m-r-ε_0 bin)

m: 6.36
r: 4.47 km
ε_0: 0.35 σ
Contribution: 8.87 %

Discretization

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

Epsilon keys

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

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							33.41
Newport-Inglewood alt 1 [7]		4.00	6.62	0.29	118.354°W	33.968°N	75.65	10.55
Palos Verdes [14]		11.02	6.97	0.92	118.471°W	33.885°N	220.95	6.21
Santa Monica alt 1 [0]		11.52	7.21	0.74	118.453°W	34.049°N	331.19	2.53
Compton [3]		9.41	7.20	-0.16	118.443°W	33.877°N	206.52	2.20
Newport-Inglewood alt 1 [6]		7.85	7.55	0.12	118.316°W	33.933°N	112.18	1.76
UC33brAvg_FM32	System							29.90
Newport-Inglewood alt 2 [7]		5.06	6.66	0.39	118.346°W	33.979°N	63.53	6.94
Palos Verdes [14]		11.02	6.97	0.92	118.471°W	33.885°N	220.95	5.58
Compton [3]		9.41	7.38	-0.24	118.443°W	33.877°N	206.52	2.25
Santa Monica alt 2 [2]		11.18	7.24	0.73	118.460°W	34.043°N	326.45	1.98
Hollywood [2]		14.23	6.94	1.12	118.422°W	34.084°N	348.99	1.78
Puente Hills (LA) [1]		12.68	7.17	0.78	118.316°W	34.049°N	35.31	1.46
Newport-Inglewood alt 2 [6]		8.80	7.55	0.17	118.305°W	33.933°N	109.43	1.29
UC33brAvg_FM31 (opt)	Grid							18.79
PointSourceFinite: -118.393, 34.000		6.73	5.67	0.89	118.393°W	34.000°N	0.00	2.55
PointSourceFinite: -118.393, 34.000		6.73	5.67	0.89	118.393°W	34.000°N	0.00	2.55
PointSourceFinite: -118.393, 34.018		8.01	5.70	1.07	118.393°W	34.018°N	0.00	1.93
PointSourceFinite: -118.393, 34.018		8.01	5.70	1.07	118.393°W	34.018°N	0.00	1.93
PointSourceFinite: -118.393, 34.054		10.77	5.83	1.35	118.393°W	34.054°N	0.00	1.66
PointSourceFinite: -118.393, 34.054		10.77	5.83	1.35	118.393°W	34.054°N	0.00	1.66
UC33brAvg_FM32 (opt)	Grid							17.90
PointSourceFinite: -118.393, 34.000		6.72	5.68	0.88	118.393°W	34.000°N	0.00	2.28
PointSourceFinite: -118.393, 34.000		6.72	5.68	0.88	118.393°W	34.000°N	0.00	2.28
PointSourceFinite: -118.393, 34.018		8.00	5.71	1.07	118.393°W	34.018°N	0.00	1.85
PointSourceFinite: -118.393, 34.018		8.00	5.71	1.07	118.393°W	34.018°N	0.00	1.85
PointSourceFinite: -118.393, 34.054		10.75	5.84	1.35	118.393°W	34.054°N	0.00	1.51
PointSourceFinite: -118.393, 34.054		10.75	5.84	1.35	118.393°W	34.054°N	0.00	1.51

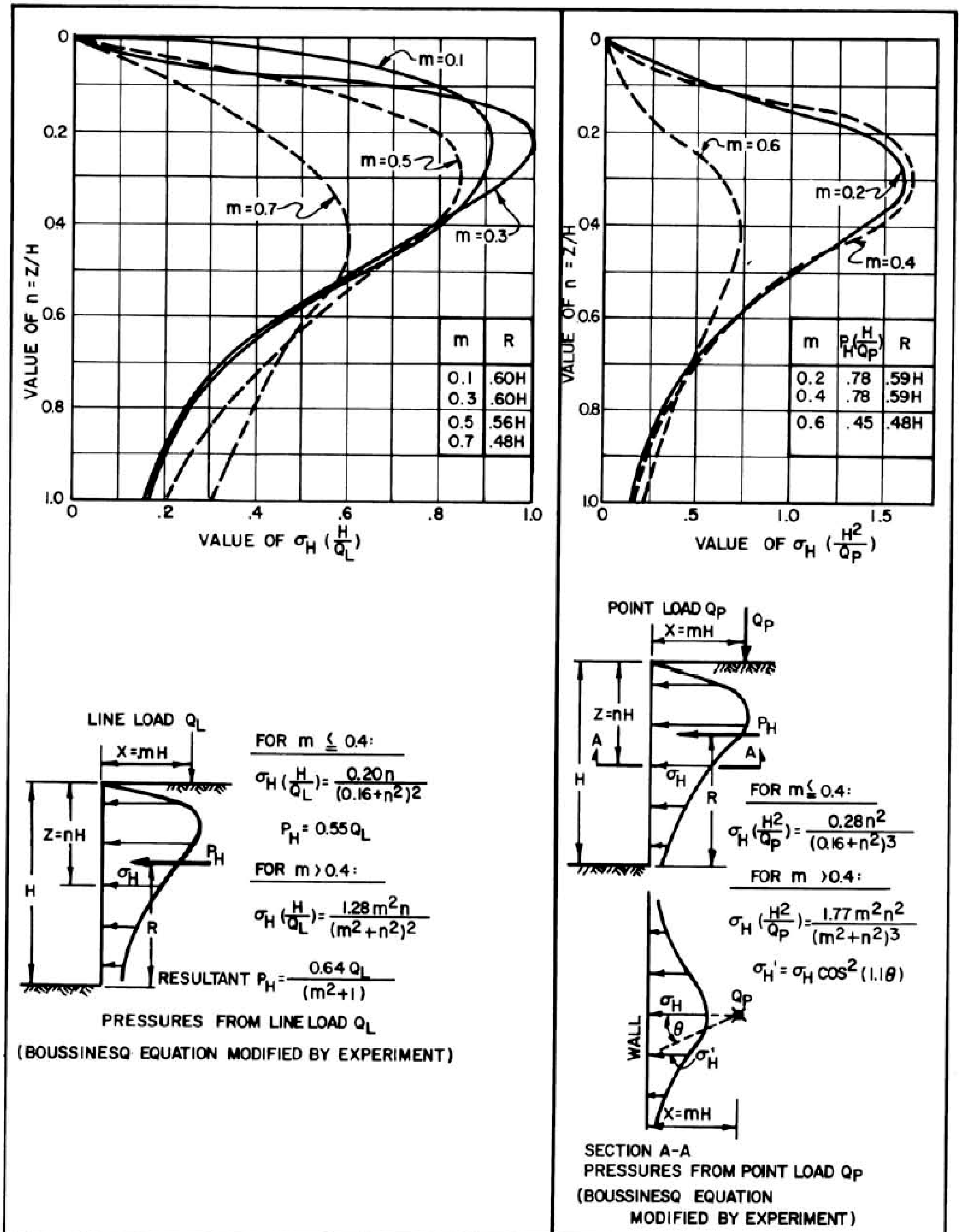


FIGURE 11
Horizontal Pressures on Rigid Wall from Surface Load

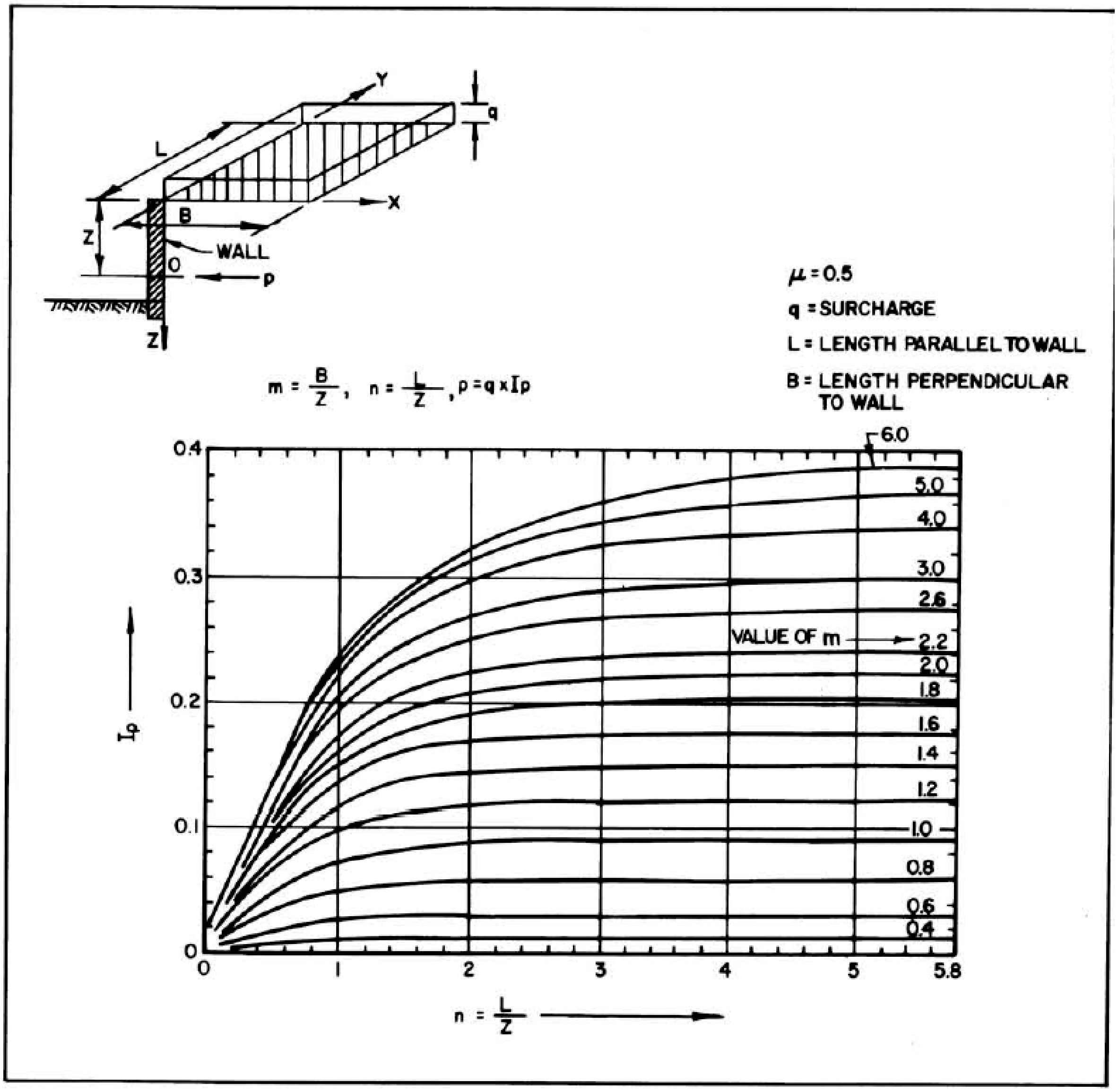


FIGURE 12
Lateral Pressure on an Unyielding Wall due to
Uniform Rectangular Surface Load

APPENDIX IV

REFERENCES

1. Compaction Report by The Twining Laboratories covering the subject site and dated May 23, 1957.
2. Compaction Approval Letter by the City of Los Angeles, Department of Building and Safety covering the subject site and dated June 5, 1957.
3. Compaction Reports by Enviropro, Inc. covering the subject site and dated May 19, 1993, August 24, 1993, and October 21, 1993.
4. Compaction Approval Letter by the City of Los Angeles, Department of Building and Safety covering the subject site and dated November 2, 1993.
5. Geotechnical Engineering Report by Jerry Kovacs & Associates covering the subject site and dated January 20, 1997.
6. Approval Letter by the City of Los Angeles, Department of Building and Safety covering the subject site and dated February 11, 1997.
7. Compaction Report by Jerry Kovacs & Associates covering the subject site and dated July 14, 1997.
8. Compaction Approval Letter by the City of Los Angeles, Department of Building and Safety covering the subject site and dated July 22, 1997.
9. Geotechnical Engineering Report by Giles Engineering Associates, Inc. covering the subject site and dated June 11, 1998.
10. Approval Letter by the City of Los Angeles, Department of Building and Safety covering the subject site and dated July 21, 1998.
11. Compaction Report by Giles Engineering Associates, Inc. covering the subject site and dated November 9, 1998.
12. Compaction Approval Letter by the City of Los Angeles, Department of Building and Safety covering the subject site and dated January 22, 1999.
13. Bowles, Joseph, E., Foundation Analysis and Design (McGraw-Hill, New York: 1988).
14. 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.
15. California Department of Conservation, Division of Mines and Geology, March 25, 1999, State of California Seismic Hazard Zones Map of the Venice Quadrangle.
16. California Department of Conservation, Division of Mines and Geology, 1998, Seismic Hazard Zone Report for the Venice 7.5 Minute Quadrangle, Los Angeles County, California. Seismic Hazard Zone Report 98-27.
17. Dibblee, T. W., 1991, Geologic Map of the Venice Quadrangle, Los Angeles County, California: Dibblee Geological Foundation.
18. Monahan, Edward J., PE, Construction of and on Compacted Fills (Wiley & Sons, New York: 1986).
19. Naval Facilities Engineering Command Foundations and Earth Structures - Design Manual 7.02 (Naval Publications and Forms Center, Philadelphia: 1986).

20. Poulos, H. G., and Davis, E. H., Pile Foundation Analysis and Design (Wiley & Sons, New York: 1980).

21. Taylor, Donald W., Fundamentals of Soil Mechanics (Wiley & Sons, New York: 1948).

22. Terzaghi, Karl, Peck, Ralph B., Mesri, Gholamreza, Soil Mechanics in Engineering Practice (Wiley & Sons, New York: 1996).

Appendix E.2

Update Report

July 27, 2022 Project 6058

CV 6136 Manchester, LLC
c/o CityView
Attn: Stephen Roberts
1901 Avenue of the Stars, Suite 1900
Los Angeles, California 90067

Subject: **UPDATE REPORT**
6136 West Manchester Avenue
8651 South La Tijera Boulevard
Westchester, California

References:

1) Preliminary Geologic and Geotechnical Engineering report by GeoConcepts, Inc. covering the subject site, dated February 7, 2022.

Dear Mr. Roberts:

Pursuant to your request, presented herein is a geotechnical update report to address the proposed development. Currently it is proposed to develop the subject site with a new five to eight story mixed use building with two to three levels of subgrade parking based on the preliminary building plans provided by AC Martin dated July 20, 2022. Final building plans have not been prepared and await the updated recommendations provided herein. These plans should be reviewed by GeoConcepts, Inc. to ensure that updated recommendations have been followed.

The subject site was previously explored by this firm on November 18, 2021 to address a new five to eight story mixed use building with one level of subgrade parking (Reference 1). The site exploration consisted of seven borings that were excavated on the site to a maximum depth of (41) feet utilizing a drill rig. The previous exploration generally encountered fill and alluvium.

RECOMMENDATIONS

Excavations

Excavations ranging in vertical height up to 35 feet will be required for the subgrade parking excavations. Minor amounts of remedial grading, up to three feet, may be required at the base of the excavation due to possible disturbance of the subgrade soils during excavation. Conventional excavation equipment may be used to make these excavations. Excavations should expose alluvium. These soils are suitable for non-surcharged vertical excavations up to 5 feet.

Excavations above 5 feet should be trimmed back at 1:1 (H:V) slope gradient 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 500 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.

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.

In the event groundwater is encountered, 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.

Vibrated Piles

Where piles are installed by vibration techniques, the passive pressure may be assumed to mobilize across a width equal to 1.4 the flange width. The allowable passive value may be doubled for isolated piles, spaced a minimum of 2 times the pile diameter. To develop the full lateral value, provisions should be implemented to assure firm contact between the soldier piles and the undisturbed alluvium. If a vibratory method of solider pile installation is utilized, predrilling may be performed prior to installation of the steel beams. If predrilling is performed, it is recommended that the bore diameter be at least 3 inches smaller than the largest dimension of the pile to prevent excessive loss in the frictional component of the pile capacity. Predrilling should not be conducted below the proposed excavation bottom.

If a vibratory method is utilized, the owner should be aware of the potential risks associated with vibratory efforts, which typically involve inducing settlement within the vicinity of the pile which could result in a potential for damage to existing improvements in the area. The level of vibration that results from the installation of the piles should not exceed a threshold where occupants of nearby structures are disturbed, despite higher vibration tolerances that a building may endure without deformation or damage. The main parameter used for vibration assessment is peak particle velocity in units of inch per second (in/sec). The acceptable range of peak particle velocity should be evaluated based on the age and condition of adjacent structures, as well as the tolerance of human response to vibration. Based on Table 19 of the *Transportation and Construction Induced Vibration Guidance Manual* (Caltrans 2004), a continuous source of vibrations (ex. vibratory pile driving) which generates a maximum peak particle velocity of 0.5 in/sec is considered tolerable for modern industrial / commercial buildings and new residential structures. The Client should be aware that a lower value may be necessary if older or fragile structures are in the immediate vicinity of the site.

Vibrations should be monitored and record with seismographs during pile installation to detect the magnitude of vibration and oscillation experienced by adjacent structures. If the vibrations exceed the acceptable range during installation, the shoring contractor should modify the installation procedure to reduce the values to within the acceptable range. Vibration monitoring is not the responsibility of the Geotechnical Engineer. If vibratory construction techniques will be implemented, it is recommended that qualified consultant be retained to provide site specific recommendations for vibration thresholds and monitoring.

Lagging

To develop the full lateral support, provisions should be implemented to assure firm contact between the lagging and the undisturbed earth materials. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the lagging to the earth materials. It is recommended that the lagging and slurry backfill be installed the same day as excavation.

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 be limited to a maximum of 400 pounds per square foot.

Lateral Pressures

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

Shoring Height (ft)	Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
15 feet	24 pcf	18H psf
25 feet	32 pcf	22H psf
35 feet	36 pcf	24H psf

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

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

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. The maximum deflection shall not exceed one-half inch (1/2) inch at the top of the shored embankment where a structure is within 1:1 (h:v) plane projected up from the base of the excavation, and for a maximum lateral deflection of (1) inch provided there are no structures within a 1:1 (h:v) plane projected up from the base of excavation. 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.

Retaining Walls

Retaining walls should be designed to resist an active earth pressure such as that exerted by compacted backfill. Retaining walls up to 25 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.

Drained Condition

Surface Slope of Retained Material Horizontal to Vertical	Active Equivalent Fluid Weight p.c.f.	At-Rest Pressure Fluid Weight p.c.f.
Level (15 Feet)	40	70
Level (25 Feet)	47	70
Level (35 Feet)	51	70

Un-drained (Hydrostatic) Condition

Surface Slope of Retained Material Horizontal to Vertical	Hydrostatic Active Equivalent Fluid Weight p.c.f.	Hydrostatic At-Rest Pressure Fluid Weight p.c.f.
Level (15 Feet)	80	100
Level (25 Feet)	85	100
Level (35 Feet)	90	100

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

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.

   As an alternative to the perforated pipe system, the drainage system may consist of rock pockets. The rock pockets should consist of a 1’x1’x1’ of ¾” gravel spaced at a maximum of 8’ on center. The weep hole pipe through the wall at each rock pocket should be a minimum 4” diameter. Where space does not permit a 1’x1’x1’ gravel pocket (such as where space behind the wall is less than 12”) the thickness of the gravel pocket may be reduced to minimum of 4” provided that H’xW’X4”>1 cubic foot. A request for modification may be required by the City of Los Angeles for gravel pockets with the reduced thickness.

   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

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 25 feet in height may be designed per the following table. The seismic equivalent fluid pressure is in addition to static active earth pressures.

The seismic loading is based on a horizontal acceleration coefficient of ½ of 2/3 PGA_M = 0.29.

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

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

Respectfully submitted,
GEOCONCEPTS, INC.

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

Raffi Dermendjian
Project Engineer
PE C. 88261
RD: 6058-3

Enclosures: Geologic Map (in Pocket)

Distribution: (3) Addressee

Retaining Walls (35 Feet High with Level Backslope)

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:	Qal	WALL HEIGHT	35 feet
SHEAR DIAGRAM:	B-5@10	BACKSLOPE ANGLE:	0 degrees
COHESION:	200 psf	SURCHARGE:	0 pounds
PHI ANGLE:	32 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):	133.3 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 22.6 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	56 degrees
AREA OF TRIAL FAILURE WEDGE	411.2 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	55514.7 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	39.3 feet
DEPTH OF TENSION CRACK	2.4 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	22.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**30784.4 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**50.3 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**pcf**

Seismic Retaining Walls (35 Feet High with Level Backslope)

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:	Qal	WALL HEIGHT	35 feet
SHEAR DIAGRAM:	B-5@10	BACKSLOPE ANGLE:	0 degrees
COHESION:	200 psf	SURCHARGE:	0 pounds
PHI ANGLE:	32 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):	200.0 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 32.0 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	48 degrees
AREA OF TRIAL FAILURE WEDGE	544.6 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	73526.4 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	41.8 feet
DEPTH OF TENSION CRACK	3.9 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	28.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**35022.6 pounds**

Hydrostatic Retaining Walls (35 Feet High with Level Backslope)

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:	Qal	WALL HEIGHT	35 feet
SHEAR DIAGRAM:	B-5@10	BACKSLOPE ANGLE:	0 degrees
COHESION:	200 psf	SURCHARGE:	0 pounds
PHI ANGLE:	32 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	75 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):	133.3 psf	FINAL TENSION CRACK:	40 feet

PHID = ATAN(TAN(PHI)/FS) = 22.6 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	56 degrees
AREA OF TRIAL FAILURE WEDGE	403.5 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	30261.6 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	35.8 feet
DEPTH OF TENSION CRACK	5.3 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	20.0 feet
CALCULATED HORIZONTAL THRUST ON WALL	**14670.0 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**24.0 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**pcf**

Shoring Piles (35 Feet High with Level Backslope)

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:	Qal	RETAINED LENGTH	35 feet
SHEAR DIAGRAM:	B-5@10	BACKSLOPE ANGLE:	0 degrees
COHESION:	200 psf	SURCHARGE:	0 pounds
PHI ANGLE:	32 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):	160.0 psf	FINAL TENSION CRACK:	40 feet

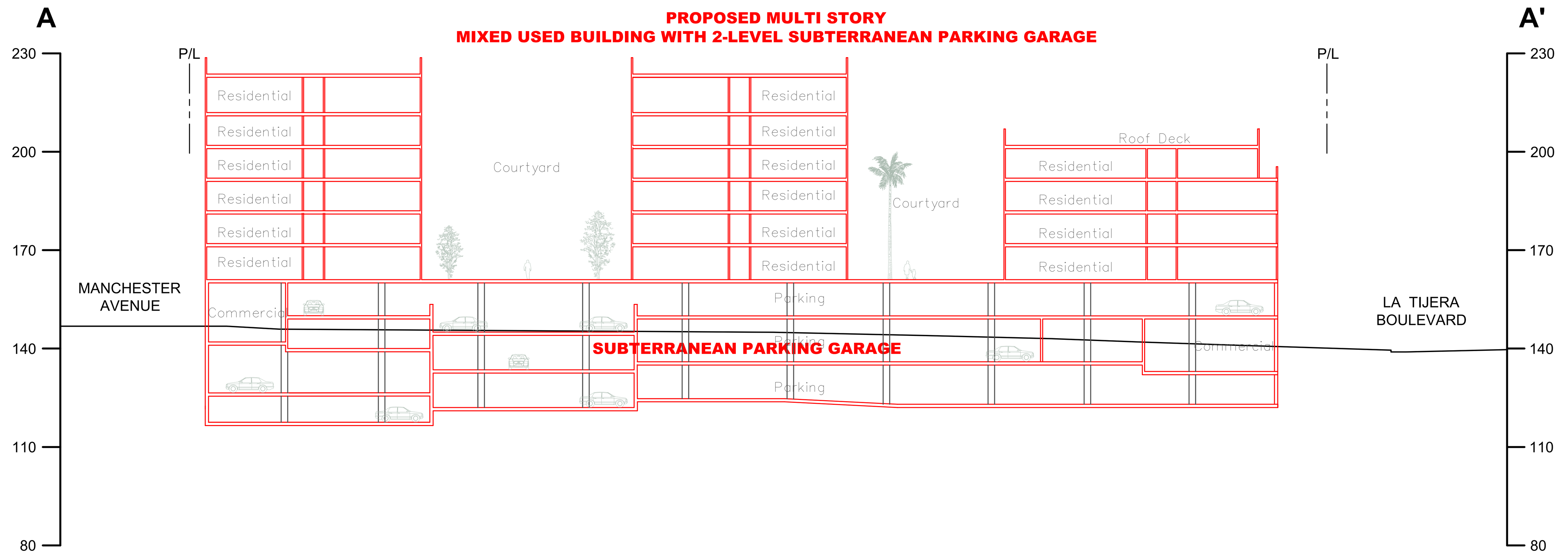
PHID = ATAN(TAN(PHI)/FS) = 26.6 degrees

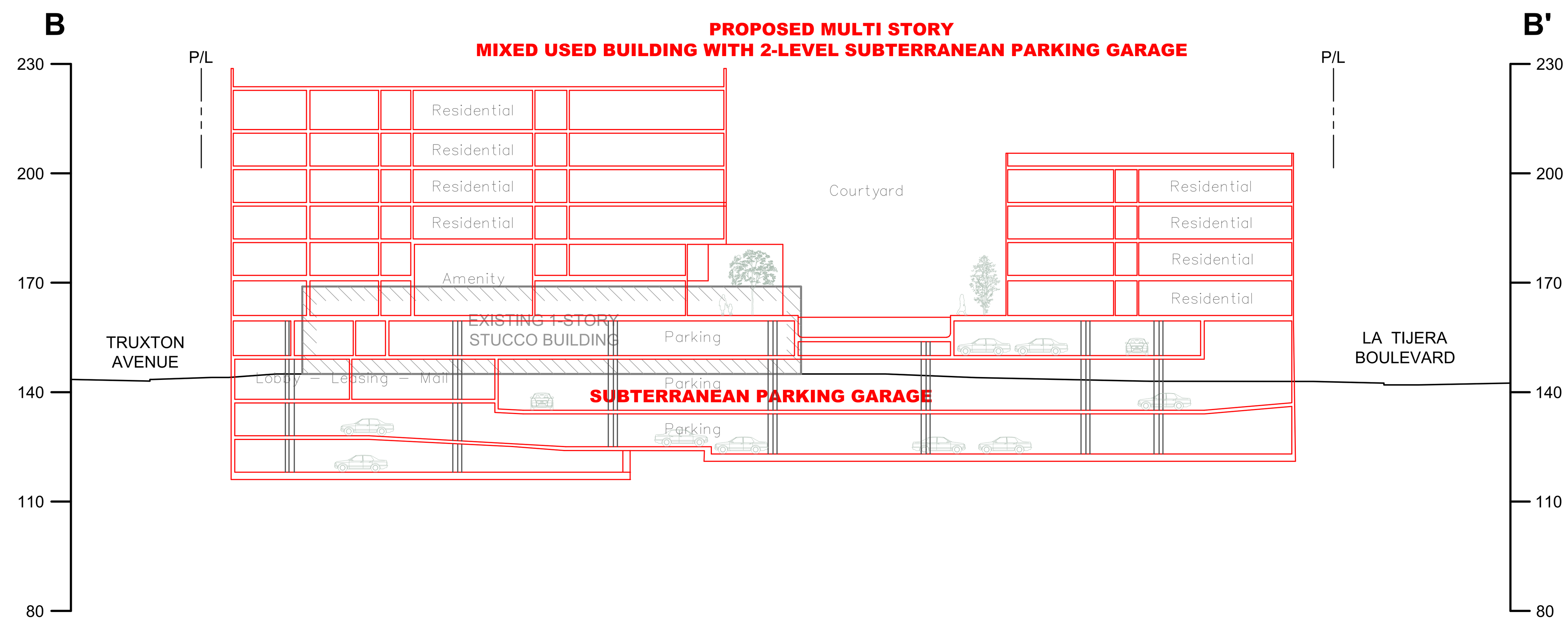
HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

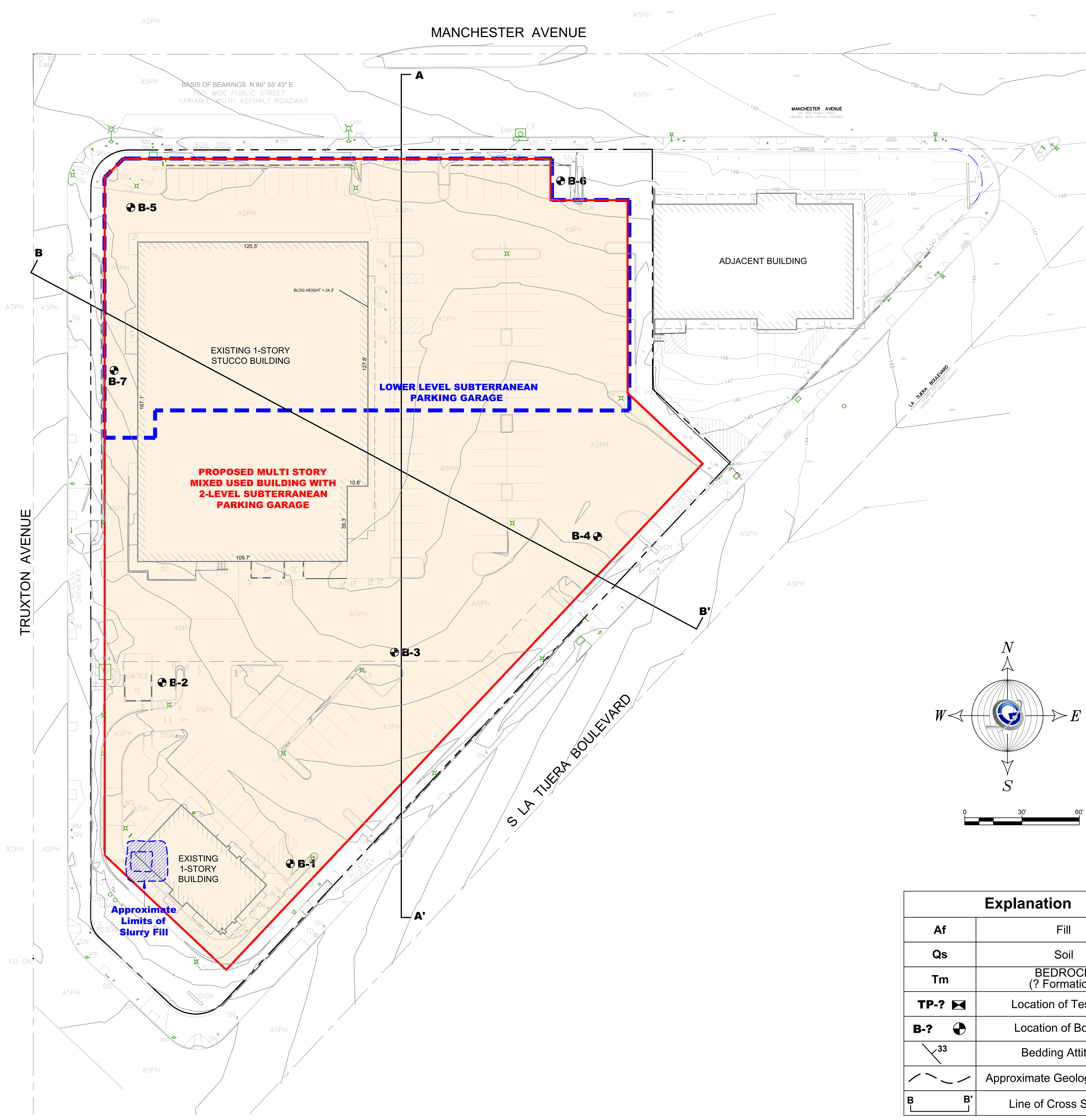
VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	58 degrees
AREA OF TRIAL FAILURE WEDGE	376.1 square feet
TOTAL EXTERNAL SURCHARGE	0.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	45136.8 pounds
NUMBER OF TRIAL WEDGES ANALYZED	1116 trials
LENGTH OF FAILURE PLANE	35.9 feet
DEPTH OF TENSION CRACK	4.6 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	19.0 feet
CALCULATED THRUST ON PILE	**21580.3 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**35.2 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**pcf**







Explanation	
Af	Fill
Qs	Soil
Tm	BEDROCK (? Formation)
TP-?	Location of Test Pits
B-?	Location of Borings
33	Bedding Attitude
	Approximate Geologic Contact
B B'	Line of Cross Section

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 and Cross Sections

Survey By: PARTNER ENGINEERING & SCIENCE
Architecture Plans By: AC MARTIN

Project Address:
6136 West Manchester Avenue
8651 South La Tijera Boulevard
Westchester, California

Date: July 2022
Scale: 1" = 30'
Job No. 6058-3