APPENDIX F:
Geotechnical Investigation Reports



This page intentionally left blank.

APPENDIX F.1

GEOTECHNICAL INVESTIGATION FOR PROPOSED L.A. CO. - USC MEDICAL CENTER CHILD CARE PROJECT N. STATE STREET BETWEEN ZONAL AVENUE AND MARENGO STREET LOS ANGELES, CA

RMA Group
Every Project Matters | www.rmacompanies.com

GEOTECHNICAL CONSULTANTS

GEOTECHNICAL INVESTIGATION
FOR
PROPOSED L.A. CO. - USC MEDICAL CENTER CHILD CARE PROJECT
N. STATE STREET BETWEEN ZONAL AVENUE AND MARENGO STREET
LOS ANGELES, CA

for

Los Angeles Co. Dept. of Public Works
Project Management (PMD 1)
900 South Fremont Avenue
5th Floor
Alhambra, CA 91803

March 30, 2018

18-0056-01

RMA Group
Every Project Matters | www.rmacompanies.com

GEOTECHNICAL CONSULTANTS

March 30, 2018

Los Angeles Co. Dept. of Public Works
Project Management (PMD 1)
900 South Fremont Ave
5th Floor
Alhambra, CA 91803

Attention: Mr. Nik Sae-Low
Project Manager

Subject: Geotechnical Investigation for
Proposed L.A. Co. - USC Medical Center Child Care Project
N. State Street between Zonal Avenue and Marengo Street
Los Angeles, CA

Dear Mr. Sae-Low:

In accordance with your request, a geotechnical investigation has been completed for the above-referenced project. The report addresses both engineering geologic and geotechnical conditions. The results of the investigation are presented in the accompanying report, which includes a description of site conditions, results of our field exploration and laboratory testing, conclusions, and recommendations.

We appreciate this opportunity to be of continued service to you. If you have any questions regarding this report, please do not hesitate to contact us at your convenience.

Respectfully submitted,

RMA Group

Jorge Meneses, PE | GE | PhD | D.GE | F. ASCE
Principal Geotechnical Engineer
GE 3041

REGISTERED PROFESSIONAL ENGINEER
JORGE F. MENESES
No. GE 3041
Exp. 06/18
GEOTECHNICAL
STATE OF CALIFORNIA

Gary Wallace, PG | CEG
Vice President - Geology
CEG 1255

PROFESSIONAL GEOLOGIST
GARY W. WALLACE
No. 1255
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

TABLE OF CONTENTS

		PAGE
1.00	**INTRODUCTION**	1
1.01	Purpose	1
1.02	Scope of the Investigation	1
1.03	Site Location and Description	1
1.04	Current and Past Land Usage	2
1.05	Planned Usage	2
1.06	Investigation Methods and Limitations	2
2.00	**FINDINGS**	3
2.01	Geologic Setting	3
2.02	Earth Materials	3
2.03	Expansive Soils	3
2.04	Surface and Groundwater Conditions	4
2.05	Faults	4
2.06	Historic Seismicity	4
2.07	Flooding Potential	4
2.08	Landslides	4
3.00	**CONCLUSIONS AND RECOMMENDATIONS**	4
3.01	General Conclusion	4
3.02	General Earthwork and Grading	5
3.03	Earthwork Shrinkage and Subsidence	5
3.04	Removals and Overexcavation	5
3.05	Rippability and Rock Disposal	6
3.06	Subdrains	6
3.07	Slopes	6
3.08	Faulting	6
3.09	Seismic Design Parameters	6
3.10	Liquefaction and Secondary Earthquake Hazards	7
3.11	Foundations	8
3.12	Foundation Setbacks from Slopes	9
3.13	Slabs on Grade	9
3.14	Miscellaneous Concrete Flatwork	10

TABLE OF CONTENTS
(Continued)

		PAGE
3.15	Footing Excavation and Slab Preparations	11
3.16	Lateral Load Resistance	11
3.17	Drainage and Moisture Proofing	12
3.18	Cement Type and Corrosion Potential	13
3.19	Temporary Slopes	13
3.20	Utility Trench Backfill	15
3.21	Pavement Sections	15
3.22	Plan Review	16
3.23	Geotechnical Observation and Testing During Rough Grading	16
3.24	Post-Grading Geotechnical Observation and Testing	16
3.25	Los Angeles County Building Code Section 111 Statement	17
4.00	**CLOSURE**	17

FIGURES AND TABLES

Figure 1	Site Location Map
Figure 2	Regional Geologic Map
Figure 3	Site Geologic Map
Figures 4 & 5	Geologic Cross Sections
Figure 6	Regional Fault Map
Figure 7	Seismic Hazard Zone Map
Figure 8	Static Slope Stability Analysis
Figure 9	Pseudo-Static Slope Stability Analysis
Table 1	Notable Faults within 100 Km
Table 2	Historical Strong Earthquakes

APPENDICES

Appendix A	Field Investigation	A1
Appendix B	Laboratory Tests	B1
Appendix C	General Earthwork and Grading Specifications	C1
Appendix D	References	D1

1.00 INTRODUCTION

1.01 Purpose

A geotechnical investigation has been completed for the proposed L.A. Co. - USC Medical Center Child Care Project in Los Angeles, California. The purpose of the investigation was to summarize geotechnical and geologic conditions at the site, to assess their potential impact on the proposed development, and to develop geotechnical and engineering geologic design parameters.

1.02 Scope of the Investigation

The general scope of this investigation included the following:

- Review of published and unpublished geologic, seismic, groundwater and geotechnical literature.
- Examination of aerial photographs.
- Contacting of underground service alert to locate onsite utility lines.
- Logging, sampling and backfilling of 3 exploratory borings drilled with a bucket auger drill rig.
- Laboratory testing of representative soil and bedrock samples.
- Geotechnical evaluation of the compiled data.
- Preparation of this report presenting our findings, conclusions and recommendations.

Our scope of work did not include a preliminary site assessment for the potential of hazardous materials onsite.

1.03 Site Location and Description

The proposed child care facility will be located at the top of a hill on the west side of North State Street between Zonal Avenue and Marengo Street in Los Angeles, California. This location is just west of the Los Angeles County – USC Medical Center (see Figure 1).

The site is currently undeveloped with the following exceptions:

1. The area immediately adjacent to North State Street is landscaped and contains some underground utilities. A chain link fence separates this landscaped area from the undeveloped portion of the site the west.
2. An east-west trending underground utility tunnel crosses the central of the site. Details regarding how the tunnel was constructed were not provided. It is currently not known if the tunnel excavation was created as an open cut that was later backfilled or if the tunnel was bored through the bedrock underlying the site. The exact (surveyed) location of the tunnel in the subsurface was also not known at the time of our study. A portion of another utility tunnel borders the western side of the site and offsite to the southeast.
3. Access to the interior of the site is provided by a concrete sidewalk and gravel driveway.

The west portion of the site is covered by numerous oak trees.

The site slopes gently to the west and southwest and gradually steepens to in the west-central portion of the site. Gradient

of the property then flattens on much of the western side of the site. Elevations within the property range from about 345 to 361 feet above sea level, producing a total relief of about 16 feet. Slopes extend offsite to the west and southwest to elevations of a 333 to 336 resulting relief of about 25 to 28 feet.

1.04 Current and Past Land Usage

The site is currently used by the County Parks and Recreation Department for storage of Gardener's equipment. We are not aware of any other prior usage of the site.

1.05 Planned Usage

It is our understanding that the site will be developed with a child care facility. Buildings covering approximately proposed 5,700 square feet are proposed in the southeast part of the site. A site plan provided to us shows building footprint is close proximity to the utility tunnel that crosses the site, the southwest property line, and an offsite slope that descends to the southwest. Reportedly the buildings will be one-story "stick-built" structures. We were not provided with grading plans but we understand pad grades will be close to existing grades. Play yards covering approximately 9,000 square feet are proposed in the western and northern parts of the site. It is not known if any slopes or retaining walls will be constructed to level the site or portions of the site.

Our investigation was performed prior to the preparation of grading or foundation plans. To aid in preparation of this report, we utilized the following assumptions:

- Maximum foundation loads of 2 to 3 kips per linear foot for continuous footings and 60 kips for isolated spread footings.
- Cuts and fills will be less than 5 feet.

1.06 Investigation Methods and Limitations

Our investigation consisted of office research, field exploration, laboratory testing, review of the compiled data, and preparation of this report. It has been performed in a manner consistent with generally accepted engineering and geologic principles and practices, and has incorporated applicable requirements of California Building Code. Definitions of technical terms and symbols used in this report include those of the ASTM International, the California Building Code, and commonly used geologic nomenclature.

Technical supporting data are presented in the attached appendices. Appendix A presents a description of the methods and equipment used in performing the field exploration and logs of our subsurface exploration. Appendix B presents a description of our laboratory testing and the test results. Standard grading specifications and references are presented in Appendices C and D, respectively.

The following limiting conditions occurred during our investigation:

- Much of the site is cover with a canopy of trees thus limiting drill sites because of overhead clearance issues.
- A previously unknown water line was struck in one boring, thus limiting the depth of the boring. An alternate boring was not attempted because the location of this pipeline in other areas was not known and the possible of other unmarked utilities in the area could not be ruled out.
- Details of the tunnel that passes through the site were unknown, including its bearing, depth, original construction method and the possibility unknown utilities overlying the tunnel. The County project manager was not able to locate any plans for the tunnel.

- Grading plans were not available, thus the possibility of cutting, filling, and construction of slopes or retaining walls was not known.

2.00 FINDINGS

2.01 Geologic Setting

The site is located in the Elysian Park - Repetto Hills area of Los Angeles County. This area consists of low lying hills and intervening flood plains. The hills are composed primarily of Miocene and Pliocene age marine, sedimentary bedrock; principally the Puente, Topanga and Fernando formations. The sedimentary bedrock reach a maximum aggregate thickness in the area of about 20,000 feet and rest on a Mesozoic age plutonic basement complex. As discussed below, sandstone and siltstone of the Puente formation was encountered in our investigation of the site.

Geology in the vicinity of the site is shown on Figure 2.

2.02 Earth Materials

Our subsurface investigation encountered artificial fill, topsoil and sedimentary bedrock.

Fill consisting of brown silty and with pieces of concrete and brick was encountered in Boring B-3 on the western side of the site. It appears the fill may have been placed as backfill against the utility tunnel offsite to the west and southwest. A live water line was hit by the boring at a depth of 5 feet preventing the continuation of the boring through the fill. A utility tunnel that crosses the central part of the site may be covered with artificial fill, unless it was constructed by tunneling through bedrock. A boring was not attempted in that area because of the possible presence of unknown utility lines over or near the tunnel and because the location and depth of the tunnel was not precisely known. Records or plans of the utility tunnel construction were not available at the time our study was performed. A thin layer of artificial fill and topsoil, approximately 1 to 3 feet thick, was also encountered in Borings B-1 and B-2 which were drilled in the central part of the site.

Boring B-1 and B-2 encountered sedimentary bedrock at depth of about 1 to 3 feet below the existing ground surface. According to Lama (1970), the bedrock is part of the Tertiary age Puente formation. Dibblee (1989) classified the bedrock as unnamed Tertiary age sandstone. For our report, we have adopted the classification of Lamar – Puente formation. In borings B-1 and B-2 the bedrock was found to consist of gray and yellow siltstone and sandstone that dips about 43 to 64 degrees to the southwest. This is generally consistent with the regional mapping by Lamar and Dibblee. A few high angle joints and gypsum seams were noted in the borings.

Our subsurface data and interpretations are presented on the accompanying Site Geologic Map (Figure 3) and Geologic Cross Sections (Figures 4 and 5).

The subsurface soils encountered in the exploratory borings drilled at the site are described in greater detail on the logs contained in Appendix A.

2.03 Expansive Soils

Expansion testing performed in accordance with ASTM D4829 indicates that bedrock underlying the site has an expansion classification of low.

Results of expansion test and other soil index tests are presented in Appendix B. Our recommendations to mitigate the effects of expansive soils in foundations design are presented in Section 3.11. Since site grading will redistribute earth materials, potential expansive properties should be verified at the completion of rough grading.

2.04 Surface and Groundwater Conditions

No areas of ponding or standing water were present at the time of our study. Further, no springs or areas of natural seepage were found.

Groundwater was not encountered during our subsurface exploration which extended to a maximum depth of 45 feet.

2.05 Faults

The site is not located within the boundaries of an Earthquake Fault Zone for fault-rupture hazard as defined by the Alquist-Priolo Earthquake Fault Zoning Act and no faults are known to pass through the property.

The nearest fault per Lamar (1970) is the Elysian Park fault which he mapped about one mile to the northeast at its nearest point. Other nearby faults include the Upper Elysian Park Blind Thrust, the Puente Hills Blind Thrust, the Hollywood fault, the Raymond fault and the Verdugo fault which have all be mapped within 10 miles of the site at their nearest points.

The accompanying Regional Fault Map (Figure 6) illustrates the location of the site with respect to major faults in the region. The distance to notable faults within 100 kilometers of the site is presented on Table 1.

2.06 Historic Seismicity

The nearest large historic earthquake in the vicinity of the site was the 1987 Whittier Narrows Earthquake. It was epicentered approximately 8 miles from the site and had a magnitude of 6.0. This event and other strong earthquakes that have occurred in this region in historic time and their approximate epicentral distances are summarized in Table 2.

Seismic design parameters relative to the requirements of the 2016 California Building Code are presented in Section 3.09.

2.07 Flooding Potential

The site is located at the top of a hill and thus is elevated above flood plains.

Control of surface runoff originating from within and outside of the site should, of course, be included in design of the project.

2.08 Landslides

Landslides were not encountered during the current subsurface investigation. Topographic landforms suggestive of landslides were not apparent in the field or on aerial photographs.

Regional geologic and landslide hazard maps do not map any landslides within the site (see Figures 2 and 7).

3.00 CONCLUSIONS AND RECOMMENDATIONS

3.01 General Conclusion

Based on specific data and information contained in this report, our understanding of the project and our general experience in engineering geology and geotechnical engineering, it is our professional judgment that the proposed development is geologically and geotechnically feasible. This is provided that the recommendations presented below are fully implemented during design, grading and construction.

3.02 General Earthwork and Grading

All grading should be performed in accordance with the General Earthwork and Grading Specifications outlined in Appendix C, unless specifically revised or amended below. Recommendations contained in Appendix C are general specifications for typical grading projects and may not be entirely applicable to this project.

It is also recommended that all earthwork and grading be performed in accordance with Appendix J of the 2016 California Building Code and/or applicable local governmental agency requirements. In the event of conflicts between this report and Appendix J, this report shall govern.

Import Soils

If importing of soils is needed to achieve design grades, it is recommended that the soils have a very low expansion potential and very low corrosion potential; otherwise, specific recommendations for foundation design and corrosion mitigation may be needed developed. Any proposed import soils will need to be testing by the geotechnical engineer prior to bringing such soils onto the property.

3.03 Earthwork Shrinkage and Subsidence

Shrinkage is the decrease in volume of soil upon removal and recompaction expressed as a percentage of the original in-place volume. Subsidence occurs as natural ground is densified to receive fill. These factors account for changes in earth volumes that will occur during grading. Since the site is underlain by sedimentary bedrock and minimal grading is anticipated, shrinkage and subsidence is expected to be nominal.

The degree to which fill soils are compacted and variations in the insitu density of existing soils will influence earth volume changes. Consequently, some adjustments in grades near the completion of grading could be required to balance the earthwork.

3.04 Removals and Overexcavation

All vegetation, trash and debris should be cleared from the grading area and removed from the site. Prior to placement of compacted fills, all non-engineered fills and loose, porous, or compressible soils will need to be removed down to competent ground. Removal and requirements will also apply to cut areas, if the depth of cut is not sufficient to reach competent ground.

Because borings indicate bedrock is shallow in the area of the proposed building and potential presence of expansive soils, complete removal of topsoil and fill and replacement with import soil within the building area is recommended. Removal depths are generally expected to be on the order of 2 to 4 feet, but could vary depending on design grades and might be greater next to the offsite descending slope and/or next the tunnel that passing through the site, depending on proposed grades and foundation type used. The exposed soils beneath all removal and overexcavation areas should be scarified an additional 12 inches, moisture conditioned and compacted to a minimum of 90% relative compaction.

We also recommend that floor slabs, concrete flatwork and paved areas shall be underlain by a minimum of 12 inches of soil compacted to a minimum of 90% relative compaction

Removal and overexcavation recommendations will need to be re-evaluated once grading and foundation plans are available particularly in areas next to existing descending slopes and the tunnel that passes through the site.

The above recommendations are based on the assumption that bedrock and soils encountered during field exploration is representative of these materials throughout the site. However, there can be unforeseen and unanticipated variations in soils between points of subsurface exploration. Hence, overexcavation depths must be verified, and

adjusted if necessary, at the time of grading. The overexcavated materials may be moisture-conditioned and re-compacted as engineered fill.

3.05 Rippability and Rock Disposal

Our exploratory borings were advanced without difficulty and did not generate any oversize materials (greater than 12 inches in maximum dimension). Accordingly we expect that all earth materials will be rippable with conventional heavy duty grading equipment and oversized materials are not expected.

If oversized materials are generated during site grading, they should be hauled from the site.

3.06 Subdrains

Since groundwater was not encountered during the course of our investigation and grading of the site will not fill any canyons, installation of canyon subdrains is not expected to be necessary.

3.07 Slopes

Preliminary slope stability calculations indicate that existing slopes will be grossly and surficially stable under static and seismic conditions. Slope stability calculations were performed on cross sections A-A and B-B, however only results for cross section A-A are presented as this cross section presents the steepest and highest slope and lower factors of safety. The slope stability calculations were performed using the Spencer method for circular and non-circular slip surface as implemented by the computer program SLIDE Version 7.032 (Rocscience 2018). Slope stability calculations are presented in Figures 8 and 9 and laboratory testing performed to develop parameters used in the stability calculations are presented in Appendix B. Figure 8 presents the results of the static slope stability analysis and Figure 9 presents the results of the pseudo-static (seismic) analysis. A seismic coefficient equivalent to 1/3 PGA_M was used in the analysis. The factors of safety for the static and pseudo-static analyses were 1.87 and 1.17 respectively. Surficial slope stability using the infinite slope procedure (Duncan et al. 2014) was also evaluated. Factor of safety was 3.20, greater than 1.50. Additional slope stability calculations may need to be preformed to verify these preliminary results once proposed grades and foundation loads are known.

A conceptual development plan available at the time our report was prepared did not show any proposed slopes. If slopes are proposed, they should be inclined no steeper than 2 horizontal to 1 vertical. Slope stability calculations might need to be performed for such slopes. Slope construction methods, such as keying and benching, should be evaluated once grading plans are available.

3.08 Faulting

Since the site is not located within the boundaries of an Earthquake Fault Zone and no faults are known to pass through the property, surface fault rupture within the site is considered unlikely.

3.09 Seismic Design Parameters

The potential damaging effects of regional earthquake activity must be considered in the design of structures.

Mapped seismic design parameters have been developed in accordance with Section 1613 of the 2016 California Building Code (CBC) using the online U.S. Geological Survey Seismic Design Maps Calculator (ASCE 10 Standard), a site location based on latitude and longitude, and site characterization as Site Class C based on our preliminary geotechnical investigation.

The parameters generated for the subject site are presented below:

2016 California Building Code (CBC) Seismic Parameters

Parameter	Value
Site Location	Latitude = 34.0599 degrees Longitude = -118.2106 degrees
Site Class	Site Class = C Soil Profile Name = very dense soil and soft rock
Mapped Spectral Accelerations (Site Class B)	S_s (0.2- second period) = 2.544g S_1 (1-second period) = 0.887g
Site Coefficients (Site Class C)	F_a = 1.000 F_v = 1.300
Maximum Considered Earthquake Spectral Accelerations (Site Class C)	S_{MS} (0.2- second period) = 2.544g S_{M1} (1-second period) = 1.153g
Design Earthquake Spectral Accelerations (Site Class C)	S_{DS} (0.2- second period) = 1.696g S_{D1} (1-second period) = 0.768g

The above table shows that the mapped spectral response acceleration parameter a 1-second period (S_1) ≥ 0.75g. Therefore, for Risk Categories I, II and II the Seismic Design Category is E and for Risk Category IV the Seismic Design Category is F (CBC Table 1604.5 and Section 1613.3.5). Consequently, as required for Seismic Design Categories C through F by CBC Section 1803.5.11, slope instability, liquefaction, total and differential settlement, and surface displacement by faulting or seismically lateral spreading or lateral flow have been evaluated. Applicable portions of CBC Section 1803.5.12 have also been evaluated including dynamic lateral loading of retaining walls.

Peak earthquake ground acceleration adjusted for site class effects (PGA_M) has been determine in accordance with ASCE 7-10 Section 11.8.3 as follows: $PGA_M = F_{PGA} \times PGA = 1.000 \times 0.967 = 0.967g$.

3.10 Liquefaction and Secondary Earthquake Hazards

Potential secondary seismic hazards that can affect land development projects include liquefaction, tsunamis, seiches, seismically induced settlement, seismically induced flooding and seismically induced landsliding.

Liquefaction

Liquefaction is a phenomenon where earthquake- induced ground vibrations increase the pore pressure in saturated, granular soils until it is equal to the confining, overburden pressure. When this occurs, the soil can completely loose its shear strength and enter a liquefied state. The possibility of liquefaction is dependent upon grain size, relative density, confining pressure, saturation of the soils, and intensity and duration of ground shaking. In order for liquefaction to occur, three criteria must be met: underlying loose, coarse-grained (sandy) soils, a groundwater depth of less than about 50 feet, and a potential for seismic shaking from nearby large-magnitude earthquake.

Since the site is underlain by bedrock, liquefaction is not a hazard at the site. It should also be noted that with site is not located with a California Seismic Hazard Zone Map for the liquefaction (Figure 7).

Tsunamis and Seiches

Tsunamis are sea waves that are generated in response to large-magnitude earthquakes. When these waves reach shorelines, they sometimes produce coastal flooding. Seiches are the oscillation of large bodies of standing water, such

as lakes, that can occur in response to ground shaking. Since the site is located at the top of a hill and there are no nearby bodies of standing water, tsunamis and seiches do not pose hazards to the site.

Seismically Induced Settlement

Seismically induced settlement occurs most frequently in areas underlain by loose, granular sediments. Damage as a result of seismically induced settlement is most dramatic when differential settlement occurs in areas with large variations in the thickness of underlying sediments. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement.

Since the site is underlain by bedrock, seismic settlement a factor that needs to be considered in design.

Seismically Induced Flooding

Since the site is located at the top of a hill, it will not be impacted by seismically induced flooding.

Seismically Induced Landsliding

Slope stability calculations performed for this study indicate that the site will not be impacted by seismically induced landsliding. It should be noted that the site is not located within and California Seismic Hazard Zone Map for the seismically induced landsliding (Figure 7).

3.11 Foundations

Isolated spread footings and/or continuous wall footings are recommended to support the proposed structures underlain by compacted fill and/or bedrock. It is possible that a combination of shallow and deep foundations may be needed in the area adjacent to the tunnel that passes through the site, depending upon the location of the tunnel relative to footings and the possible presence of undocumented fill over the tunnel. Because of the limited information provided about the utility tunnel at the time of our study, we cannot discard the possibility that deep foundations or mat foundation may be needed for section of the proposed building adjacent to the tunnel. Additional information about the tunnel will be needed to refine our recommendations. This information may need to include survey data about the depth and location of the tunnel, proposed grades, specific locations of the proposed structures (and footings) relative to the tunnel, records of the original tunnel construction, a site specific utility survey, and possibly additional subsurface exploration. Once more information is available we may need to revise our current foundation recommendations.

If the recommendations in the section on grading are followed and footings are established in compacted fill materials, footings may be designed using the following allowable soil bearing values:

- Continuous Wall Footings:

  Footings having a minimum width of 12 inches and a minimum depth of 12 inches below the lowest adjacent grade have allowable bearing capacity of 3,500 pounds per square foot (psf). This value may be increased by 10% for each additional foot of width and/or depth to a maximum value of 4,000 psf. Reinforcement for potential soil expansion is not needed, but reinforcement for structural design should be decided by the structural engineer.

- Isolated Spread Footings:

  Footings having a minimum width of 24 inches and a minimum depth of 12 inches below the lowest adjacent grade have allowable bearing capacity of 3,500 psf. This value may be increased by 10% for each additional foot of width or depth to a maximum value of 4,500 psf. Reinforcement for potential soil

expansion is not needed, but reinforcement for structural design should be decided by the structural engineer.

- Retaining Wall Footings:

  Footings for retaining walls should be founded a minimum depth of 12 inches and have a minimum width of 12 inches. Footings may be designed using the allowable bearing capacity and lateral resistance values recommended for building footings. However, when calculating passive resistance, the upper 6 inches of the footings should be ignored in areas where the footings will not be covered with concrete flatwork. This value may also be increased by 10% for each additional foot of width or depth to a maximum value of 4,000 psf. Reinforcement should be provided for structural considerations as determined by the design engineer.

- Deep Foundations or Mat Foundations

  As discussed above a section of the proposed building may lay on top of the fill soil right above the utility tunnel. The remarkable difference in compressibility between the fill soil and bedrock may need to be mitigated by using deep foundations (i.e., CIDH piles or minipiles) or mat foundations on the building section on top of the fill. Geotechnical design of these foundations (axial and lateral capacities for deep foundations, or bearing capacity and modulus of subgrade reaction for mats) will be provided once additional information about tunnel and proposed buildings are available.

The above bearing capacities for spread and continuous footings represent an allowable net increase in soil pressure over existing soil pressure and may be increased by one-third for short-term wind or seismic loads. The maximum expected settlement of footings designed with the recommended allowable bearing capacity is expected to be on the order of ½ inch with differential settlement on the order of ¼ inch over a distance of 30 feet. Once potential extent of artificial fill on top of the utility tunnel, if present, is defined, these estimates of settlements may need to be revised.

Due to the preliminary nature of the expansion tests performed for this study, we recommend additional testing be performed near the completion of rough grading to verify the test results and recommended foundation design criteria.

3.12 Foundation Setbacks from Slopes

Setbacks for footings adjacent to slopes should conform to the requirements of the 2016 California Building Code. Specifically, footings should maintain a horizontal distance or setback between any adjacent slope face and the bottom outer edge of the footing.

For slopes descending away from the foundation, the horizontal distance may be calculated by using h/3, where h is the height of the slope. The horizontal setback should not be less than 5 feet, nor need not be greater than 40 feet per the California Building Code. Where structures encroach within the zone of h/3 from the top of the slope the setback may be maintained by deepening the foundations. Flatwork and utilities within the zone of h/3 from the top of slope may be subject to lateral distortion caused by gradual downslope creep. Walls, fences and landscaping improvements constructed at the top of descending slopes should be designed with consideration of the potential for gradual downslope creep.

For ascending slopes, the horizontal setback required may be calculated by using h/2 where h is the height of the slope. The horizontal setback need not be greater than 15 feet per the 2016 California Building Code.

3.13 Slabs on Grade

If building slabs are underlain by soils with a very low expansion potential, we recommend the use of Type I, unreinforced slabs on grade for floors on the subject project. Type I floor slabs have a nominal thickness of 4 inches and are divided into squares or rectangles using weakened plane joints, each with maximum dimensions not exceeding 25 feet. If weakened plane joints are not planned, then the slabs shall be reinforced with 6x6-10/10 welded wire fabric. If slabs are

underlain by expansive soils, reinforcement may be required from a geotechnical perspective.

Special care should be taken on floors slabs to be covered with thin-set tile or other inflexible coverings. These areas may be reinforced with 6x6-10/10 welded wire fabric placed at mid-height of the slab, to mitigate drying shrinkage cracks. Alternatively, inflexible flooring may be installed with unbonded fabric or liners to prevent reflection of slab cracks through the flooring.

A moisture vapor retarder/barrier is recommended beneath all slabs-on-grade that will be covered by moisture-sensitive flooring materials such as vinyl, linoleum, wood, carpet, rubber, rubber-backed carpet, tile, impermeable floor coatings, adhesives, or where moisture-sensitive equipment, products, or environments will exist. We recommend that design and construction of the vapor retarder or barrier conform to Section 1805 of the 2016 California Building Code (CBC) and pertinent sections of American Concrete Institute (ACI) guidance documents 302.1R-04, 302.2R-06 and 360R-10.

The moisture vapor retarder/barrier should consist of a minimum 10 mils thick polyethylene with a maximum perm rating of 0.3 in accordance with ASTM E 1745. Seams in the moisture vapor retarder/barrier should be overlapped no less than 6 inches or in accordance with the manufacturer's recommendations. Joints and penetrations should be sealed with the manufacturer's recommended adhesives, pressure-sensitive tape, or both. The contractor must avoid damaging or puncturing the vapor retarder/barrier and repair any punctures with additional polyethylene properly lapped and sealed.

ACI guidelines allow for the placement of moisture vapor retarder/barriers either directly beneath floor slabs or below an intermediate granular soil layer.

Placing the moisture retarder/barrier directly beneath the floor slab will provide improved curing of the slab bottom and will eliminate potential problems caused by water being trapped in a granular fill layer. Concrete slabs poured directly on a vapor retarder/barrier can experience shrinkage cracking and curling due to differential rates of curing through the thickness of the slab. Therefore, for concrete placed directly on the vapor retarded, we recommend a maximum water cement ratio of 0.45 and the use of water-reducing admixtures to increase workability and decrease bleeding.

If granular soil is placed over the vapor retarder/barrier, we recommend that the layer be at least 2 inches thick in accordance with traditional practice in southern California. Granular fill should consist of clean fine graded materials with 10 to 30% passing the No. 100 sieve and free from clay or silt. The granular layer should be uniformly compacted and trimmed to provide the full design thickness of the proposed slab. The granular fill layer should not be left exposed to rain or other sources of water such as wet-grinding, power washing, pipe leaks or other processes, and should be dry at the time of concrete placement. Granular fill layers that become saturated should be removed and replaced prior to concrete placement.

An additional layer of sand may be placed beneath the vapor retarder/barrier at the developer's discretion to minimize the potential of the retarder/barrier being punctured by underlying soils.

Building slab design recommendations will need to be reviewed and possibly revised once a grading plan is available.

3.14 Miscellaneous Concrete Flatwork

Miscellaneous concrete flatwork and walkways may be designed with a minimum thickness of 4 inches. Large slabs should be reinforced with a minimum of 6x6-10/10 welded wire mesh placed at mid-height in the slab. Control joints should be constructed to create squares or rectangles with a maximum spacing of 15 feet.

Walkways may be constructed without reinforcement. Walkways should be separated from foundations with a thick expansion joint filler. Control joints should be constructed into non-reinforced walkways at a maximum of 5 feet spacing.

The subgrade soils beneath all miscellaneous concrete flatwork should be compacted to a minimum of 90 percent relative compaction for a minimum depth of 12 inches. The geotechnical engineer should monitor the compaction of the subgrade soils and perform testing to verify that proper compaction has been obtained.

3.15 Footing Excavation and Slab Preparations

All footing excavations should be observed by the geotechnical consultant to verify that they have been excavated into competent soils. The foundation excavations should be observed prior to the placement of forms, reinforcement steel, or concrete. These excavations should be evenly trimmed and level. Prior to concrete placement, any loose or soft soils should be removed. Excavated soils should not be placed on slab or footing areas unless properly compacted.

Prior to the placement of the moisture barrier and sand, the subgrade soils underlying the slab should be observed by the geotechnical consultant to verify that all under-slab utility trenches have been properly backfilled and compacted, that no loose or soft soils are present, and that the slab subgrade has been properly compacted to a minimum of 90 percent relative compaction within the upper 12 inches.

Footings may experience and overall loss in bearing capacity or an increased potential to settle where located in close proximity to existing or future utility trenches. Furthermore, stresses imposed by the footings on the utility lines may cause cracking, collapse and/or a loss of serviceability. To reduce this risk, footings should extend below a 1:1 plane projected upward from the closest bottom of the trench.

Slabs on grade and walkways should be brought to a minimum of 2% and a maximum of 6% above their optimum moisture content for a depth of 18 inches prior to the placement of concrete. The geotechnical consultant should perform insitu moisture tests to verify that the appropriate moisture content has been achieved a maximum of 24 hours prior to the placement of concrete or moisture barriers.

3.16 Lateral Load Resistance

Lateral loads may be resisted by soil friction and the passive resistance of the soil. The following parameters are recommended.

- Passive Earth Pressure = 320 pcf (equivalent fluid weight).
- Coefficient of Friction (soil to footing) = 0.34
- Retaining structures should be designed to resist the following lateral active earth pressures:

Surface Slope of Retained Materials (Horizontal:Vertical)	Equivalent Fluid Weight (pcf)
Level	42
5:1	44
4:1	46
3:1	51
2:1	75

These active earth pressures are only applicable if the retained earth is allowed to strain sufficiently to achieve the active state. The required minimum horizontal strain to achieve the active state is approximately 0.0025H. Retaining structures should be designed to resist an at-rest lateral earth pressure if this horizontal strain cannot be achieved.

- At-rest Lateral Earth Pressure = 60 pcf (equivalent fluid weight)

The Mononobe-Okabe method is commonly utilized for determining seismically induced active and passive lateral earth pressures and is based on the limit equilibrium Coulomb theory for static stress conditions. This method entails three fundamental assumptions (e.g., Seed and Whitman, 1970): Wall movement is sufficient to ensure either active or passive conditions, the driving soil wedge inducing the lateral earth pressures is formed by a planar failure surface starting at the heel of the wall and extending to the free surface of the backfill, and the driving soil wedge and the retaining structure act as rigid bodies, and therefore, experiences uniform accelerations throughout the respective bodies (U.S. Army Corps of Engineers, 2003, Engineering and Design - Stability Analysis of Concrete Structures).

- Seismic Lateral Earth Pressure = 23 pcf (equivalent fluid weight).

The seismic lateral earth pressure given above is an inverted triangle, and the resultant of this pressure is an increment of force which should be applied to the back of the wall in the upper 1/3 of the wall height.

Per CBC Section 1803.5.12 dynamic seismic lateral earth pressures shall be applied to foundation walls and retaining walls supporting more than 6 feet of backfill. Dynamic seismic lateral earth pressures may also be applied to shorter walls at the discretion of the structural engineer.

3.17 Drainage and Moisture Proofing

Surface drainage should be directed away from the proposed structure into suitable drainage devices. Neither excess irrigation nor rainwater should be allowed to collect or pond against building foundations or within low-lying or level areas of the lot. Surface waters should be diverted away from the tops of slopes and prevented from draining over the top of slopes and down the slope face.

Walls and portions thereof that retain soil and enclose interior spaces and floors below grade should be waterproofed and dampproofed in accordance with CBC Section 1805.

Retaining structures should be drained to prevent the accumulation of subsurface water behind the walls. Backdrains should be installed behind all retaining walls exceeding 3 feet in height. A typical detail for retaining wall back drains is

presented in Appendix C. All backdrains should be outlet to suitable drainage devices. Retaining wall less than 3 feet in height should be provided with backdrains or weep holes. Dampproofing and/or waterproofing should also be provided on all retaining walls exceeding 3 feet in height.

3.18 Cement Type and Corrosion Potential

Soluble sulfate tests indicate that concrete at the subject site will have a negligible exposure to water-soluble sulfate in the soil. Our recommendations for concrete exposed to sulfate-containing soils are presented in the table below.

Recommendations for Concrete exposed to Sulfate-containing Soils

Sulfate Exposure	Water Soluble Sulfate (SO_4) in Soil (% by Weight)	Sulfate (SO_4) in Water (ppm)	Cement Type (ASTM C150)	Maximum Water-Cement Ratio (by Weight)	Minimum Compressive Strength (psi)
Negligible	0.00 - 0.10	0-150	--	--	2,500
Moderate	0.10 - 0.20	150-1,500	II	0.50	4,000
Severe	0.20 - 2.00	1,500-10,000	V	0.45	4,500
Very Severe	Over 2.00	Over 10,000	V plus pozzolan or slag	0.45	4,500

Use of alternate combinations of cementitious materials may be permitted if the combinations meet design recommendations contained in American Concrete Institute guideline ACI 318-11.

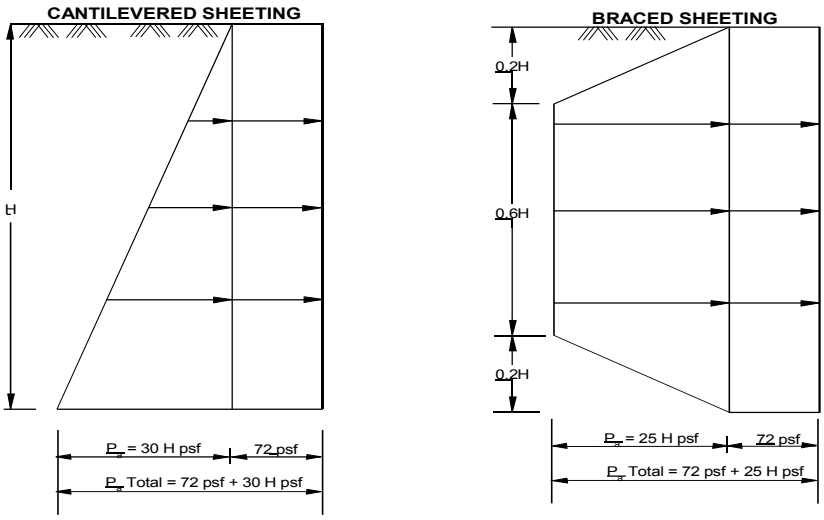
The soils were also tested for soil reactivity (pH), electrical resistivity (ohm-cm) and chloride content. The test results indicate that the on-site soils have a soil reactivity of 7.5, an electrical resistivity of 940 ohm-cm, and a chloride content of 222 ppm. A neutral or non-corrosive soil has a value ranging from 5.5 to 8.4. Generally, soils that could be considered moderately corrosive to ferrous metals have resistivityvalues of about 3,000 ohm-cm to 10,000 ohm-cm. Soils with resistivity values less than 3,000 ohm-cm can be considered corrosive and soils with resistivity values less than 1,000 ohm-cm can be considered extremely corrosive. Soil with a chloride content of 500 ppm or greater are generally considered corrosive.

Based on our analysis, it appears that the underlying onsite soils are extremely corrosive to ferrous metals. Protection of buried pipes utilizing coatings on all underground pipes; clean backfills and a cathodic protection system can be effective in controlling corrosion. Based the preliminary screening tests indicate extremely corrosive soils, a qualified corrosion engineer should be consulted to further assess the corrosive properties of the soil.

3.19 Temporary Slopes

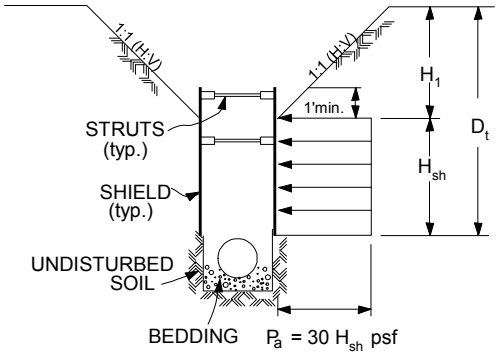
Excavation of utility trenches will require either temporary sloped excavations or shoring. Temporary excavations in existing alluvial soils may be safely made at an inclination of 1:1 or flatter. If vertical sidewalls are required in excavations greater than 5 feet in depth, the use of cantilevered or braced shoring is recommended. Excavations less than 5 feet in depth may be constructed with vertical sidewalls without shoring or shielding. Our recommendations for lateral earth pressures to be used in the design of cantilevered and/or braced shoring are presented below. These values incorporate a uniform lateral pressure of 72 psf to provide for the normal construction loads imposed by vehicles, equipment, materials, and workmen on the surface adjacent to the trench excavation. However, if vehicles, equipment, materials, etc., are kept a minimum distance equal to the height of the excavation away from the edge of

the excavation, this surcharge load need not be applied.



SHORING DESIGN: LATERAL SHORING PRESSURES

Design of the shield struts should be based on a value of 0.65 times the indicated pressure, Pa, for the approximate trench depth. The wales and sheeting can be designed for a value of 2/3 the design strut value.



HEIGHT OF SHIELD, H_{sh} = DEPTH OF TRENCH, D_t , MINUS DEPTH OF SLOPE, H_1

TYPICAL SHORING
DETAIL

Placement of the shield may be made after the excavation is completed or driven down as the material is excavated from inside of the shield. If placed after the excavation, some overexcavation may be required to allow for the shield width and advancement of the shield. The shield may be placed at either the top or the bottom of the pipe zone. Due to the anticipated thinness of the shield walls, removal of the shield after construction should have negligible effects on the load factor of pipes. Shields may be successively placed with conventional trenching equipment.

Vehicles, equipment, materials, etc. should be set back away from the edge of temporary excavations a minimum distance of 15 feet from the top edge of the excavation. Surface waters should be diverted away from temporary excavations and prevented from draining over the top of the excavation and down the slope face. During periods of heavy rain, the slope face should be protected with sandbags to prevent drainage over the edge of the slope, and a visqueen liner placed on the slope face to prevent erosion of the slope face.

Periodic observations of the excavations should be made by the geotechnical consultant to verify that the soil conditions have not varied from those anticipated and to monitor the overall condition of the temporary excavations over time. If at any time during construction conditions are encountered which differ from those anticipated, the geotechnical consultant should be contacted and allowed to analyze the field conditions prior to commencing work within the excavation.

Cal/OSHA construction safety orders should be observed during all underground work.

3.20 Utility Trench Backfill

The onsite fill soils will not be suitable for use as pipe bedding for buried utilities. All pipes should be bedded in a sand, gravel or crushed aggregate imported material complying with the requirements of the Standard Specifications for Public Works Construction Section 306-1.2.1. Bedding materials should be densified to at least 90% relative compaction (ASTM D1557) by mechanical methods. The geotechnical consultant should review and approve of proposed bedding materials prior to use.

The on-site soils are expected to be suitable as trench backfill provided they are screened of organic matter and cobbles over 12 inches in diameter. Trench backfill should be densified to at least 90% relative compaction (ASTM D1557). On-site granular soils may be water densified initially. Supplemental mechanical compaction methods may be required in finer ground soils to attain the required 90% relative compaction.

All utility trench backfill within street right of way, utility easements, under or adjacent to sidewalks, driveways, or building pads should be observed and tested by the geotechnical consultant to verify proper compaction. Trenches excavated adjacent to foundations should not extend within the footing influence zone defined as the area within a line projected at a 1:1 drawn from the bottom edge of the footing. Trenches crossing perpendicular to foundations should be excavated and backfilled prior to the construction of the foundations. The excavations should be backfilled in the presence of the geotechnical engineer and tested to verify adequate compaction beneath the proposed footing.

Cal/OSHA construction safety orders should be observed during all underground work.

3.21 Pavement Sections

Pavement sections should be determined at the completion of rough grading. Representative R-value samples should be obtained and tested toward the completion of rough grading.

An R-value test was performed on anticipated subgrade soil at the site in order to provide information on their soil properties for design of pavement structural sections. Structural sections were designed using the procedures outlined in Chapter 630 of the California Highway Design Manual (Caltrans, 2017). This procedure uses the principle that the pavement structural section must be of adequate thickness to distribute the load from the design traffic index (TI) to the subgrade soils in such a manner that the stresses from the applied loads do not exceed the strength of the soil (R-value).

Development of the design traffic indexes on the basis of a traffic study is beyond the scope of this report; however, our experience indicates that traffic indexes of 5.0 is typical for light auto traffic lanes and 8.0 for heavy trucks. We have provided alternate structural sections for each traffic index. Selection of the final pavement structural section should be based on economic considerations which are beyond the scope of this investigation. Recommended structural sections are as follows:

- Light Auto Traffic Lanes (TI=5.0, R-Value=28):
  4 inches of asphaltic concrete over
  4 inches of crushed aggregate base

- Heavy trucks (TI=8.0, R-Value=28):
  6 inches of asphaltic concrete over
  9 inches of crushed aggregate base

Portland cement concrete (PCC) pavements for areas which are not subject to traffic loads may be designed with a minimum thickness of 4.0 inches of Portland cement concrete on compacted native soils. If traffic loads are anticipated, PCC pavements should be designed for a minimum thickness of 6.0 inches of Portland cement concrete on <4.0> inches of crushed aggregate base.

Prior to paving, the subgrade soils should be scarified and the moisture adjusted to within 2% of the optimum moisture content. The subgrade soils should be compacted to a minimum of 90% relative compaction. All aggregate base courses should be compacted to a minimum of 95% relative compaction.

3.22 Plan Review

Once plans depicting the location (horizontal and depth) of the tunnel crossing the site, specific building locations and proposed grades are developed, we will need to review, refine and possibly revise the recommendations presented in the report. We will also need to review grading and foundation plans when they have been prepared to confirm, refine or revise our recommendations.

3.23 Geotechnical Observation and Testing During Rough Grading

The geotechnical engineer should be contacted to provide observation and testing during the following stages of grading:

- During the clearing and grubbing of the site.
- During the demolition of any existing structures, buried utilities or other existing improvements.
- During excavation and overexcavation of compressible soils.
- During all phases of grading including ground preparation and filling operations.
- When any unusual conditions are encountered during grading.

A final geotechnical report summarizing conditions encountered during grading should be submitted upon completion of the rough grading operations.

3.24 Post-Grading Geotechnical Observation and Testing

After the completion of grading the geotechnical engineer should be contacted to provide additional observation and testing during the following construction activities:

- During trenching and backfilling operations of buried improvements and utilities to verify proper backfill and compaction of the utility trenches.
- After excavation and prior to placement of reinforcing steel or concrete within footing trenches to verify that footings are properly founded in competent materials.
- During fine or precise grading involving the placement of any fills underlying driveways, sidewalks, walkways, or

other miscellaneous concrete flatwork to verify proper placement, mixing and compaction of fills.

- When any unusual conditions are encountered during construction.

3.25 Los Angeles County Building Code Section 111 Statement

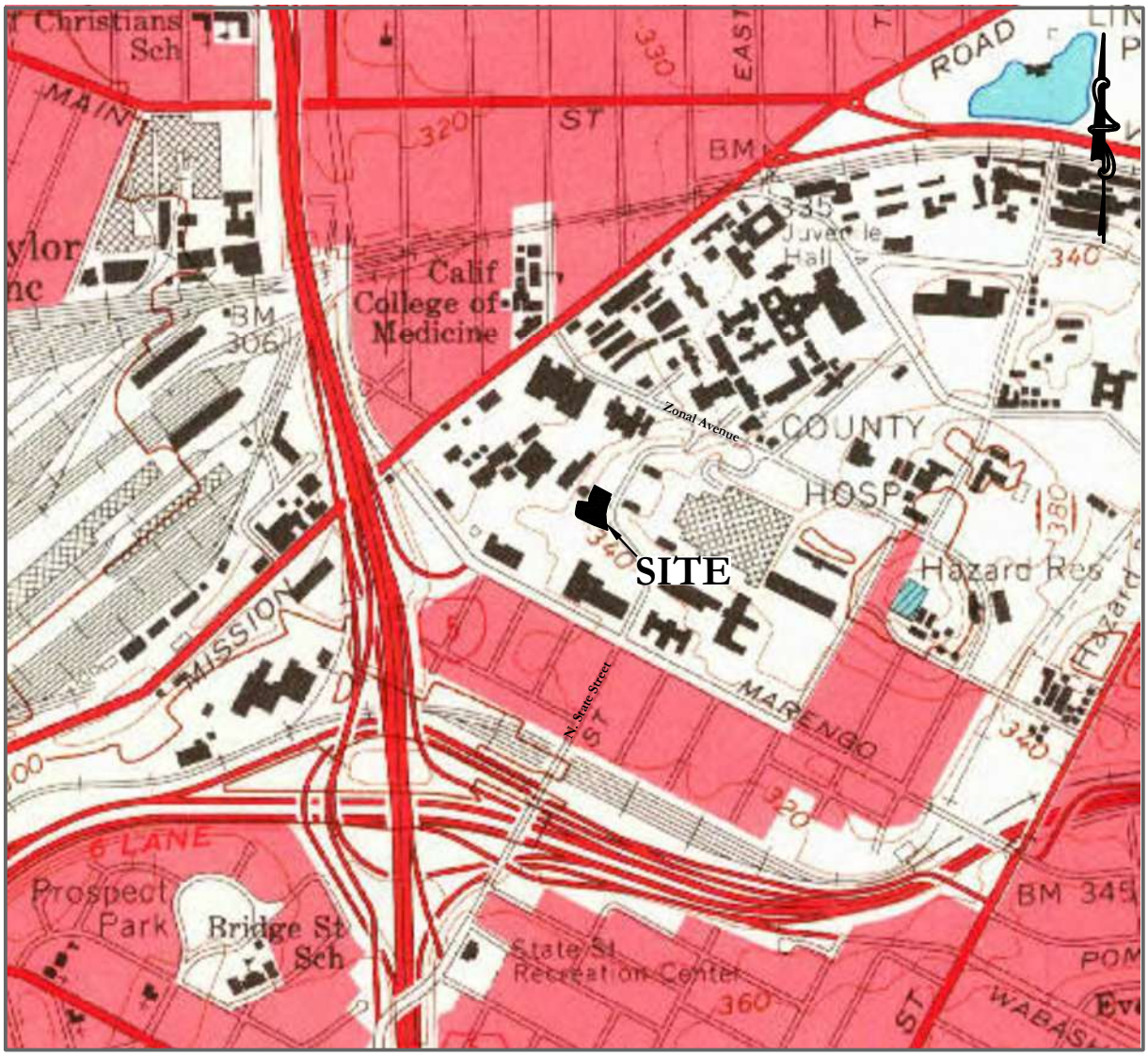
Base in the results of our investigation, it is our opinion that the proposed buildings will not be subject to future landsliding and slippage or settlement beyond that which can be accommodated by design. It is also our opinion that the proposed construction will not adversely affect the stability of properties outside of the building site. These opinions are contingent upon the full implementation of the recommendations presented in the report.

4.00 CLOSURE

The findings, conclusions and recommendations in this report were prepared in accordance with generally accepted engineering and geologic principles and practices. No other warranty, either expressed or implied, is made. This report has been prepared for Los Angeles County Department of Public Works to be used solely for design purposes. Anyone using this report for any other purpose must draw their own conclusions regarding required construction procedures and subsurface conditions.

The geotechnical and geologic consultant should be retained during the earthwork and foundation phases of construction to monitor compliance with the design concepts and recommendations and to provide additional recommendations as needed. Should subsurface conditions be encountered during construction that are different from those described in this report, this office should be notified immediately so that our recommendations may be re-evaluated.

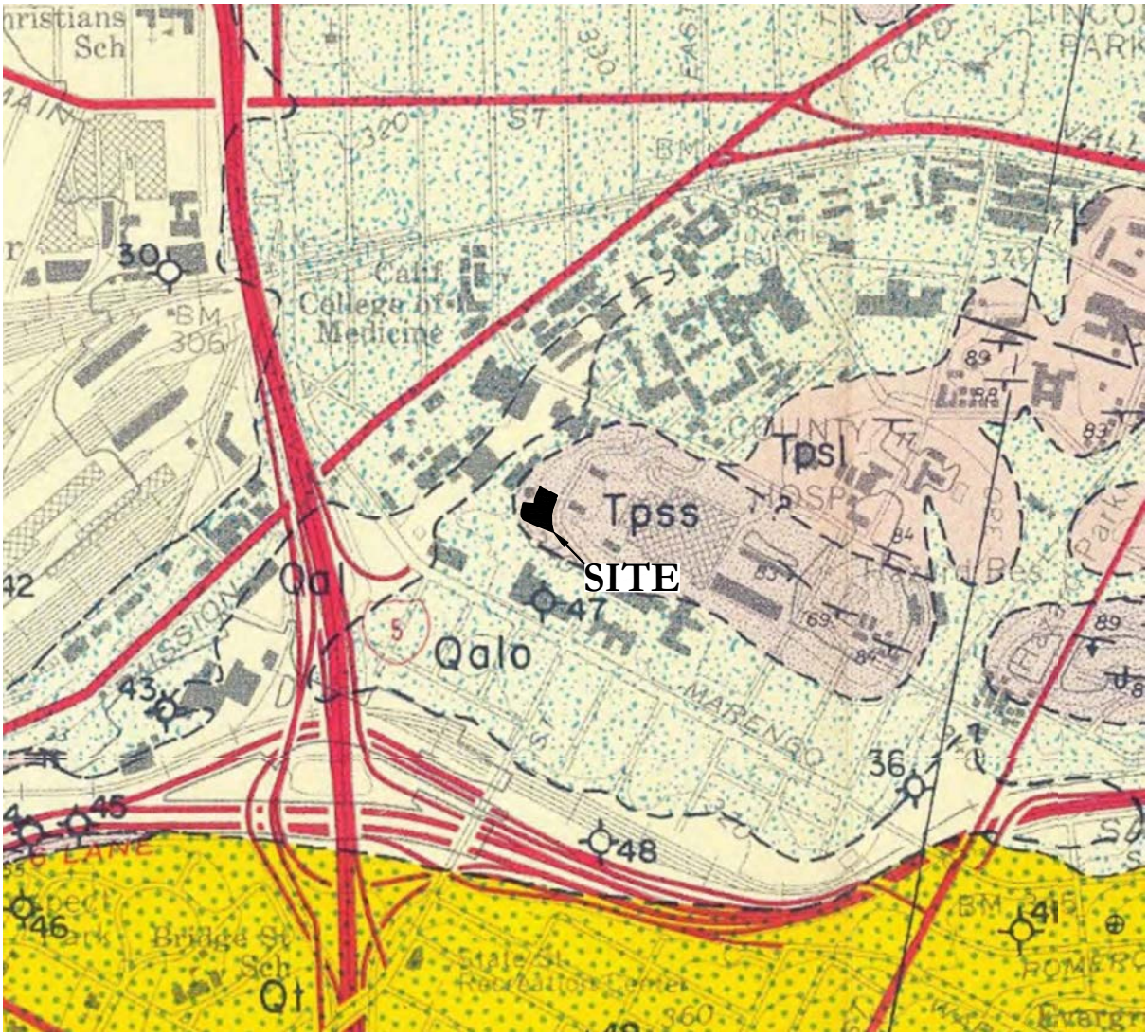
FIGURES AND TABLES



SITE LOCATION MAP

Scale: 1" = 1,000'

Base Map: U.S. Geological Survey, Los Angeles City and County, 1:24000 Map, 1966



REGIONAL GEOLOGIC MAP - Lamar

Scale: 1" = 1000'

Partial Legend

Qal - Alluvium
Qalo - Older Allvium
Qt - Terrace Deposits
Tpsl - Puente Formation (siltstone)
Tpss - Puente Formation (sandstone)

Base Map:
Lamar, Geologic Map of the Elysian Park - repetto hills area, Los Angeles County, California, California Division of Mines and Geology, 1970

RMA Group
Every Project Matters | www.rmacompanies.com

GRAPHIC SCALE
30 0 15 30 60
1" = 30'

B-1					
@3'	N85°W 52°	@20.5'	N71°W 60°	@36.5'	N75°W 54°
@6'	N68°W 44°	@26'	N79°W 62°	@37'	N78°W 64°
@9.5'	N88°W 43°	@28.5'	N78°W 46°	@40.5'	N75°W 51°
@16'	N78°W 56°	@31'	N86°W 50°		
@20'	N2°E 50°	@32.5'	N78°W 55°		

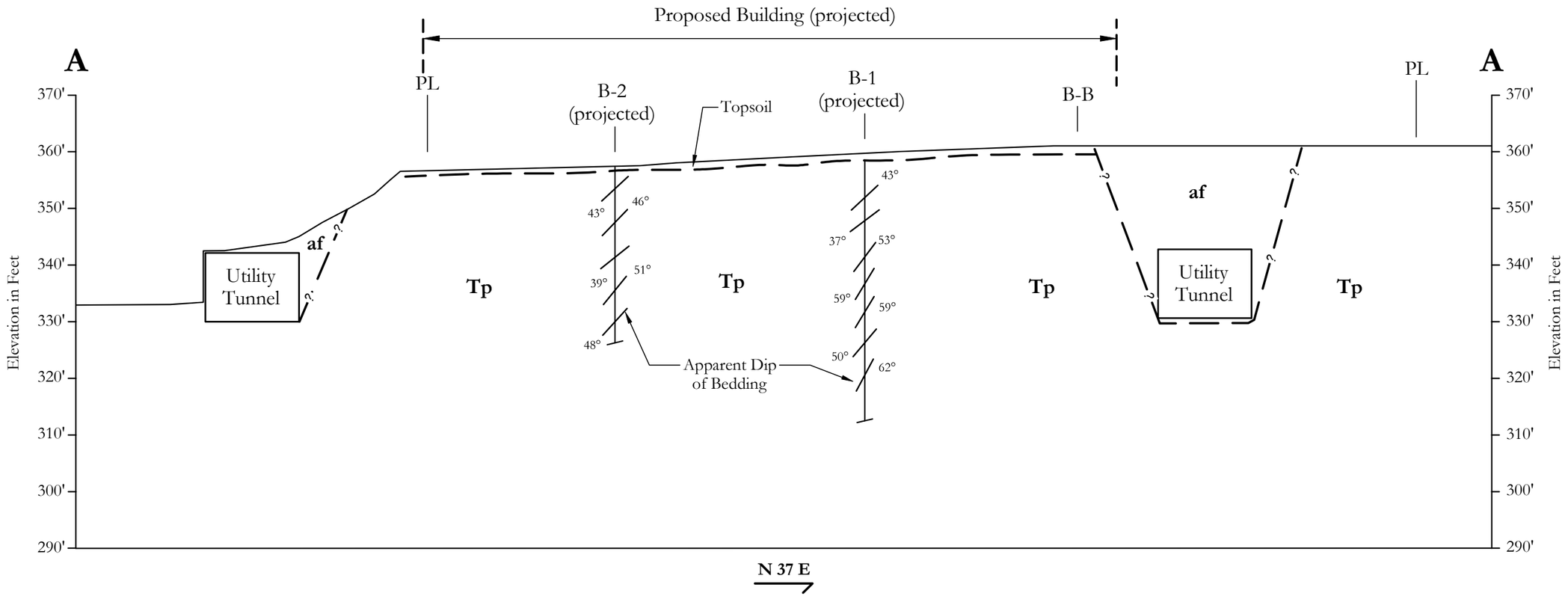
B-2	
@2.5'	N85°W 46°
@4'	64° N10°E
@6.5'	N85°W 50°
@10'	N80°W 49°
@13'	N80°W 44°
@16'	N82°W 43°
@19.5'	N60°W
@19.5'	N70°W 49°
@19.5'	N1°E 80°
@22'	N72°W 53°
@23'	N72°W 53°
@27'	N76°W 50°
@30'	N80°W 54°
@31'	N70°W 51°

GEOLOGIC LEGEND

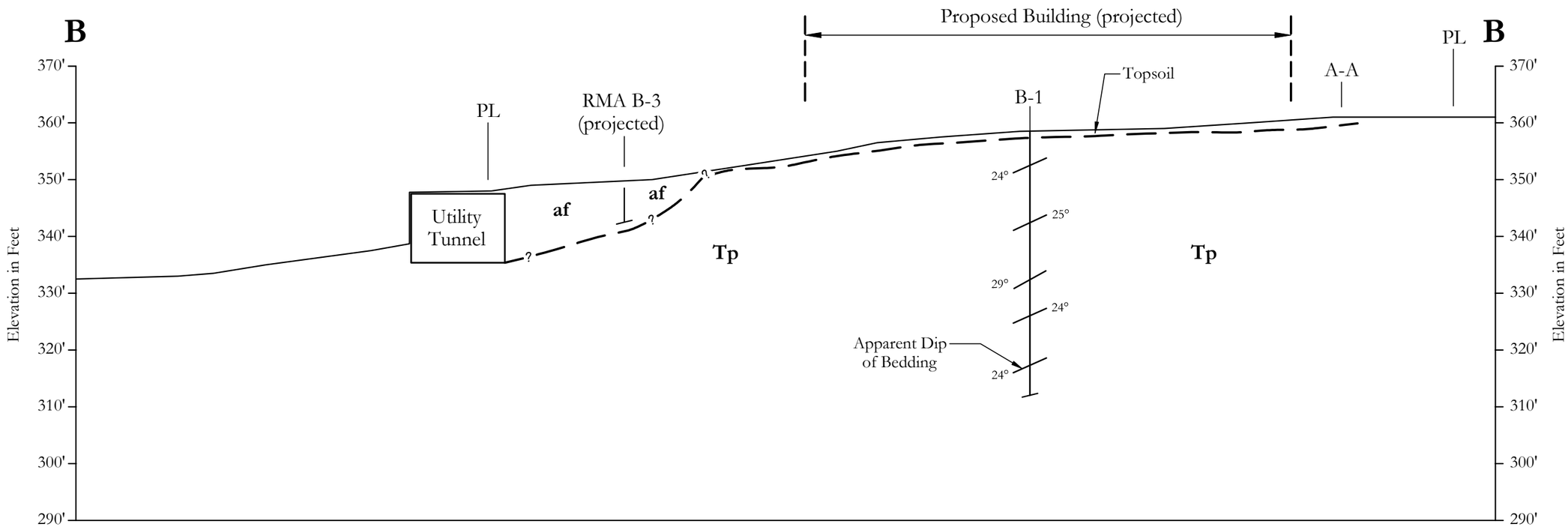
- af - Artificial fill
- Tp - Puente Formation
- 50° - Strike and dip of bedding
- 80° - Joint attitude
- - Vertical joint attitude
- B-3 - Approximate location of exploratory boring
- ? - Approximate geologic contact, queried where uncertain
- B - Geologic cross-section

SITE GEOLOGIC MAP

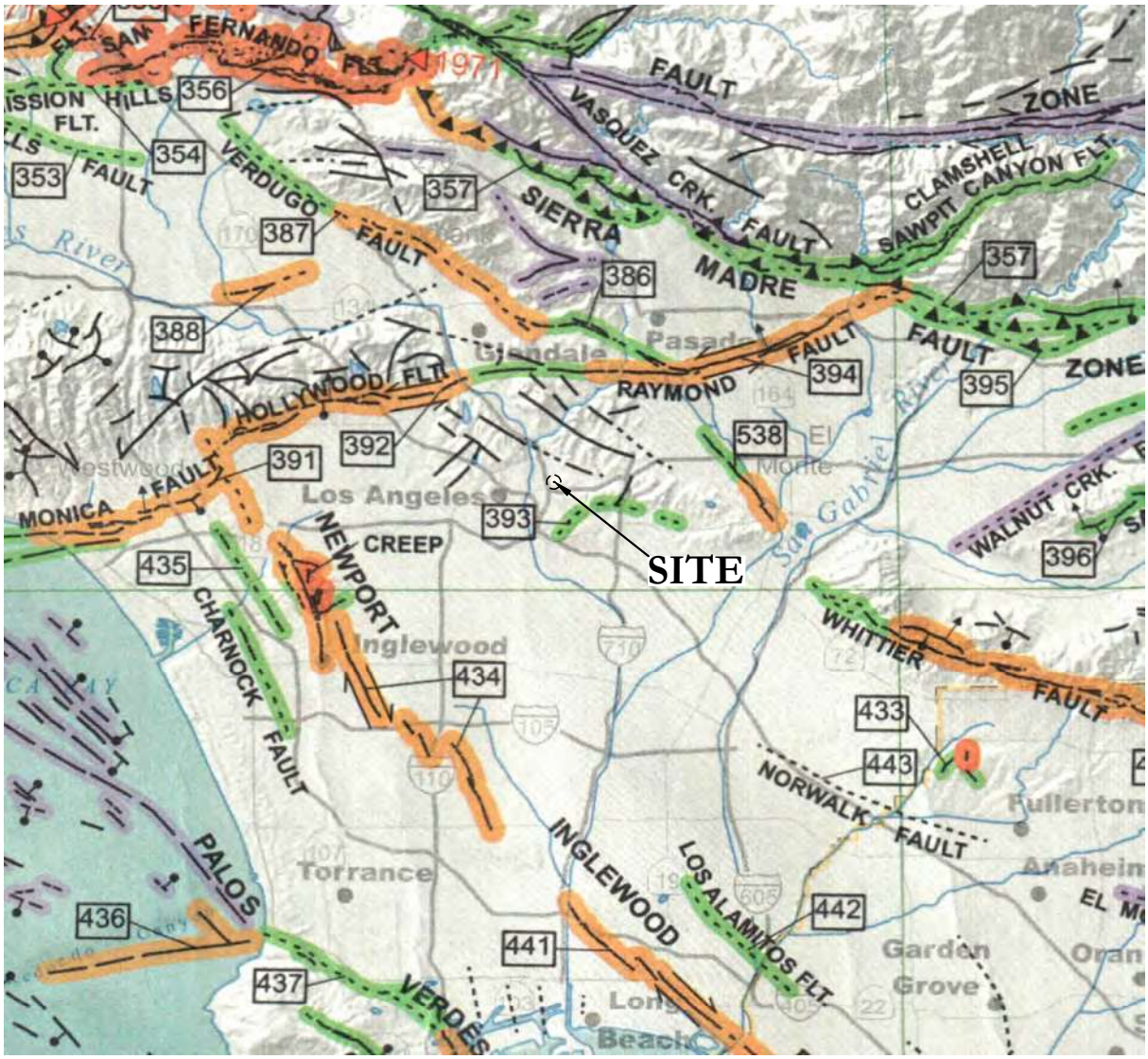
Source: NAC Architecture, undated



GEOLOGIC CROSS SECTION A-A
Scale: 1" = 20', H=V



GEOLOGIC CROSS SECTION B-B
Scale: 1" = 20', H=V



REGIONAL FAULT MAP

Scale: 1" ≈ 7 miles

Partial Legend

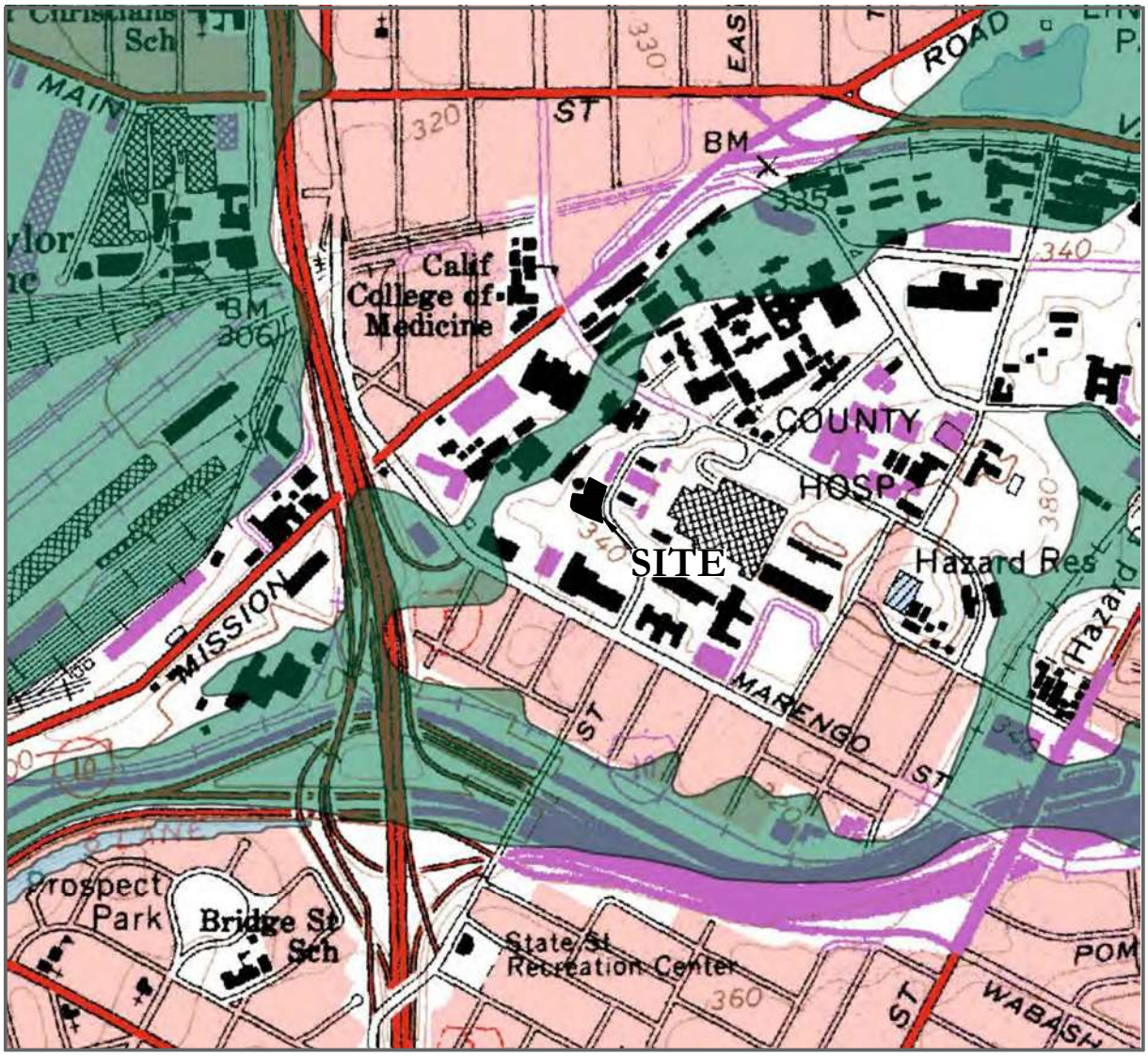
Orange - Holocene fault displacement

Green - Late Quaternary fault displacement

Purple - Quaternary fault

Black - Pre-Quaternary fault

Base Map: California Geological Survey Fault Activity Map of California, 2010



EARTHQUAKE ZONES OF REQUIRED INVESTIGATION

Scale: 1" ≈ 1000'

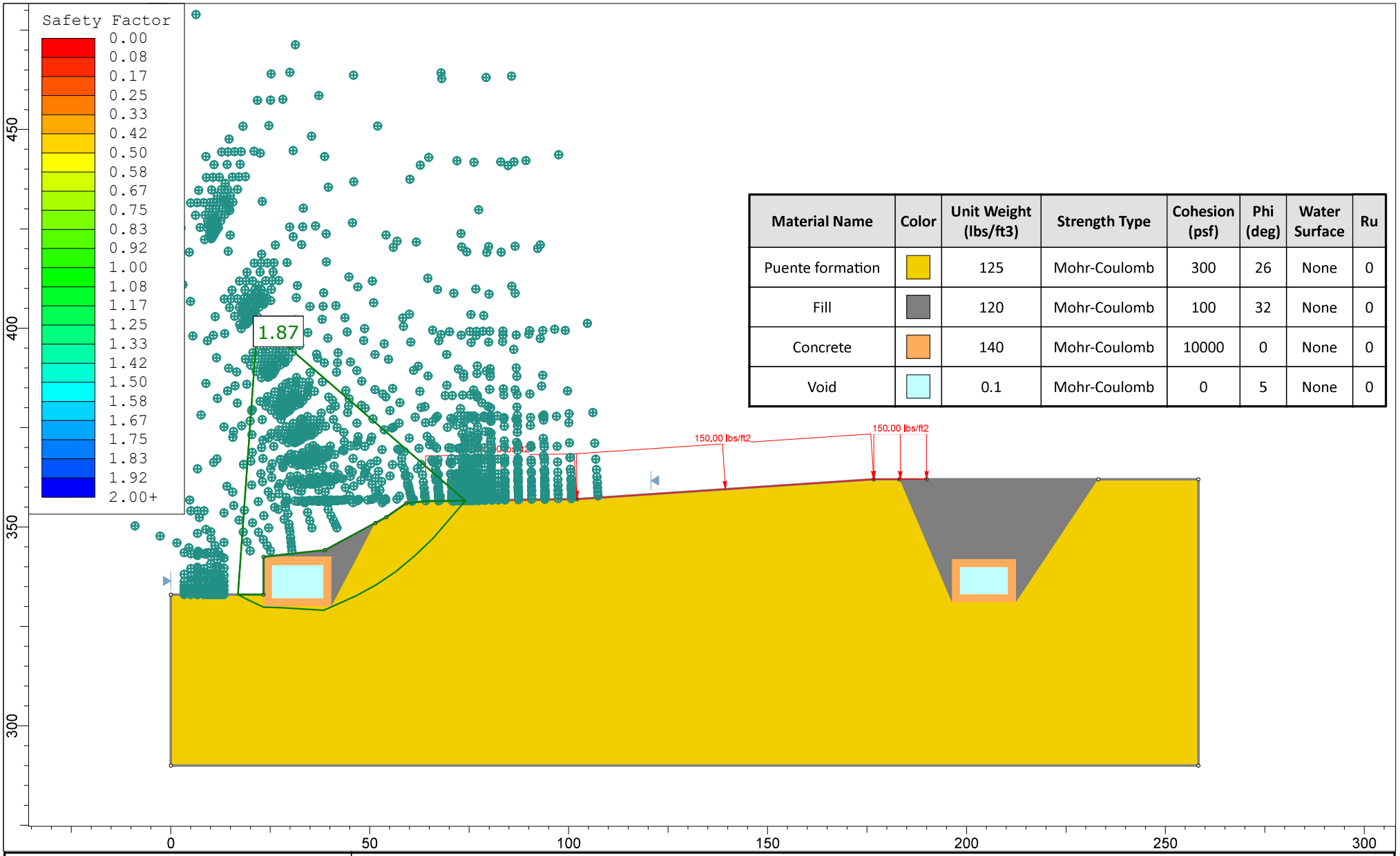
Partial Legend

SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide 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 Resources Code Section 2693(c) would be required.

Base Map: California Geological Survey Earthquake Zones of Required Investigation, Los Angeles Quadrangle, revised 2017

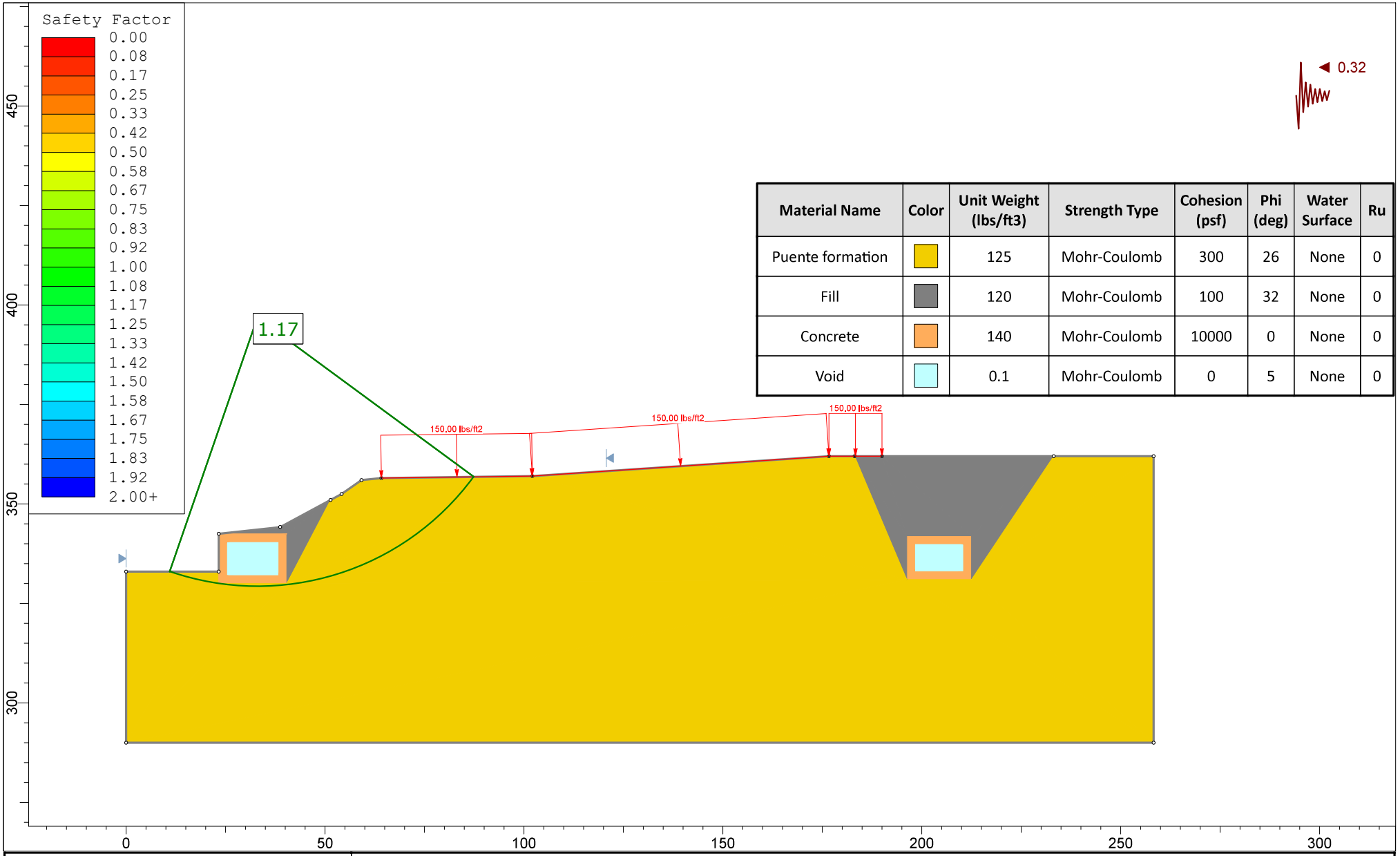


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Puente formation		125	Mohr-Coulomb	300	26	None	0
Fill		120	Mohr-Coulomb	100	32	None	0
Concrete		140	Mohr-Coulomb	10000	0	None	0
Void		0.1	Mohr-Coulomb	0	5	None	0

RMA Companies
Every Project Matters | www.rmacompanies.com

SLIDEINTERPRET 7.032

Project	USC Medical Center Child Care		
Analysis Description	Static slope stability analysis		
Drawn By	JM	Scale 1:400	Company RMA Group, Inc.
Date	3/29/2018, 3:24:32 PM		File Name Cross section AA.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Puente formation		125	Mohr-Coulomb	300	26	None	0
Fill		120	Mohr-Coulomb	100	32	None	0
Concrete		140	Mohr-Coulomb	10000	0	None	0
Void		0.1	Mohr-Coulomb	0	5	None	0

RMA Companies
Every Project Matters | www.rmacompanies.com

SLIDEINTERPRET 7.032

Project	USC Medical Center Child Care		
Analysis Description	Pseudo-static slope stability analysis		
Drawn By	JM	Scale 1:400	Company RMA Group, Inc.
Date	3/29/2018, 3:24:32 PM		File Name Cross section AA.slmd

NOTABLE FAULTS WITHIN 100 KILOMETERS AND SEISMIC DATA

Fault Zone & geometry	Distance (km)	Distance (mi.)	Maximum Moment Magnitude	Slip Rate (mm/yr)
Anacapa-Dume (r-ll-o)	45	28	7.5	3.0
Chino-Central Ave. (rl-r-o)	43	27	6.7	1.0
Clamshell-Sawpit (r)	24	15	6.5	0.5
Cleghorn (ll-ss)	74	46	6.5	3.0
Coronado Bank (rl-ss)	92	57	7.4	3.0
Cucamonga (r)	45	28	6.9	5.0
Elsinore - Glen Ivy (rl-ss)	58	36	6.8	5.0
Upper Elysian Park (r)	0.7	0	6.4	1.3
Hollywood (ll-r-o)	7	4	6.4	1.0
Holser (r)	51	32	6.5	0.4
Malibu Coast (ll-r-o)	30	19	6.7	0.3
Newport-Inglewood (rl-ss)	15	9	6.9	1.5
North Frontal - Western (r)	91	57	7.2	1.0
Northridge (r)	30	19	7	1.5
Oak Ridge - onshore (r)	60	37	7.0	4.0
Palos Verde (rl-ss)	31	19	7.3	3.0
Puente Hills Blind Thrust (r)	5	3	7.1	0.7
Raymond (ll-r-o)	7	4	6.5	1.5
San Andreas (rl-ss)	53	33	7.5	24.0
San Cayetano (r)	66	41	7.0	6.0
San Gabriel (rl-ss)	29	18	7.2	1.0
San Jacinto - San Bernardino (rl-ss)	68	42	6.7	12.0
San Joaquin Hills (r)	48	30	6.6	0.5
San Jose (ll-r-o)	31	19	6.4	0.5
Santa Monica (ll-r-o)	19	12	6.6	1.0
Santa Susana (r)	39	24	6.7	5.0
Santa Ynez (ll-ss)	87	54	7.0	2.0
Sierra Madre (r)	17	11	7.2	2.0
San Fernando (r)	26	16	6.7	2.0
Verdugo (r)	10	6	6.9	0.5
Whittier (rl-ss)	20	12	6.8	2.5

Notes:

Fault geometry - (ss) strike slip, (r) reverse, (n) normal, (rl) right lateral, (ll) left lateral, (o) oblique

Fault and Seismic Data - California Geological Survey (Cao), 2003

HISTORIC STRONG EARTHQUAKES IN SOUTHERN CALIFORNIA SINCE 1812

Date	Event	Causitive Fault	Magnitude	Epicentral Distance (miles)
Dec. 12, 1812	Wrightwood	San Andreas?	7.3	30
Jan. 9, 1857	Fort Tejon	San Andreas	7.9	213
Dec. 16, 1858	San Bernardino Area	uncertain	6.0	54
Feb. 9,1890	San Jacinto	uncertain	6.3	121
May 28, 1892	San Jacinto	uncertain	6.3	121
July 30, 1894	Lytle Creek	uncertain	6.0	40
July 22, 1899	Cajon Pass	uncertain	6.4	45
Dec.25, 1899	San Jacinto	San Jacinto	6.7	74
Sept. 20, 1907	San Bernardino Area	uncertain	5.3	66
May 15, 1910	Elsinore	Elsinore	6.0	54
April 21, 1918	Hemet	San Jacinto	6.8	75
July 23, 1923	San Bernardino	San Jacinto	6.0	54
March 11, 1933	Long Beach	Newport-Inglewood	6.4	28
April 10, 1947	Manix	Manix	6.4	117
Dec. 4, 1948	Desert Hot Springs	San Andreas or Banning	6.5	108
July 21, 1952	Wheeler Ridge	White Wolf	7.3	81
Feb. 9, 1971	San Fernando	San Fernando	6.6	27
July 8, 1986	North Palm Springs	Banning or Garnet Hills	5.6	95
Oct. 1, 1987	Whittier Narrows	Puente Hills Thrust	6.0	8
Feb. 28, 1990	Upland	San Jose	5.5	31
June 28, 1991	Sierra Madre	Clamshell Sawpit	5.8	19
April 22, 1992	Joshua Tree	Eureka Peak	6.1	112
June 28, 1992	Landers	Johnson Valley & others	7.3	105
June 28, 1992	Big Bear	uncertain	6.5	82
Jan. 17, 1994	Northridge	Northridge Thrust	6.7	22
Oct. 16, 1999	Hector Mine	Lavic Lake	7.1	120

Notes:

Earthquake data: U.S. Geological Survey P.P. 1515 & online data, Southern California Earthquake Center
California Geological Survey online data
Magnitudes prior to 1932 are estimated from intensity.
Magnitudes after 1932 are moment, local or surface wave magnitudes.
Attenuation relationship - Boore et al., 1997 (mean values), values at distances > 50 miles are approxima

Site Location:

Site Longitude: - 118.2106
Site Latitude: 34.0599

APPENDIX A

FIELD INVESTIGATION

APPENDIX A

FIELD INVESTIGATION

A-1.00 FIELD EXPLORATION

A-1.01 Number of Borings

Our subsurface investigation consisted of 3 borings drilled with a bucket auger drill rig.

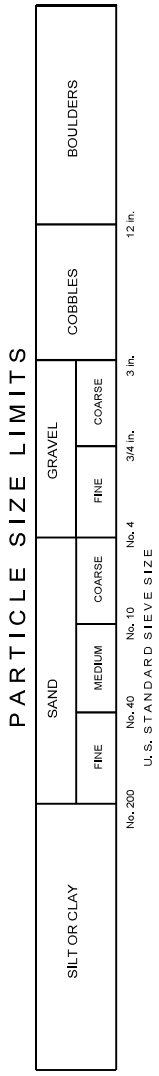
A-1.02 Location of Borings

A Site Geologic Map showing the approximate locations of the borings is presented as Figure 3.

A-1.03 Boring Logging

The borings were surface and downhole logged by an engineering geologist from our staff. Logs of the borings are attached in this appendix. The logs contain factual information and interpretation of subsurface conditions between samples. The strata indicated on these logs represent the approximate boundary between earth units and the transition may be gradual. The logs show subsurface conditions at the dates and locations indicated, and may not be representative of subsurface conditions at other locations and times.

Identification of the soils encountered during the subsurface exploration was made using the field identification procedure of the Unified Soils Classification System (ASTM D2488). A legend indicating the symbols and definitions used in this classification system and a legend defining the terms used in describing the relative compaction, consistency or firmness of the soil are attached in this appendix. Bag samples of the major earth units were obtained for laboratory inspection and testing, and the in-place density of the various strata encountered in the exploration was determined



MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No. 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No. 4 sieve size.	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravel, gravel-sand mixtures. little or no fines.
			GP	Poorly graded gravel or gravel-sand mixtures, little or no fines.
		GRAVELS WITH FINES (Appreciable amt. of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (More than 50% of coarse fraction is SMALLER than the No. 4 sieve size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures.
			SC	Clayey sands, sand-clay mixtures.
FINE GRAINED SOILS (More than 50% of material is SMALLER than No. 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatamaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils.

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

UNIFIED SOIL CLASSIFICATION SYSTEM

I. SOIL STRENGTH/DENSITY

BASED ON STANDARD PENETRATION TESTS

Compactness of sand		Consistency of clay	
Penetration Resistance N (blows/Ft)	Compactness	Penetration Resistance N (blows/ft)	Consistency
0-4	Very Loose	<2	Very Soft
4-10	Loose	2-4	Soft
10-30	Medium Dense	4-8	Medium Stiff
30-50	Dense	8-15	Stiff
>50	Very Dense	15-30	Very Stiff
		>30	Hard

N = Number of blows of 140 lb. weight falling 30 in. to drive 2-in OD sampler 1 ft.

BASED ON RELATIVE COMPACTION

Compactness of sand		Consistency of clay	
% Compaction	Compactness	% Compaction	Consistency
<75	Loose	<80	Soft
75-83	Medium Dense	80-85	Medium Stiff
83-90	Dense	85-90	Stiff
>90	Very Dense	>90	Very Stiff

II. SOIL MOISTURE

Moisture of sands		Moisture of clays	
% Moisture	Description	% Moisture	Description
<5%	Dry	<12%	Dry
5-12%	Moist	12-20%	Moist
>12%	Very Moist	>20%	Very Moist, wet

SOIL DESCRIPTION LEGEND

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-1

Date Drilled:	3-6-18	Drilling Equipment:	Bucket auger
Logged By:	KRD	Boring Hole Diameter:	24"
Location:	See Site Geologic Map	Drive Weights:	0-24' – 2,400 lbs, 25'-44' – 1,550 lbs
Elevation (ft):	356'	Drop:	12"

Depth (ft)	Sample Type	Blows (blows/ft)	Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
						SM		Fill and topsoil: Gray-brown silty fine to medium sand with trace gravel, moist, medium dense, fill 3' thick on east side of boring.
								Puente formation: Gray siltstone with thin interbedded sandstone, BN85W, 52SW
5	R	5		16.8	109.6			@ 6' - Gray silty sandstone @ 6½' - Sandstone bed, BN68W, 44SW
10								@ 9½' - BN88W, 43SW @12' – ½" Thick gypsum layer, BN79W, 45SW
15	R	9		18.3	104.0			@ 16' - BN78W, 56SW @18'-19' – ¼" Thick gypsum seam along bedding
20								@ 20' - Discontinuous gypsum seams, JN2E, 59NW Gypsum Filled Joint @ 20½' - Sandstone, BN71W, 60SW Gray silty sandstone, fine grained, dense, moderately friable @ 26' - Slightly darker gray, BN79W, 62SW
25	R	20		22.8	102.4			@ 28½' - Yellow sandstone, BN78W, 46SW

Sample Types:

R - Ring Sample | Bulk Sample | Groundwater | B – Bedding attitude

T - Tube Sample | S - SPT Sample | End of Boring | J – Joint attitude

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-1

Date Drilled:	3-6-18	Drilling Equipment:	Bucket auger
Logged By:	KRD	Boring Hole Diameter:	24"
Location:	See Site Geologic Map	Drive Weights:	0-24' – 2,400 lbs, 25'-44' – 1,550 lbs
Elevation (ft):	356'	Drop:	12"

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
35	R	30/ 10"		19.5	110.7			@31' – BN86W, 50SW @32½' – Gray sandy siltstone interbeds 4" thick, BN78W, 55SW @34'-35' – Layer of gray sandy siltstone @35½' – BN75W, 54SW @35½'- 36' – Layer of gray sandy siltstone @37' – Gray sandy siltstone layer 6" thick, BN78W, 64SW @38' – Sandy siltstone, thin yellow siltstone interbeds @40½' – Siltstone, BN75W, 51SW
40								
45	R	50/8"		19.7	101.3			Total depth 45.5' No groundwater Hole Backfilled
50								

Material Description

This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.

Sample Types:

R - Ring Sample | - Bulk Sample | - Groundwater

T - Tube Sample | S - SPT Sample | - End of Boring

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-2

Date Drilled:	3-6-18	Drilling Equipment:	Bucket auger
Logged By:	KRD	Boring Hole Diameter:	24"
Location:	See Site Geologic Map	Drive Weights:	0-24' – 2,400 lbs, 25'-44' – 1,550 lbs
Elevation (ft):	355'	Drop:	12"

Depth (ft)	Sample Type	Blows (blows/ft)	Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
						SM		Topsoil: Brown silty sand
								Puente formation (Tp): Yellow sandstone, gray siltstone layers about 1' thick.
								@2½' – Yellow sandstone, calcium carbonite staining on joints, BN85W, 46SW
								@4' – JN10E, 64NW
5								@6½' – Siltstone bed BN85W, 50SW
10	R	11		15.1	105.6			@10' – Siltstone bed BN80W, 49SW
								@11' – Increase in siltstone layers, 3" thick
								@13' – Sandstone, BN80W, 44SW, siltstone below, gray, sandy
15								@16' – Siltstone bed, BN82W, 43SW
								@19½' – Few roots in joints, JN60W vertical, BN70W, 49SW, JN1E, 80SE
20	R	12		19.4	98.2			
								@22' – Sandstone bed, some gypsum, BN72W, 53SW
								@23' – Gypsum layer, BN72W, 53SW
25								@27' – Thin gypsum bed, BN76W, 50SW, ¼" thick
								@27½' – Gypsum bed
								@29' – Gypsum bed

This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.

Sample Types:

R - Ring Sample | - Bulk Sample | - Groundwater | B – Bedding attitude

T - Tube Sample | S - SPT Sample | - End of Boring | J – Joint attitude

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-2

Date Drilled:	3-6-18	Drilling Equipment:	Bucket auger
Logged By:	KRD	Boring Hole Diameter:	24"
Location:	See Site Geologic Map	Drive Weights:	0-24' – 2,400 lbs, 25'-44' – 1,550 lbs
Elevation (ft):	355'	Drop:	12"

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
35	R	10						@30' – BN80W, 54SW @31' – Small roots, gypsum bed BN70W, 51SW Total depth 35' No groundwater Hole Backfilled
40								
45								
50								

This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.

Sample Types:

R - Ring Sample | Bulk Sample - | Groundwater - | T - Tube Sample | S - SPT Sample | End of Boring -

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-3

Date Drilled:	3-6-18	Drilling Equipment:	Bucket auger
Logged By:	KRD	Boring Hole Diameter:	24"
Location:	See Site Geologic Map	Drive Weights:	0-24' – 2,400 lbs, 25'-44' – 1,550 lbs
Elevation (ft):	348'	Drop:	12"

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
0						SM		Artificial fill (af): Brown silty fine to coarse sand with pieces of concrete and brick
5								Water line encountered and broken at 5'. Hole abandoned at 5' due to waterline No groundwater Hole Backfilled
10								
15								
20								
25								

This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.

Sample Types:

R - Ring Sample | Bulk Sample | Groundwater | B – Bedding attitude

T - Tube Sample | S - SPT Sample | End of Boring | J – Joint attitude

APPENDIX B

LABORATORY TESTS

APPENDIX B

LABORATORY TESTS

B-1.00 LABORATORY TESTS

B-1.01 Maximum Density

The Maximum density - optimum moisture relationship for a major soil type encountered during the field exploration was performed in the laboratory using the standard procedures of ASTM D1557.

B-1.02 Expansion Tests

Expansion index testing was performed on representative samples encountered by the test methods outlined in ASTM D4829.

B-1.03 Soluble Sulfates and Chlorides

A test was performed on representative sample encountered during the investigation using the Caltrans Test Methods CTM 417 and CTM 422.

B-1.04 Soil Reactivity (pH) and Electrical Resistivity

A representative soil sample was tested for soil reactivity (pH) and electrical resistivity using California Test Method S3.0 and S5.0. The pH measurement determines the degree of acidity or alkalinity in the soils.

B-1.05 Particle Size Analysis

Particle size analysis was performed on a representative sample in accordance to the standard test methods of the ASTM D422. The hydrometer portion of the standard procedure was not performed and the material retained on the #200 screen was washed.

B-1.06 Direct Shear

Direct shear tests were performed on representative samples of the underlying bedrock using the standard test method of ASTM D3080 (consolidated and drained). Tests were performed on remolded and ring samples. Multi-cyclic of one sample was performed. The remolded sample was tested at 90 percent relative compaction.

Shear tests were performed on a direct shear machine of the strain-controlled type. To simulate possible adverse field conditions, the samples were saturated prior to shearing. Several samples were sheared at varying normal loads and the results plotted to establish the angle of the internal friction and cohesion of the tested samples.

B-1.07 Resistance Value (R-Value)

A Resistance Value test was performed on a representative sample by the test methods outlined in California 301.

B-1.08 Moisture Determination

Moisture content of the bedrock samples was performed in accordance to standard method for determination of water content of soil by drying oven, ASTM D2216. The mass of material remaining after oven drying is used as the mass of the solid particles.

B-1.09 Density of Split-Barrel Samples

Bedrock samples were obtained by using a split-barrel sampler in accordance to standard method of ASTM D1586.

B-1.10 Test Results

Test results for all laboratory tests performed on the subject project are presented in this appendix.

SAMPLE INFORMATION

Sample Number	Sample Description	Sample Location	
		Boring No.	Depth (ft)
1	Gray siltstone	1	1-4
2	Yellow sandstone	2	1-4

MAXIMUM DENSITY - OPTIMUM MOISTURE

Test Method: ASTM D1557

Sample Number	Optimum Moisture (Percent)	Maximum Density (lbs/ft^3)
1	16.3	109.5

EXPANSION TEST

Test Method: ASTM D4829

Sample Number	Molding Moisture Content (Percent)	Final Moisture Content (Percent)	Initial Dry Density (lbs/ft^3)	Expansion Index	Expansion Classification
2	16.5	32.8	89.8	32	Low

SOLUBLE SULFATES AND CHLORIDES

Test Method: CTM 417 and CTM 422

Sample Number	Soluble Sulfate (ppm)	Chlorides (ppm)
1	222	300

SOIL REACTIVITY (pH) AND ELECTRICAL CONDUCTIVITY

Test Method: ASTM D4972

Sample Number	pH	Resistivity (Ohm-cm)
1	7.5	940

PARTICLE SIZE ANALYSIS

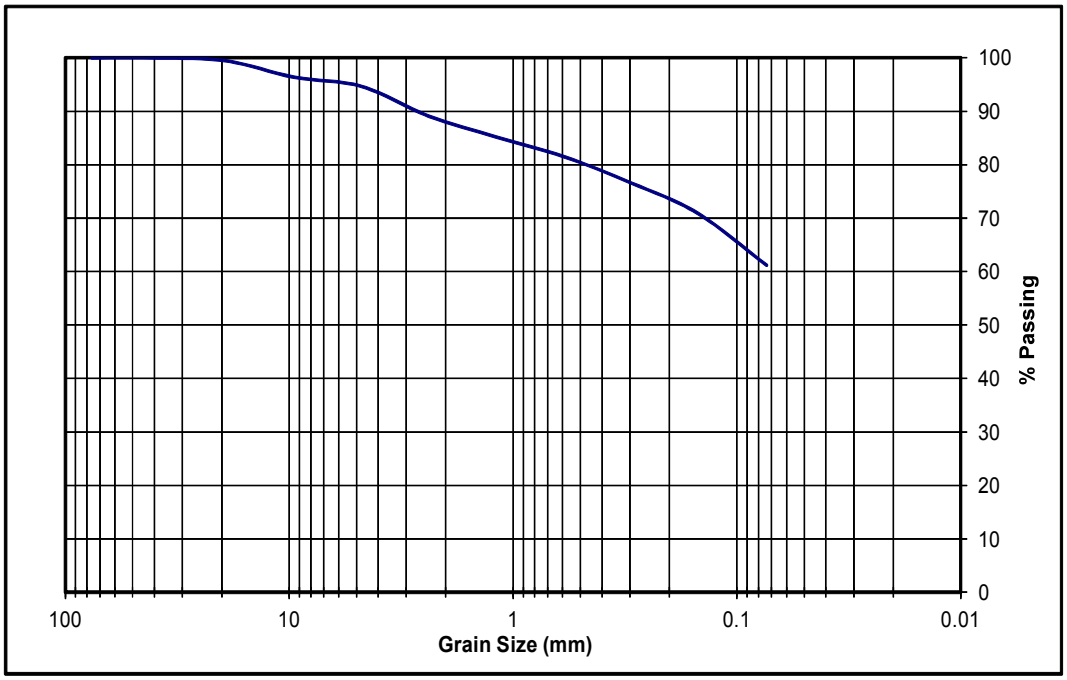
ASTM D422

Sample ID: 1
Location: B-1, 1'-4'

Fraction A: Dry Net Weight (gms): 1,216
Fraction B: Dry Net Weight (gms): 519.5

	Screen Size	Net Retained Weight (gms)	Net Passing Weight (gms)	% Passing
Fraction A:	3"	0	1216	100
	1-1/2"	0	1216	100
	3/4"	7	1209	99
	3/8"	44	1172	96
	#4	65	1151	95

	Screen Size	Net Retained Weight (gms)	Net Passing Weight (gms)	% Passing
Fraction B:	#8	30.5	489.0	89
	#16	51.8	467.7	85
	#30	72.3	447.2	81
	#50	99.1	420.4	77
	#100	130.2	389.3	71
	#200	183.7	335.8	61



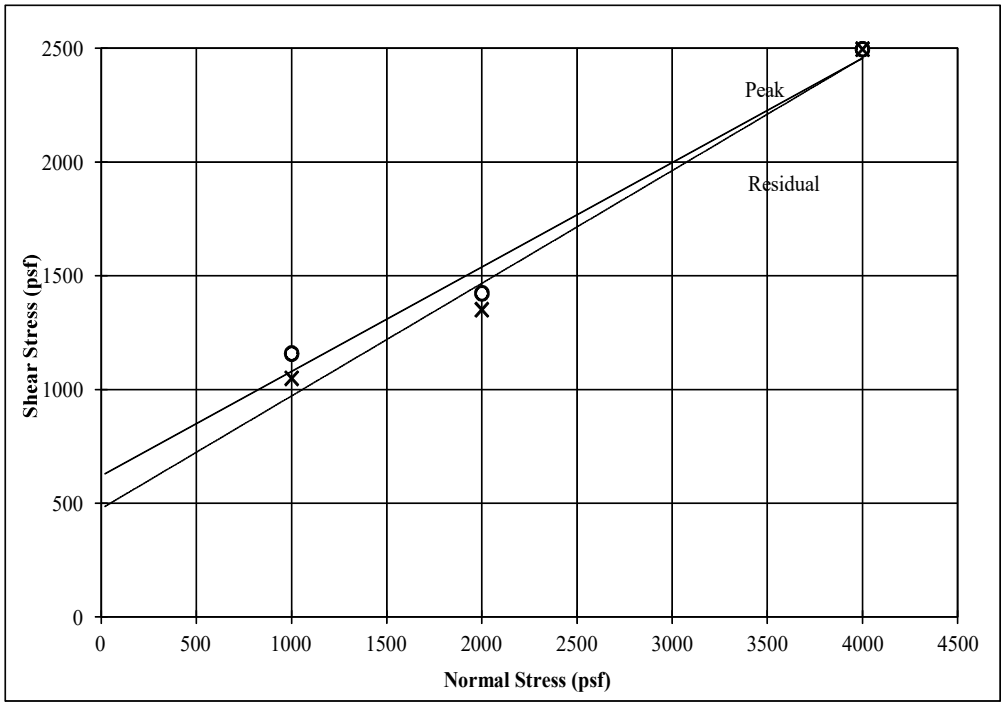
DIRECT SHEAR TEST
ASTM D3080

Sample ID: 1
Location: B-1, 1'-4'

Maximum Dry Density (pcf) =	109.5
Optimum Moisture Content (%) =	16.3
Initial Dry Density (pcf) =	98.6
Initial Moisture Content (%) =	16.3
Final Moisture Content (%) =	26.4

Normal Pressure	Peak Shear Resist	Residual Shear Resist
1000	1158	1049
2000	1423	1351
4000	2496	2496

	Peak	Residual
Cohesion (psf) =	620	480
Friction Angle (deg) =	25	26



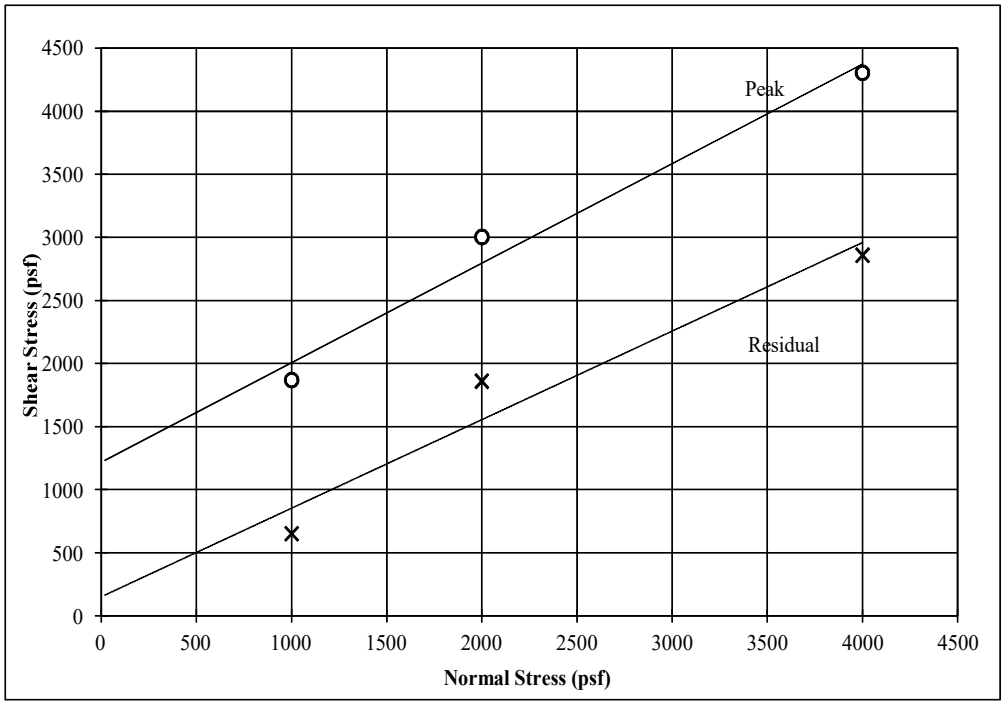
DIRECT SHEAR TEST
ASTM D3080

Location: B-2 @ 20'

Initial Dry Density (pcf) = 98.2
Initial Moisture Content (%) = 19.4
Final Moisture Content (%) = 27.5

Normal Pressure	Peak Shear Resist	Residual Shear Resist
1000	1869	651
2000	3003	1860
4000	4305	2858

	Peak	Residual
Cohesion (psf) =	1220	150
Friction Angle (deg) =	38	35



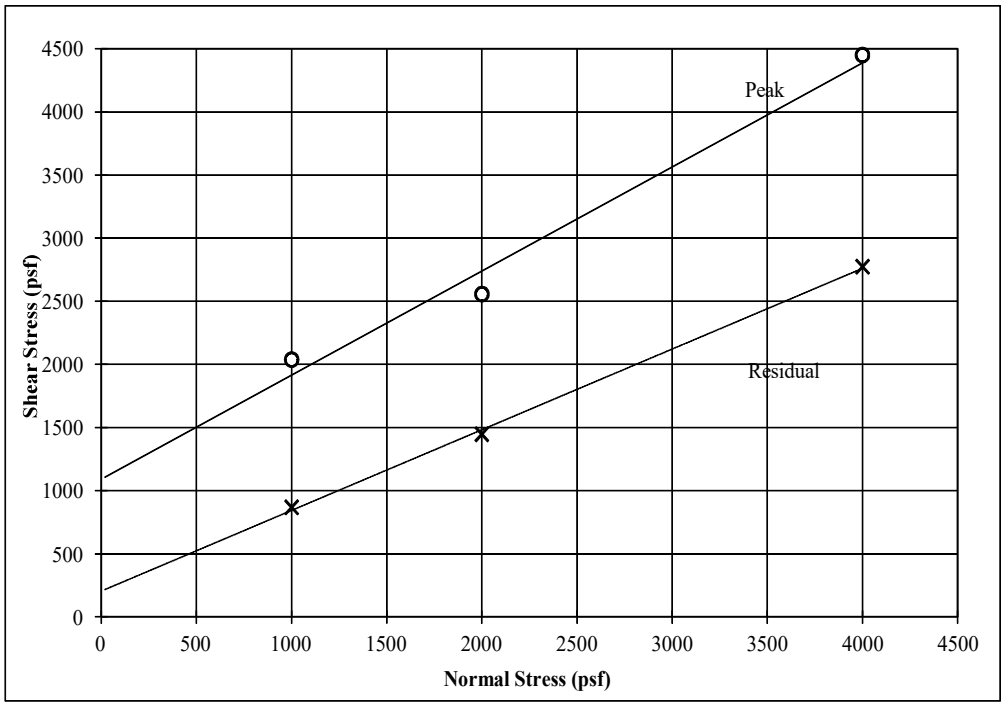
DIRECT SHEAR TEST
ASTM D3080

Run: Initial shear
Location: B-1 @ 15'

Initial Dry Density (pcf) = 104.0
Initial Moisture Content (%) = 18.3
Final Moisture Content (%) = 27.0

Normal Pressure	Peak Shear Resist	Residual Shear Resist
1000	2038	868
2000	2557	1447
4000	4450	2774

	Peak	Residual
Cohesion (psf) =	1090	200
Friction Angle (deg) =	40	33



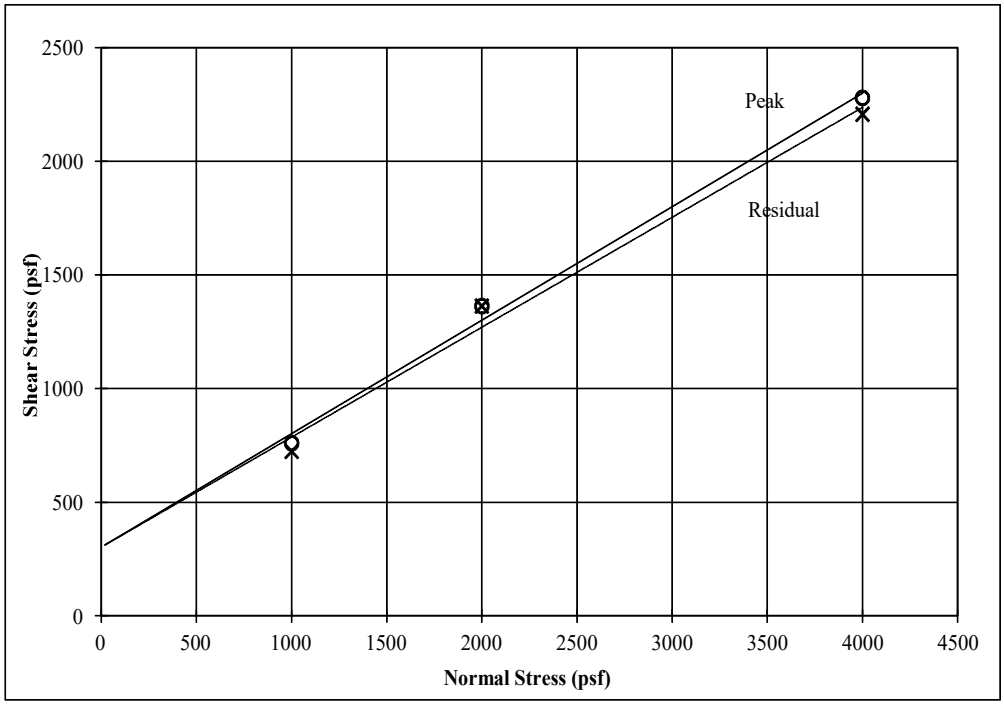
DIRECT SHEAR TEST
ASTM D3080

Run: First reshear
Location: B-1 @ 15'

Initial Dry Density (pcf) = 104.0
Initial Moisture Content (%) = 18.3
Final Moisture Content (%) = 27.0

Normal Pressure	Peak Shear Resist	Residual Shear Resist
1000	760	724
2000	1363	1363
4000	2279	2207

	Peak	Residual
Cohesion (psf) =	300	300
Friction Angle (deg) =	27	26



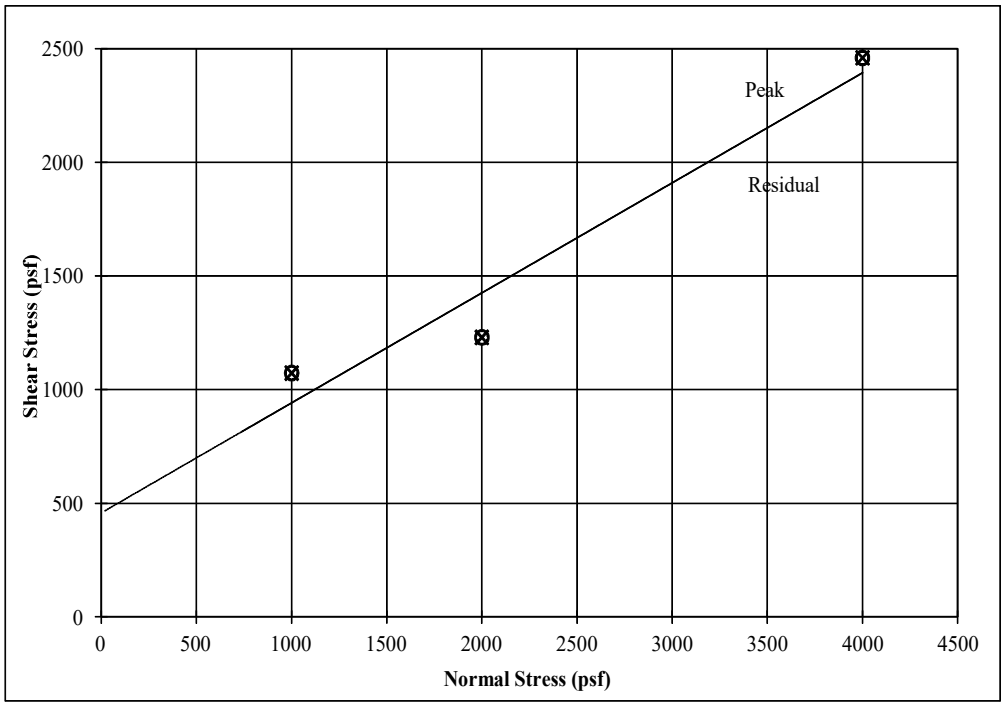
DIRECT SHEAR TEST
ASTM D3080

Run: Scond reshear
Location: B-1 @ 15'

Initial Dry Density (pcf) = 104.0
Initial Moisture Content (%) = 18.3
Final Moisture Content (%) = 27.0

Normal Pressure	Peak Shear Resist	Residual Shear Resist
1000	1073	1073
2000	1230	1230
4000	2460	2460

	Peak	Residual
Cohesion (psf) =	460	460
Friction Angle (deg) =	26	26



CTM 301 - DETERMINATION OF RESISTANCE "R" VALUE OF TREATED AND UNTREATED BASES, SUBBASES, AND BASEMENT SOILS BY THE STABILOMETER

Sample ID: 4

Specimen No	A	B	C
Moisture Content (%)	18.9	18.5	18.2
Dry Density (pcf)	99.6	101.9	103.8
Exudation Pressure (psi)	183	348	693
Stabilometer R Value	11	30	36
Expansion Pressure Dial	17	33	94

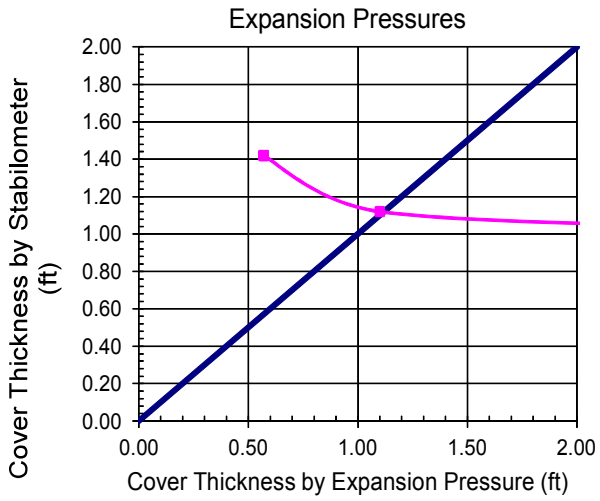
Use: Traffic Index = 5.0 Gravel Factor = 1.00

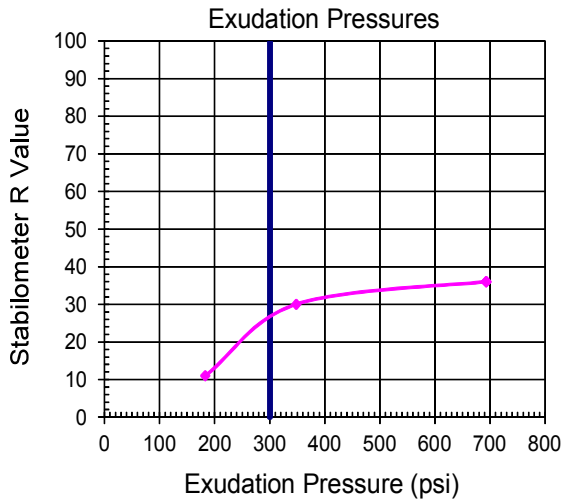
Thickness by Expansion (ft)	0.57	1.10	3.13
Thickness by Stabilometer (ft)	1.42	1.12	1.02

Equilibrium Thick (ft) 1.12

Equilibrium Pressure R Value **30**
Exudation Pressure R Value @ 300 psi **28**

Use Exudation R Value





Expansion Pressure R-Value is based on the following structural section:

Thickness of AC (ft)=	**0.25**	G_f(ac) =	2.50	W(ac) =	145
Thickness of Aggregate Base (ft)=	**0.42**	G_f(base) =	1.10	W(base) =	130
		G_f(avg) =	1.62	W(avg) =	136

APPENDIX C

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

APPENDIX C

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

C-1.00 GENERAL DESCRIPTION

C-1.01 Introduction

These specifications present our general recommendations for earthwork and grading as shown on the approved grading plans for the subject project. These specifications shall cover all clearing and grubbing, removal of existing structures, preparation of land to be filled, filling of the land, spreading, compaction and control of the fill, and all subsidiary work necessary to complete the grading of the filled areas to conform with the lines, grades and slopes as shown on the approved plans.

The recommendations contained in the geotechnical report of which these general specifications are a part of shall supersede the provisions contained hereinafter in case of conflict.

C-1.02 Laboratory Standard and Field Test Methods

The laboratory standard used to establish the maximum density and optimum moisture shall be ASTM D1557.

The insitu density of earth materials (field compaction tests) shall be determined by the sand cone method (ASTM D1556), direct transmission nuclear method (ASTM D6938) or other test methods as considered appropriate by the geotechnical consultant.

Relative compaction is defined, for purposes of these specifications, as the ratio of the in-place density to the maximum density as determined in the previously mentioned laboratory standard.

C-2.00 CLEARING

C-2.01 Surface Clearing

All structures marked for removal, timber, logs, trees, brush and other rubbish shall be removed and disposed of off the site. Any trees to be removed shall be pulled in such a manner so as to remove as much of the root system as possible.

C-2.02 Subsurface Removals

A thorough search should be made for possible underground storage tanks and/or septic tanks and cesspools. If found, tanks should be removed and cesspools pumped dry.

Any concrete irrigation lines shall be crushed in place and all metal underground lines shall be removed from the site.

C-2.03 Backfill of Cavities

All cavities created or exposed during clearing and grubbing operations or by previous use of the site shall be cleared of deleterious material and backfilled with native soils or other materials approved by the soil engineer. Said backfill shall be compacted to a minimum of 90% relative compaction.

C-3.00 ORIGINAL GROUND PREPARATION

C-3.01 Stripping of Vegetation

After the site has been properly cleared, all vegetation and topsoil containing the root systems of former vegetation shall be stripped from areas to be graded. Materials removed in this stripping process may be used as fill in areas designated by the soil engineer, provided the vegetation is mixed with a sufficient amount of soil to assure that no appreciable settlement or other detriment will occur due to decaying of the organic matter. Soil materials containing more than 3% organics shall not be used as structural fill.

C-3.02 Removals of Non-Engineered Fills

Any non-engineered fills encountered during grading shall be completely removed and the underlying ground shall be prepared in accordance to the recommendations for original ground preparation contained in this section. After cleansing of any organic matter the fill material may be used for engineered fill.

C-3.03 Overexcavation of Fill Areas

The existing ground in all areas determined to be satisfactory for the support of fills shall be scarified to a minimum depth of 6 inches. Scarification shall continue until the soils are broken down and free from lumps or clods and until the scarified zone is uniform. The moisture content of the scarified zone shall be adjusted to within 2% of optimum moisture. The scarified zone shall then be uniformly compacted to 90% relative compaction.

Where fill material is to be placed on ground with slopes steeper than 5:1 (H:V) the sloping ground shall be benched. The lowermost bench shall be a minimum of 15 feet wide, shall be a minimum of 2 feet deep, and shall expose firm material as determined by the geotechnical consultant. Other benches shall be excavated to firm material as determined by the geotechnical consultant and shall have a minimum width of 4 feet.

Existing ground that is determined to be unsatisfactory for the support of fills shall be overexcavated in accordance to the recommendations contained in the geotechnical report of which these general specifications are a part.

C-4.00 FILL MATERIALS

C-4.01 General

Materials for the fill shall be free from vegetable matter and other deleterious substances, shall not contain rocks or lumps of a greater dimension than is recommended by the geotechnical consultant, and shall be approved by the geotechnical consultant. Soils of poor gradation, expansion, or strength properties shall be placed in areas designated by the geotechnical consultant or shall be mixed with other soils providing satisfactory fill material.

C-4.02 Oversize Material

Oversize material, rock or other irreducible material with a maximum dimension greater than 12 inches, shall not be placed in fills, unless the location, materials, and disposal methods are specifically approved by the geotechnical consultant. Oversize material shall be placed in such a manner that nesting of oversize material does not occur and in such a manner that the oversize material is completely surrounded by fill material compacted to a minimum of 90% relative compaction. Oversize material shall not be placed within 10 feet of finished grade without the approval of the geotechnical consultant.

C-4.03 Import

Material imported to the site shall conform to the requirements of Section 4.01 of these specifications. Potential import material shall be approved by the geotechnical consultant prior to importation to the subject site.

C-5.00 PLACING AND SPREADING OF FILL

C-5.01 Fill Lifts

The selected fill material shall be placed in nearly horizontal layers which when compacted will not exceed approximately 6 inches in thickness. Thicker lifts may be placed if testing indicates the compaction procedures are such that the required compaction is being achieved and the geotechnical consultant approves their use.
Each layer shall be spread evenly and shall be thoroughly blade mixed during the spreading to insure uniformity of material in each layer.

C-5.02 Fill Moisture

When the moisture content of the fill material is below that recommended by the soils engineer, water shall then be added until he moisture content is as specified to assure thorough bonding during the compacting process.

When the moisture content of the fill material is above that recommended by the soils engineer, the fill material shall be aerated by blading or other satisfactory methods until the moisture content is as specified.

C-5.03 Fill Compaction

After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted to not less than 90% relative compaction. Compaction shall be by sheepsfoot rollers, multiple-wheel pneumatic tired rollers, or other types approved by the soil engineer.

Rolling shall be accomplished while the fill material is at the specified moisture content. Rolling of each layer shall be continuous over its entire area and the roller shall make sufficient trips to insure that the desired density has been obtained.

C-5.04 Fill Slopes

Fill slopes shall be compacted by means of sheepsfoot rollers or other suitable equipment. Compacting of the slopes may be done progressively in increments of 3 to 4 feet in fill height. At the completion of grading, the slope face shall be compacted to a minimum of 90% relative compaction. This may require track rolling or rolling with a grid roller attached to a tractor mounted side-boom.

Slopes may be over filled and cut back in such a manner that the exposed slope faces are compacted to a minimum of 90% relative compaction.

The fill operation shall be continued in six inch (6") compacted layers, or as specified above, until the fill has been brought to the finished slopes and grades as shown on the accepted plans.

C-5.05 Compaction Testing

Field density tests shall be made by the geotechnical consultant of the compaction of each layer of fill. Density tests shall be made at locations selected by the geotechnical consultant.

Frequency of field density tests shall be not less than one test for each 2.0 feet of fill height and at least every one thousand cubic yards of fill. Where fill slopes exceed four feet in height their finished faces shall be tested at a frequency of one test

for each 1000 square feet of slope face.

Where sheepsfoot rollers are used, the soil may be disturbed to a depth of several inches. Density reading shall be taken in the compacted material below the disturbed surface. When these readings indicate that the density of any layer of fill or portion thereof is below the required density, the particular layer or portion shall be reworked until the required density has been obtained.

C-6.00 SUBDRAINS

C-6.01 Subdrain Material

Subdrains shall be constructed of a minimum 4-inch diameter pipe encased in a suitable filter material. The subdrain pipe shall be Schedule 40 Acrylonitrile Butadiene Styrene (ABS) or Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe or approved equivalent. Subdrain pipe shall be installed with perforations down. Filter material shall consist of 3/4" to 1 1/2" clean gravel wrapped in an envelope of filter fabric consisting of Mirafi 140N or approved equivalent.

C-6.02 Subdrain Installation

Subdrain systems, if required, shall be installed in approved ground to conform the approximate alignment and details shown on the plans or herein. The subdrain locations shall not be changed or modified without the approval of the geotechnical consultant. The geotechnical consultant may recommend and direct changes in the subdrain line, grade or material upon approval by the design civil engineer and the appropriate governmental agencies.

C-7.00 EXCAVATIONS

C-7.01 General

Excavations and cut slopes shall be examined by the geotechnical consultant. If determined necessary by the geotechnical consultant, further excavation or overexcavation and refilling of overexcavated areas shall be performed, and/or remedial grading of cut slopes shall be performed.

C-7.02 Fill-Over-Cut Slopes

Where fill-over-cut slopes are to be graded the cut portion of the slope shall be made and approved by the geotechnical consultant prior to placement of materials for construction of the fill portion of the slope.

C-8.00 TRENCH BACKFILL

C-.01 General

Trench backfill within street right of ways shall be compacted to 90% relative compaction as determined by the ASTM D1557 test method. Backfill may be jetted as a means of initial compaction; however, mechanical compaction will be required to obtain the required percentage of relative compaction. If trenches are jetted, there must be a suitable delay for drainage of excess water before mechanical compaction is applied.

C-9.00 SEASONAL LIMITS

C-9.01 General

No fill material shall be placed, spread or rolled while it is frozen or thawing or during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall not be resumed until field tests by the soils engineer indicate that the moisture content and density of the fill are as previously specified.

C-10.00 SUPERVISION

C-10.01 Prior to Grading

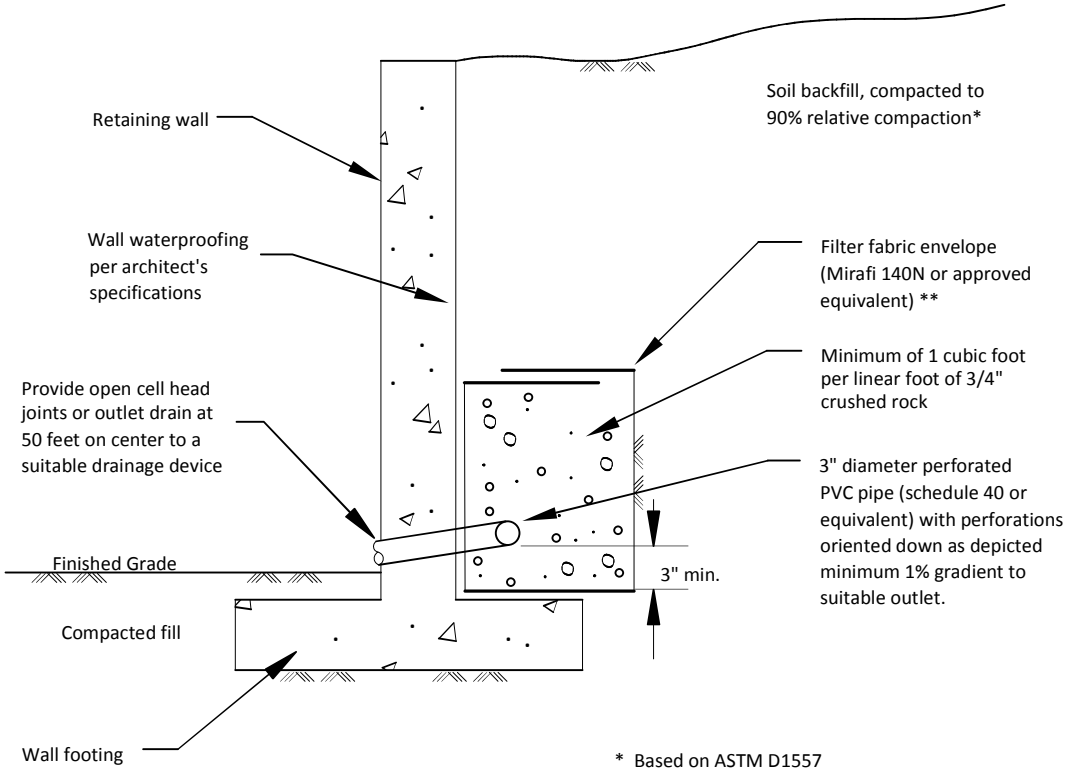
The site shall be observed by the geotechnical consultant upon completion of clearing and grubbing, prior to the preparation of any original ground for preparation of fill.

The supervisor of the grading contractor and the field representative of the geotechnical consultant shall have a meeting and discuss the geotechnical aspects of the earthwork prior to commencement of grading.

C-10.02 During Grading

Site preparation of all areas to receive fill shall be tested and approved by the geotechnical consultant prior to the placement of any fill.

The geotechnical consultant or his representative shall observe the fill and compaction operations so that he can provide an opinion regarding the conformance of the work to the recommendations contained in this report.



* Based on ASTM D1557

** If class 2 permeable material (See gradation to left) is used in place of 3/4" - 1 1/2" gravel. Filter fabric may be deleted. Class 2 permeable material compacted to 90% relative compaction. *

SPECIFICATIONS FOR CLASS 2 PERMEABLE MATERIAL (CAL TRANS SPECIFICATIONS)

Sieve Size	% Passing
1"	100
3/4"	90-100
3/8"	40-100
No.4	25-40
No.8	18-33
No.30	5-15
No.50	0-7
No.200	0-3

RETAINING WALL DRAINAGE DETAIL

APPENDIX D

REFERENCES

APPENDIX D

REFERENCES

1. California Building Standards Commission, 2016 California Building Code.

2. California Division of Mines and Geology, 1998, Seismic Hazard Evaluation of the Los Angeles 7.5-minute quadrangle, Los Angeles County, California: California Division of Mines and Geology, Open-File Report 98-20.

3. California Geological Survey, Earthquake Zones of Required Investigation, Los Angeles Quadrangle, 2017 Earthquake Fault Zones and 199 Seismic Hazard Zones.

4. Cao, Y. and others, 2003, The Revised 2002 California Probabilistic Seismic Hazard Maps, June 2003.

5. Dibblee, T.W., and Ehrenspeck, H.E., ed., 1989, Geologic map of the Los Angeles quadrangle, Los Angeles County, California: Dibblee Geological Foundation, Dibblee Foundation Map DF-22.

6. Duncan, J.M., Wright, S.G., and Brandon, T.L. 2014, Soil Strength and Slope Stability, John Wiley & Sons, Inc

7. .

8. Google Earth, Aerial Photographs, 27-2011, 2009-2005, 2003 and 1994.

9. Historicaerials.com, Aerial Photographs, 2010, 2009, 2005, 2004, 1994, 1980, 192, 1964, 1952 and 1948.

10. Jennings, C.W., and Bryant, W.A, 2010, Fault Activity Map of California, California Geological Survey, Geologic Data Map No. 6.

11. Lamar, D.L., 1970, Geology of the Elysian Park-Repetto Hills area, Los Angeles County, California: California Division of Mines and Geology Special Report 101.

12. Los Angeles County Department of Public Works, 2013, Manual for Preparation of Geotechnical Reports,

13. Martin, G.R. and Lew, M., 1999, Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction Hazards in California, Southern California Earthquake Center publication.

14. Rocscience (2018), SLIDE version 7.032, Rocscience Inc., Toronto, Canada

15. Seed, H.B. and Whitman, R.V., 1970, Design of Earth Structures for Dynamic Loads *in* American Society of Civil Engineers Specialty Conference State-of-the Art Paper, Lateral Stresses in the Ground and Design of Earth-Retaining Structures.

16. Tokimatsu, K. and Seed, H.B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Soil Mechanics and Foundation Engineering, Vol. 113, No. 8.

17. U.S. Army Corps of Engineers, 2003, Engineering and Design - Stability Analysis of Concrete Structures, Publication CECW-E, Circular No. 1110-2-6058, Appendix G, http://www.usace.army.mil/publications/eng-circulars/ec1110-2-6058/

18. U.S. Geological Survey, 2018, Seismic Design Maps Calculator, ASCE 7-10 Standard, htpp://earthquake.usgs.gov/designmaps/us/report.

19. Yerkes, R.F., 1997, Preliminary geologic map of the Los Angeles 7.5' quadrangle, southern California: U.S. Geological Survey, Open-File Report OF-97-254.

APPENDIX F.2

GEOTECHNICAL INVESTIGATION, PROPOSED HEALTH AGENCY RECUPERATIVE CAMPUS, 1774 ZONAL AVENUE, LOS ANGELES, CALIFORNIA

GEOTECHNICAL INVESTIGATION

PROPOSED HEALTH AGENCY RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

TRACT: 2082, LOT: LT1

GEOCON
WEST, INC.
GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR
KFA, LP
SANTA MONICA, CALIFORNIA

PROJECT NO. A9733-06-01

APRIL 9, 2018

Project No. A9733-06-01
April 9, 2018

Mr. Joe Killefer
KFA, LP
1625 Olympic Boulevard
Santa Monica, California 90404

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED HEALTH AGENCY RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA
TRACT: TR 2082 LOT: LT 1

Dear Mr. Killefer:

In accordance with your authorization of our proposal dated December 7, 2017, we have performed a geotechnical investigation for the proposed health agency recuperative campus located at 1774 Zonal Avenue in the City of Los Angeles, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned.

Very truly yours,

GEOCON WEST, INC.

REGISTERED PROFESSIONAL ENGINEER PETRINA ZEN C87489 CIVIL STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER H. HARRY DERKALOUSDIAN C 79694 CIVIL STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST SUSAN FRANZEN KIRKGARD No. 1754 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

Petrina Zen	Harry Derkalousdian	Susan F. Kirkgard
PE 87489	PE 79694	CEG 1754

(EMAIL) Addressee

TABLE OF CONTENTS

1. PURPOSE AND SCOPE 1
2. SITE AND PROJECT DESCRIPTION 1
3. GEOLOGIC SETTING 2
4. SOIL AND GEOLOGIC CONDITIONS 2
   - 4.1 Artificial Fill 2
   - 4.2 Alluvium 3
   - 4.3 Puente Formation 3
5. GROUNDWATER 3
6. GEOLOGIC HAZARDS 3
   - 6.1 Surface Fault Rupture 3
   - 6.2 Seismicity 5
   - 6.3 Seismic Design Criteria 5
   - 6.4 Liquefaction Potential 7
   - 6.5 Slope Stability 8
   - 6.6 Earthquake-Induced Flooding 9
   - 6.7 Tsunamis, Seiches, and Flooding 9
   - 6.8 Oil Fields & Methane Potential 9
   - 6.9 Subsidence 10
7. CONCLUSIONS AND RECOMMENDATIONS 11
   - 7.1 General 11
   - 7.2 Soil and Excavation Characteristics 13
   - 7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate 13
   - 7.4 Grading 14
   - 7.5 Controlled Low Strength Material (CLSM) 17
   - 7.6 Shrinkage 18
   - 7.7 Foundation Design 18
   - 7.8 Foundation Settlement 19
   - 7.9 Miscellaneous Foundations 20
   - 7.10 Lateral Design 20
   - 7.11 Concrete Slabs-on-Grade 20
   - 7.12 Preliminary Pavement Recommendations 22
   - 7.13 Retaining Walls Design 23
   - 7.14 Retaining Wall Drainage 24
   - 7.15 Temporary Excavations 25
   - 7.16 Surface Drainage 26
   - 7.17 Plan Review 26

LIMITATIONS AND UNIFORMITY OF CONDITIONS

LIST OF REFERENCES

MAPS, TABLES, AND ILLUSTRATIONS
- Figure 1, Vicinity Map
- Figure 2, Site Plan
- Figure 3, Regional Fault Map
- Figure 4, Regional Seismicity Map
- Figures 5 and 6, DE Empirical Estimation of Liquefaction Potential
- Figures 7 and 8, MCE Empirical Estimation of Liquefaction Potential
- Figures 9 and 10, Retaining Wall Drain Detail

TABLE OF CONTENTS (Continued)

APPENDIX A
FIELD INVESTIGATION
Figures A1 through A5, Boring Logs

APPENDIX B
LABORATORY TESTING
Figures B1 and B2, Direct Shear Test Results
Figures B3 and B4, Consolidation Test Results
Figure B5, Atterberg Limits Test Results
Figure B6, Grain Size Analysis Test Results
Figure B7, Laboratory Test Results
Figure B8, Corrosivity Test Results

GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed health agency recuperative campus located at 1774 Zonal Avenue in the City of Los Angeles, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on March 6, 2018, by excavating five 8-inch diameter borings to depths of approximately 15½ and 56 feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 1774 Zonal Avenue in the City of Los Angeles, California. The site is a rectangular-shaped parcel bounded by Zonal Avenue to the north, by a paved surface parking lot to the east, by Women's and Children's Hospital campus to the west, and by a paved surface parking lot for a commercial development to the south. An approximately 40-foot-high, 2:1 (H:V gradient) graded slope ascends from the southeast corner of the site to the entrance drive for the University of Southern California Health Sciences Campus located east-southeast of the site. The site is predominantly vacant with the exception of a temporary trailer structure located in the southwest portion of the parcel. Topography at the site is relatively level, with no pronounced highs or lows. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. Vegetation onsite consists of native grass and weeds in localized areas.

Based on the information provided by the Client, it is our understanding that the proposed development will consist of a three-story recuperative center structure, and four two-story crisis residential treatment center structures (see Site Plan, Figure 2). All structures will utilize shipping containers as the primary structures and will be constructed at or near present grade.

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structures will be up to 300 kips, and wall loads will be up to 4 kips per linear foot.

Once the design phase and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Any changes in the design, location or elevation of any structure, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

3. GEOLOGIC SETTING

The site is located in the northeastern portion of the Los Angeles Basin, along the southwestern flank of the Repetto Hills. The Los Angeles Basins is a coastal plain bounded by the Santa Monica Mountains on the north, the Elysian Hills and Repetto Hills on the northeast, the Puente Hills and Whittier Fault on the east, the Palos Verdes Peninsula and Pacific Ocean on the west and south, and the Santa Ana Mountains and San Joaquin Hills on the southeast. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits underlain by a basement complex of igneous and metamorphic composition. Regionally, the site is located within the northern portion of the Peninsular Ranges geomorphic province. This geomorphic province is characterized by northwest-trending physiographic and geologic features such as the Newport-Inglewood Fault Zone and the Whittier Fault Zone located approximately 9.5 miles to the southwest and the 9.8 miles to the southeast.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Quaternary age alluvial deposits consisting of sand, silt, and gravel (Dibblee, 1989; California Geological Survey, 2012). The Quaternary age sediments are in turn underlain by sedimentary bedrock of the Miocene age Puente Formation (Lamar, 1970; Dibblee, 1989). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 1.5 feet below existing ground surface. The artificial fill generally consists of brown to dark brown, fine- to medium-grained silty sand with some fine to coarse gravel. The artificial fill is characterized as dry and loose. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Alluvium

Quaternary age alluvium was encountered beneath the fill. The alluvium generally consists of light brown to dark brown, yellowish brown or olive brown silty sand, sandy silt and silt. The alluvial soils are primarily fine- to medium-grained, slightly moist to wet and medium dense to dense or soft to hard.

4.3 Puente Formation

Miocene age sedimentary bedrock of the Puente Formation was encountered in borings B1 and B5 at depths of approximately 21½ and 14 feet beneath the existing ground surface, respectively. The bedrock consists of interbedded siltstone and fine-grained sandstone that is thinly bedded, soft and highly weathered.

5. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Los Angeles Quadrangle (California Division of Mines and Geology [CDMG], 1998) indicates the historically highest groundwater level in the area is approximately 20 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was encountered in boring B1 at a depth of approximately 17 feet below the existing ground surface. Based on the depth to groundwater encountered in our borings, the reported historic high groundwater levels (CDMG, 1998) and the depth of proposed construction, groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.16).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (CGS, 2018a). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2018b; CGS, 2017) or a city-designated Preliminary Fault Rupture Study Area (City of Los Angeles, 2018) for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Raymond Fault located approximately 4.0 miles to the north (Ziony and Jones, 1989). Other nearby active faults include the Hollywood Fault, the Verdugo Fault, the Newport-Inglewood Fault Zone, and the Whittier Fault Zone located approximately 4.2 miles north-northwest, 5.6 miles north, 9.5 miles southwest, and 9.8 miles southeast of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 33 miles north of the site.

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987 M_w 5.9 Whittier Narrows earthquake and the January 17, 1994 M_w 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. These thrust faults and others in the Los Angeles area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 4, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	73	ESE
Near Redlands	July 23, 1923	6.3	55	E
Long Beach	March 10, 1933	6.4	34	SSE
Tehachapi	July 21, 1952	7.5	79	NW
San Fernando	February 9, 1971	6.6	26	NNW
Whittier Narrows	October 1, 1987	5.9	7	E
Sierra Madre	June 28, 1991	5.8	18	NE
Landers	June 28, 1992	7.3	102	E
Big Bear	June 28, 1992	6.4	80	E
Northridge	January 17, 1994	6.7	21	WNW
Hector Mine	October 16, 1999	7.1	117	ENE

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in Southern California and the effects of ground shaking can be mitigated if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

6.3 Seismic Design Criteria

The following table summarizes summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the computer program *U.S. Seismic Design Maps*, provided by the USGS. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.556g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.891g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.556g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.336g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.704g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.891g	Section 1613.3.4 (Eqn 16-40)

The table below presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10.

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.972g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.972g	Section 11.8.3 (Eqn 11.8-1)

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2016 California Building Code and ASCE 7-10, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain "Life Safety" during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2008 Conterminous U.S. Dynamic edition. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.63 magnitude event occurring at a hypocentral distance of 5.71 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.64 magnitude occurring at a hypocentral distance of 9.48 kilometers from the site.

Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

6.4 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

The current standard of practice, as outlined in the "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California" and "Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California" requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

The State of California Seismic Hazard Zone Map for the Los Angeles Quadrangle (CDMG, 1999) indicates that the site is located in an area identified as having a potential for liquefaction. Also, according to the Los Angeles County Safety Element (1990), the site is located within an area identified as having a potential for liquefaction.

Liquefaction analysis of the soils underlying the site was performed using an updated version of the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance and field performance data.

Screening criteria developed by Bray and Sancio (2006) characterize fine-grained soils which are not susceptible to liquefaction as soils with a plasticity index (PI) that is greater than 18 or with a saturated moisture content that is less than 80 percent of the liquid limit. In order to apply the screening criteria, laboratory testing was performed to evaluate the Atterberg Limits and saturated moisture content of select soil samples. Laboratory test results used for the screening criteria are presented as Figure B5.

The liquefaction analysis was performed for a Design Earthquake level by using a historic high groundwater table of 17 feet below the ground surface, a magnitude 6.64 earthquake, and a peak horizontal acceleration of 0.648g ($\frac{2}{3}PGA_M$). The enclosed liquefaction analysis, included herein for boring B1, indicates that the alluvial soils below the historic high groundwater level would not be susceptible to liquefaction settlement during Design Earthquake ground motion (see enclosed calculation sheets, Figures 5 and 6).

It is our understanding that the intent of the Building Code is to maintain "Life Safety" during Maximum Considered Earthquake level events. Therefore, additional analysis was performed to evaluate the potential for liquefaction during a MCE event. The structural engineer should evaluate the proposed structure for the anticipated MCE liquefaction induced settlements and verify that anticipated deformations would not cause the foundation system to lose the ability to support the gravity loads and/or cause collapse of the structure.

The liquefaction analysis was also performed for the Maximum Considered Earthquake level by using a historic high groundwater table of 17 feet below the ground surface, a magnitude 6.63 earthquake, and a peak horizontal acceleration of 0.972g (PGA_M). The enclosed liquefaction analysis, included herein for boring B1, indicates that the alluvial soils below the historic high groundwater level would not be susceptible to liquefaction settlement during Maximum Considered Earthquake ground motion (see enclosed calculation sheets, Figures 7 and 8).

6.5 Slope Stability

The topography at the site is relatively level and the topography in the immediate site vicinity slopes gently to the west or southwest. The site is located within a City of Los Angeles Hillside Grading Area and a Hillside Ordinance Area (City of Los Angeles, 2018). However, the site and adjacent sites are not located within an area identified as having a potential for seismic slope instability (CDMG, 1999; City of Los Angeles, 2018). As previously indicated, an approximately 40-foot-high, 2:1 (H:V gradient) graded slope ascends from the southeast corner of the site. Based on the 2:1 gradient of the slope, it is not considered a stability hazard and is not anticipated to adversely impact the site. There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The Los Angeles County Safety Element (Leighton, 1990) indicates that the site is located within the Ascot Reservoir inundation area. However, this reservoir, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design, construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

6.7 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area. Therefore, tsunamis are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Therefore, flooding resulting from a seismically-induced seiche is considered unlikely.

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2018; LACDPW, 2018b). Therefore, flooding is not anticipated to adversely impact the site.

6.8 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Well Finder Website, the site is not located within the limits of an oilfield and oil or gas wells are not located within 1,200 feet of the site. However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the DOGGR.

The site is located within the boundaries of a city-designated Methane Zone (City of Los Angeles, 2018). Therefore, there is a potential for methane or other volatile gases to occur at the site. The City of Los Angeles will require a methane study be performed for the proposed development. It is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction.

7.1.2 The enclosed seismically-induced settlement analyses indicates that the alluvial soils underlying the site would not be susceptible to liquefaction as a result of the Design Earthquake peak ground acceleration ($\frac{2}{3}PGA_M$).

7.1.3 Up to 1½ feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the Grading section of this report are followed (see Section 7.4).

7.1.4 Based on these considerations, it is recommended that the upper 5 feet of existing earth materials within the building footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as needed to remove any encountered fill or soft soils as necessary at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. Proposed building foundations should be underlain by a minimum of 3 feet of newly placed engineered fill. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.4).

7.1.5 Subsequent to the recommended grading, the proposed structures may be supported on a conventional shallow spread foundation system deriving support in newly placed engineered fill.

7.1.6 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the upper 12 inches of the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.1.7 It is anticipated that stable excavations for the recommended grading associated with the proposed structure can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.15).

7.1.8 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils found at or below a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

7.1.9 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft alluvial soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of subgrade soil should be scarified and properly compacted for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.12).

7.1.10 Due to the shallow existing groundwater elevation at the site, and minimum City of Los Angeles setback requirements for infiltration systems from existing groundwater elevations and foundations, a stormwater infiltration system is not recommended for this development. It is suggested that stormwater be retained, filtered and discharged in accordance with the requirements of the local governing agency.

7.1.11 Once the design and foundation loading configuration for the proposed structure proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be re-evaluated by this office.

7.1.12 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular soils are encountered.

7.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of existing adjacent improvements.

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.15).

7.2.4 The upper 5 feet of existing site soils encountered during this investigation are considered to have a "moderate" to "high" expansive potential (EI = 81 & 101); and are classified as "expansive" based on the 2016 California Building Code (CBC) Section 1803.5.3. Recommendations presented herein assume that the building foundations and slabs will derive support in these materials.

7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "severely corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B8) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that ABS pipes be considered in lieu of cast-iron for subdrains and retaining wall drains.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B8) and indicate that the on-site materials possess "severe" sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3. *The table below presents a summary of concrete requirements set forth by 2016 CBC Section 1904 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.*

REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Exposure	Exposure Class	Water-Soluble Sulfate Percent by Weight	Cement Type (ASTM C150)	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
Not Applicable	S0	SO_4<0.10	--	--	2,500
Moderate	S1	0.10≤SO_4<0.20	II	0.50	4,000
Severe	**S2**	**0.20≤SO_4≤2.00**	**V**	**0.45**	**4,500**
Very Severe	S3	SO_4>2.00	V+Pozzolan or Slag	0.45	4,500

7.3.3 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

7.4 Grading

7.4.1 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and alluvial soil encountered during exploration is suitable for re-use as engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris are removed.

7.4.2 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer, geotechnical engineer, and building official in attendance. Special soil handling requirements can be discussed at that time.

7.4.3 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.) and the City of Los Angeles Inspector.

7.4.4 As a minimum, it is recommended that the upper five feet of existing earth materials within the proposed building footprint area be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as necessary to remove deeper artificial fill or soft alluvial soil at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint area, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. Proposed building foundations should be underlain by a minimum of three feet of newly placed engineered fill. The limits of existing fill and/or soft alluvial soils removal will be verified by the Geocon representative during site grading activities.

7.4.5 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the upper 12 inches of the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.4.6 The City of Los Angeles Department of Building and Safety requires a minimum compactive effort of 95 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition) where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeter. Soils with more than 15 percent finer than 0.005 millimeter may be compacted to 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition). Fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned at least two percent over optimum moisture content and properly compacted in accordance with ASTM D 1557 (latest edition).

7.4.7. Where new paving is to be placed, it is recommended that all existing fill and soft alluvium be excavated and properly compacted for paving support. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to two percent above optimum moisture content, and compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.12).

7.4.8 It is anticipated that stable excavations for the recommended grading can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of the existing offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.15).

7.4.9 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils found at or below a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

7.4.10 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry as backfill is also acceptable (see Section 7.5). Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

7.4.11 All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 50 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B8). Import soils placed in the building area should be placed uniformly across the building pad or in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).

7.4.12 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding materials, fill, steel, gravel, or concrete.

7.5 Controlled Low Strength Material (CLSM)

7.5.1 Controlled Low Strength Material (CLSM) may be utilized in lieu of compacted soil as engineered fill where approved in writing by the Geotechnical Engineer. Where utilized within the City of Los Angeles use of CLSM is subject to the following requirements:

Standard Requirements

1. CLSM shall be ready-mixed by a City of Los Angeles approved batch plant;
2. CLSM shall not be placed on uncertified fill, on incompetent natural soil, nor below water;
3. CLSM shall not be placed on a sloping surface with a gradient steeper than 5:1 (horizontal to vertical);
4. Placement of the CLSM shall be under the continuous inspection of a concrete deputy inspector;
5. The excavation bottom shall be accepted by the soil engineer and the City Inspector prior to placing CLSM.

Requirements for CLSM that will be used for support of footings

1. The cement content of the CLSM shall not be less than 188 pounds per cubic yard (min. 2 sacks);
2. The excavation bottom must be level, cleaned of loose soils and approved in writing by Geocon prior to placement of the CLSM;
3. The ultimate compressive strength of the CLSM shall be no less than 100 pounds per square inch (psi) when tested on the 28th-day per ASTM D4832 (latest edition), Standard Test Method for Preparation and Testing of Controlled Low Strength Material Test Cylinders. Compression testing will be performed in accordance with ASTM C39 and City of Los Angeles requirements;
4. Samples of the CLSM will be collected during placement, a minimum of one test (two cylinders) for each 50 cubic yards or fraction thereof;
5. Overexcavation for CLSM placement shall extend laterally beyond the footprint of any proposed footings as required for placement of compacted fill, unless justified otherwise by the soil engineer that footings will have adequate vertical and horizontal bearing capacity.

7.6 Shrinkage

7.6.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 10 and 15 percent should be anticipated when excavating and compacting the upper five feet of existing earth materials on the site to an average relative compaction of 95 percent.

7.4.2 If import soils will be utilized in the building pad, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.). Soils can be borrowed from non-building pad areas and later replaced with imported soils.

7.7 Foundation Design

7.7.1 Subsequent to the recommended grading, a conventional shallow spread foundation system may be utilized for support of the proposed structure provided foundations derive support in newly placed engineered fill. Foundations should be underlain by a minimum of three feet of newly placed engineered fill.

7.7.2 Continuous footings may be designed for an allowable bearing capacity of 2,200 pounds per square foot (psf), and should be a minimum of 12 inches in width, 30 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.7.3 Isolated spread foundations may be designed for an allowable bearing capacity of 2,600 psf, and should be a minimum of 24 inches in width, 30 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.7.4 The allowable soil bearing pressure above may be increased by 200 psf and 400 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 4,200 psf.

7.7.5 The allowable bearing pressures may be increased by one-third for transient loads due to wind or seismic forces.

7.7.6 If depth increases are utilized for the perimeter foundations, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary. Additional grading should be conducted as-needed in order to maintain the required 3-foot thick blanket of engineered fill below proposed foundations.

7.7.7 Continuous footings should be reinforced with four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. Reinforcement for spread footings should be designed by the project structural engineer.

7.7.8 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only, and are not intended to be used in lieu of those required for structural purposes.

7.7.9 Due to the expansive potential of the subgrade soils, the moisture content in the slab and foundation subgrade should be maintained at 2 percent above optimum moisture content prior to and at the time of concrete placement.

7.7.10 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

7.7.11 This office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

7.8 Foundation Settlement

7.8.1 The maximum expected static settlement for a structure supported on a conventional foundation system deriving support in the recommended bearing materials and designed with a maximum bearing pressure of 4,000 psf is estimated to be less than 1 inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ¾ inch over a distance of 20 feet.

7.8.2 Once the design and foundation loading configurations for the proposed structures proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

7.9 Miscellaneous Foundations

7.9.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures which will not be tied to the proposed structure may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils found at or below a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12 inch embedment into the recommended bearing materials.

7.9.2 If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 30 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.9.3 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated.

7.10 Lateral Design

7.10.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces in the competent alluvial soils or in properly compacted engineered fill.

7.10.2 Passive earth pressure for the sides of foundations and slabs poured against properly compacted engineered fill or competent alluvial soils may be computed as an equivalent fluid having a density of 220 pcf with a maximum earth pressure of 2,200 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.11 Concrete Slabs-on-Grade

7.11.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Pavement Recommendations* section of this report (Section 7.12).

7.11.2 Subsequent to the recommended grading, concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 4 inches thick and minimum slab reinforcement should consist of No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.

7.11.3 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the Los Angeles Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Los Angeles Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

7.11.4 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.11.5 Exterior slabs for walkways or flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to two percent above optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 8 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

7.11.6 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

7.12 Preliminary Pavement Recommendations

7.12.1 Where new paving is to be placed, it is recommended that all existing fill and soft or unsuitable alluvial materials be excavated and properly recompacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing unsuitable material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to 2 percent above optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.12.2 The following pavement sections are based on an assumed R-Value of 20. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.

7.12.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking And Driveways	4.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	12.0

7.12.4 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction"* (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation"* (Caltrans). The use of Crushed Miscellaneous Base in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction"* (Green Book).

7.12.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 95 percent relative compactions, respectively, determined by ASTM Test Method D 1557 (latest edition).

7.12.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

7.13 Retaining Walls Design

7.13.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 5 feet. In the event that walls significantly higher than 5 feet are planned, Geocon should be contacted for additional recommendations.

7.13.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* sections of this report (see Section 7.7).

7.13.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 30 pcf.

7.13.4 Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 50 pcf.

7.13.5 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvial soils or engineered fill derived from onsite soils. If import soil will be used to backfill proposed retaining walls, revised earth pressures may be required to account for the geotechnical properties of the import soil used as engineered fill. This should be evaluated once the use of import soil is established. All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site.

7.13.6 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

7.13.7 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

7.14 Retaining Wall Drainage

7.14.1 Retaining walls should be provided with a drainage system extended at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 9). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.14.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 10). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

7.14.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

7.14.4 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

7.15 Temporary Excavations

7.15.1 Excavations on the order of 5 feet in height may be required during grading operations. The excavations are expected to expose artificial fill and alluvial soils, which are suitable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present, and where not surcharged by adjacent traffic or structures.

7.15.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to maximum height of 8 feet. A uniform slope does not have a vertical portion.

7.15.3 If excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such as slot-cutting or shoring may be necessary in order to maintain lateral support of offsite improvements. Recommendations for alterative temporary excavation measures can be provided under separate cover, if needed.

7.15.4 Where sloped embankments are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. Geocon personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

7.16 Surface Drainage

7.16.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

7.16.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2016 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.16.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond.

7.16.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or an impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

7.17 Plan Review

7.17.1 Grading, shoring and foundation plans should be reviewed by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon West, Inc.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

LIST OF REFERENCES

California Department of Water Resources, 2018, *Groundwater Level Data by Township, Range, and Section,* http://www.water.ca.gov/waterdatalibrary/groundwater/ hydrographs/index_trs.cfm.

California Department of Water Resources, 1961, *Planned Utilization of Groundwater Basins of the Coastal Plain of Los Angeles County*, Bulletin 104, Appendix A.

California Division of Mines and Geology, 1999; *State of California Seismic Hazard Zones, Los Angeles Quadrangle,* Official Map, Released: March 25, 1999.

California Division of Mines and Geology, 1998, *Seismic Hazard Evaluation of the Los Angeles 7.5-Minute Quadrangle, Los Angeles County, California,* Open File Report 98-20.

California Division of Oil, Gas and Geothermal Resources, 2018, Division of Oil, Gas, and Geothermal Resources Well Finder, http://maps.conservation.ca.gov.doggr/index.html#close. Accessed March 12, 2018.

California Geological Survey, 2018a, *Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California,* Special Publication 42, Revised 2018.

California Geological Survey, 2018b, CGS Information Warehouse, Regulatory Map Portal, http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps.

California Geological Survey, 2017, *Earthquake Zones of Required Investigation, Los Angeles Quadrangle, Los Angeles County, California*, *dated June 15, 2017*.

California Geological Survey, 2012, *Geologic Compilation of Quaternary Surficial Deposits in Southern California, Los Angeles 30' X 60' Quadrangle*, A Project for the Department of Water Resources by the California Geological Survey, Compiled from existing sources by Trinda L. Bedrossian, CEG and Peter D. Roffers, CGS Special Report 217, Plate 9, Scale 1:100,000.

Dibblee, T. W., Jr., 1989, *Geologic Map of the Los Angeles Quadrangle, Los Angeles County, California,* Dibblee Foundation Map # DF-22.

FEMA, 2018, Online Flood Hazard Maps, *Flood Insurance Rate Map, Los Angeles County, California and Unincorporated Areas, Map Number 06037C1636F, Date Accessed: March 12, 2018,* http://www.esri.com/hazards/index.html.

Jennings, C. W. and Bryant, W. A., 2010, *Fault Activity Map of California,* California Geological Survey Geologic Data Map No. 6.

Lamar, D.L., 1970, *Geology of the Elysian Park-Repetto Hills Area, Los Angeles County, California,* California Division of Mines and Geology Special Report 101.

Leighton and Associates, Inc., 1990, *Technical Appendix to the Safety Element of the Los Angeles County General Plan, Hazard Reduction in Los Angeles County*.

Los Angeles, City of, 2018, NavigateLA website, http://navigatela.lacity.org.

LIST OF REFERENCES (Continued)

Los Angeles, City of, 1996, *Safety Element of the Los Angeles City General Plan*.

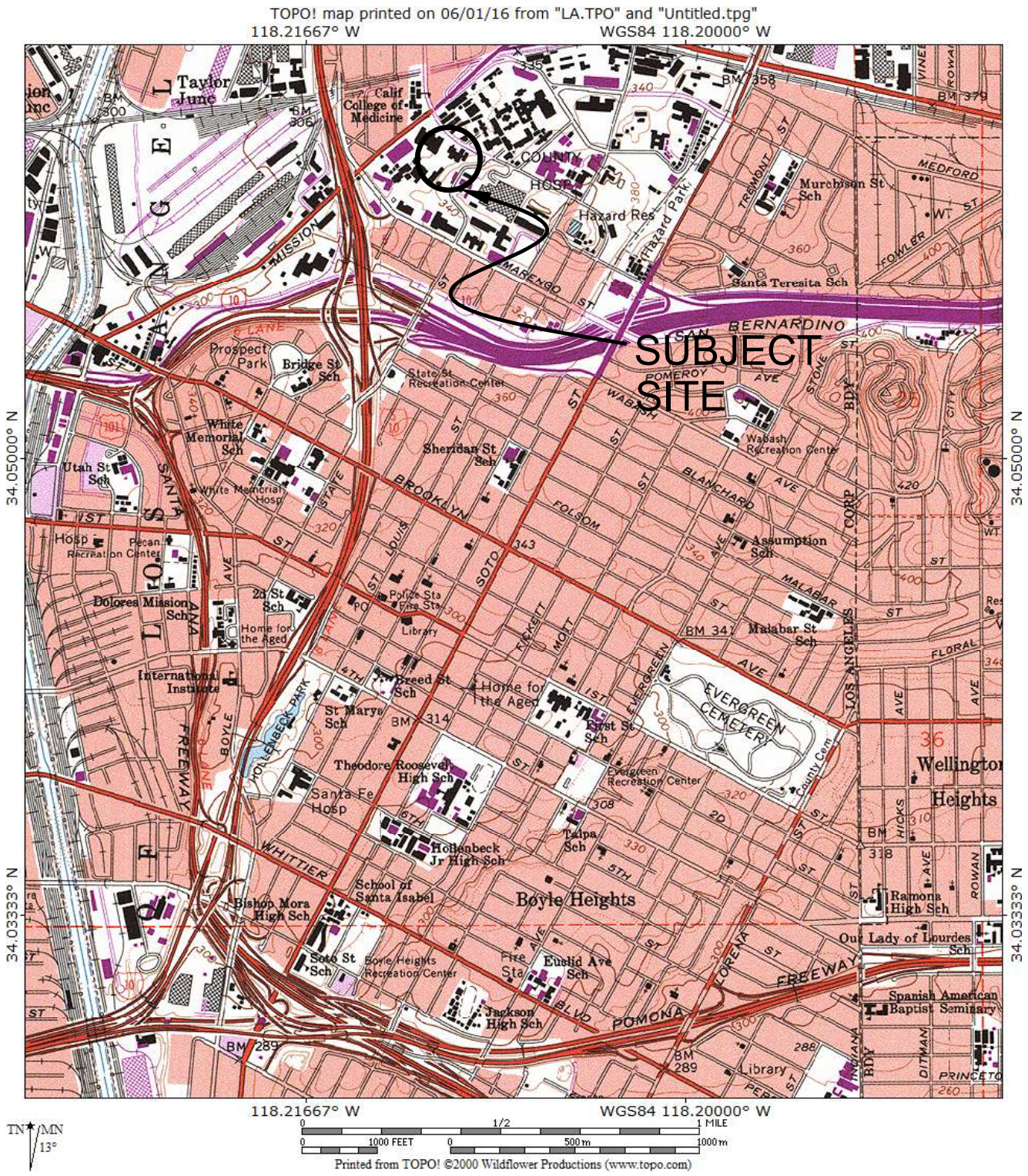
Los Angeles County Department of Public Works, 2018a, Ground Water Wells Website, http://dpw2.co.la.ca.us/website/wells/viewer.asp.

Los Angeles County Department of Public Works, 2018b, Flood Zone Determination Website, http://dpw.lacounty.gov/apps/wmd/floodzone/map.htm.

Toppozada, T., Branum, D., Petersen, M, Hallstrom, C., and Reichle, M., 2000, *Epicenters and Areas Damaged by M> 5 California Earthquakes, 1800 – 1999,* California Geological Survey, Map Sheet 49.

U.S. Geological Survey Seismic Design Maps Web Application, 2016, http://earthquake.usgs.gov/designmaps/us/application.php.

Ziony, J. I., and Jones, L. M., 1989, *Map Showing Late Quaternary Faults and 1978–1984 Seismicity of the Los Angeles Region, California*, U.S. Geological Survey Miscellaneous Field Studies Map MF-1964.



REFERENCE:U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, LOS ANGELES, CA QUADRANGLE

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

VICINITY MAP

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

DRAFTED BY: RA	CHECKED BY: SFK

APRIL 2018	PROJECT NO. A9733-06-01	FIG. 1



0 40' 80'

LEGEND

B5 Approximate Location of Boring

GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

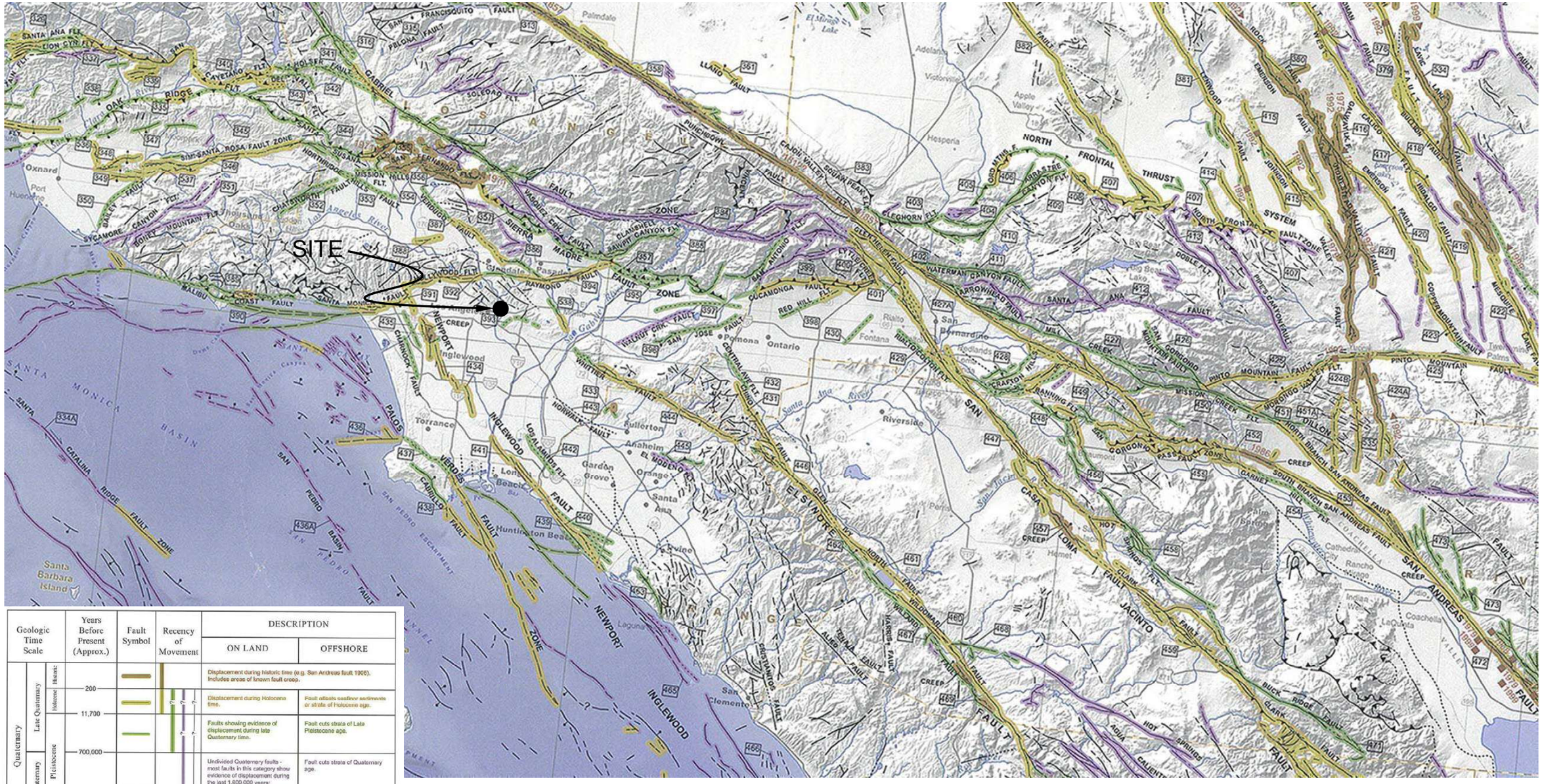
DRAFTED BY: PZ	CHECKED BY: HHD

SITE PLAN

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018	PROJECT NO. A9733-06-01	FIG. 2

Reference: Jennings, C.W. and Bryant, W. A., 2010, Fault Activity Map of California, California Geological Survey Geologic Data Map No. 6.



Geologic Time Scale		Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
					ON LAND	OFFSHORE
Quaternary	Late Quaternary – Historic	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
	Late Quaternary – Holocene	11,700		? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
	Pleistocene			? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary – Pleistocene	700,000		?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary		1,600,000*			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.
		4.5 billion (Age of Earth)				

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.8 Ma criterion.

N

0 12 24 Miles

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RA | CHECKED BY: SFK

REGIONAL FAULT MAP

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. 3

SITE

SANTA BARBARA

VENTURA

LOS ANGELES

ORANGE

RIVERSIDE

IMPERIAL

SAN DIEGO

120°

33°

119°

EPICENTER MAP LEGEND

Period	1800 - 1868	1869 - 1931	1932 - 1999
≥ 7.0			
6.5 - 6.9			
6.0 - 6.4			
5.5 - 5.9			
5.0 - 5.4			

Magnitude (M)

Historical Faulting

Holocene Faulting

Highways (Major)

Highways (Minor)

Lakes

65 Last two digits of M ≥ 6.5 earthquake year

N

0 20 40 Miles

Reference: Toppozada, T., Branum, D., Petersen, M., Hallstrom, C., Cramer, C., and Reichle, M., 2000, Epicenters and Areas Damaged by M≥5 California Earthquakes, 1800 - 1999, California Geological Survey, Map Sheet 49.

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RA | CHECKED BY: SFK

REGIONAL SEISMICITY MAP

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. 4

GEOCON

Client : 1774 Zonal Ave
File No. : A9733-06-01
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.64
Peak Horiz. Acceleration PGA_M (g):	0.972
2/3 PGA_M (g):	0.648
Calculated Mag.Wtg.Factor:	0.736
Historic High Groundwater:	17.0
Groundwater Depth During Exploration:	17.0

By Thomas F. Blake (1994-1996)
ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.15
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf): 62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	132.4	0	15.0	5.0	1		84	2.000	38.8	132.4	Infin.	0.998	0.309	--
2.0	132.4	0	15.0	5.0	1	38	84	2.000	45.8	132.4	Infin.	0.993	0.308	--
3.0	132.4	0	15.0	5.0	1	38	84	2.000	45.8	132.4	Infin.	0.989	0.306	--
4.0	132.4	0	15.0	5.0	1	38	84	2.000	45.8	132.4	Infin.	0.984	0.305	--
5.0	132.4	0	15.0	5.0	1	38	84	1.872	43.3	132.4	Infin.	0.979	0.304	--
6.0	132.4	0	15.0	5.0	1	38	84	1.694	39.9	132.4	Infin.	0.975	0.302	--
7.0	132.4	0	15.0	5.0	1	38	84	1.558	37.2	132.4	Infin.	0.970	0.301	--
8.0	140.0	0	35.0	7.5	1		119	1.448	65.5	140.0	Infin.	0.966	0.299	--
9.0	140.0	0	35.0	7.5	1		119	1.355	61.4	140.0	Infin.	0.961	0.298	--
10.0	140.0	0	41.0	10.0	1		123	1.279	67.8	140.0	Infin.	0.957	0.297	--
11.0	140.0	0	41.0	10.0	1		123	1.214	64.4	140.0	Infin.	0.952	0.295	--
12.0	140.0	0	41.0	10.0	1		123	1.158	61.4	140.0	Infin.	0.947	0.294	--
13.0	129.6	0	6.0	15.0	0			1.111	9.3	129.6	~	0.943	0.292	~
14.0	129.6	0	6.0	15.0	0			1.071	8.9	129.6	~	0.938	0.291	~
15.0	129.6	0	6.0	15.0	0			1.035	8.6	129.6	~	0.934	0.289	~
16.0	129.6	0	6.0	15.0	0			1.002	8.4	129.6	~	0.929	0.288	~
17.0	129.6	1	6.0	15.0	0			0.979	8.2	67.2	~	0.925	0.291	~
18.0	128.3	1	11.0	20.0	0			0.964	16.4	65.9	~	0.920	0.297	~
19.0	128.3	1	11.0	20.0	0			0.951	16.1	65.9	~	0.915	0.303	~
20.0	128.3	1	11.0	20.0	0			0.937	15.9	65.9	~	0.911	0.308	~
21.5	128.3	1	11.0	20.0	0			0.921	15.6	65.9	~	0.905	0.314	~
22.0	122.2	1	14.0	25.0	0			0.916	21.1	59.8	~	0.901	0.316	~
23.0	122.2	1	14.0	25.0	0			0.902	20.8	59.8	~	0.897	0.322	~
24.0	122.2	1	14.0	25.0	0			0.892	20.6	59.8	~	0.893	0.326	~
25.0	122.2	1	14.0	25.0	0			0.882	20.3	59.8	~	0.888	0.330	~
26.0	122.2	1	14.0	25.0	0			0.872	20.1	59.8	~	0.883	0.333	~
27.0	125.3	1	14.0	25.0	0			0.863	19.9	62.9	~	0.879	0.336	~
28.0	125.3	1	40.0	30.0	1		98	0.853	58.9	62.9	Infin.	0.874	0.339	Non-Liq.
29.0	125.3	1	40.0	30.0	1		98	0.844	58.2	62.9	Infin.	0.870	0.341	Non-Liq.
30.0	125.3	1	40.0	30.0	1		98	0.835	57.6	62.9	Infin.	0.865	0.344	Non-Liq.
31.0	125.3	1	40.0	30.0	1		98	0.826	57.0	62.9	Infin.	0.861	0.346	Non-Liq.
32.0	129.5	1	40.0	30.0	1		98	0.818	56.4	67.1	Infin.	0.856	0.348	Non-Liq.
33.0	129.5	1	40.0	30.0	1		98	0.809	55.8	67.1	Infin.	0.851	0.349	Non-Liq.
34.0	129.5	1	40.0	30.0	1		98	0.801	55.2	67.1	Infin.	0.847	0.351	Non-Liq.
35.0	129.5	1	51.0	35.0	1		108	0.793	69.7	67.1	Infin.	0.842	0.352	Non-Liq.
36.0	129.5	1	51.0	35.0	1		108	0.785	69.0	67.1	Infin.	0.838	0.353	Non-Liq.
37.0	129.5	1	51.0	35.0	1		108	0.777	68.4	67.1	Infin.	0.833	0.354	Non-Liq.
38.0	129.5	1	51.0	35.0	1		108	0.770	67.7	67.1	Infin.	0.829	0.355	Non-Liq.
39.0	129.5	1	51.0	35.0	1		108	0.762	67.1	67.1	Infin.	0.824	0.355	Non-Liq.
40.0	129.5	1	50.0	40.0	1		103	0.755	65.1	67.1	Infin.	0.819	0.356	Non-Liq.
41.0	129.5	1	50.0	40.0	1		103	0.748	64.6	67.1	Infin.	0.815	0.356	Non-Liq.
42.0	129.5	1	50.0	40.0	1		103	0.742	64.0	67.1	Infin.	0.810	0.357	Non-Liq.
43.0	129.5	1	50.0	40.0	1		103	0.735	63.4	67.1	Infin.	0.806	0.357	Non-Liq.
44.0	129.5	1	50.0	40.0	1		103	0.729	62.9	67.1	Infin.	0.801	0.357	Non-Liq.
45.0	129.5	1	54.0	45.0	1		104	0.723	67.3	67.1	Infin.	0.797	0.357	Non-Liq.
46.0	129.5	1	54.0	45.0	1		104	0.717	66.8	67.1	Infin.	0.792	0.357	Non-Liq.
47.0	129.5	1	54.0	45.0	1		104	0.711	66.2	67.1	Infin.	0.787	0.357	Non-Liq.
48.0	129.5	1	54.0	45.0	1		104	0.705	65.7	67.1	Infin.	0.783	0.356	Non-Liq.
49.0	129.5	1	54.0	45.0	1		104	0.700	65.2	67.1	Infin.	0.778	0.356	Non-Liq.
50.0	129.5	1	50.0	50.0	1		98	0.694	59.9	67.1	Infin.	0.774	0.356	Non-Liq.

Figure 5

GEOCON

Client : 1774 Zonal Ave
File No. : A9733-06-01
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.64
PGAM (g):	0.972
2/3 PGAM (g):	0.65
Calculated Mag.Wtg.Factor:	0.736
Historic High Groundwater:	17.0
Groundwater @ Exploration:	17.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	τ_{av}/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain $[e_{15}]$ (%)	EQ. SETTLE. Pe (in.)
1.0	15	132.3608	0.033	0.033	84	39	0.421	--	0.00	0.00
2.0	15	132.3608	0.099	0.099	84	46	0.421	--	0.00	0.00
3.0	15	132.3608	0.165	0.165	84	46	0.421	--	0.00	0.00
4.0	15	132.3608	0.232	0.232	84	46	0.421	--	0.00	0.00
5.0	15	132.3608	0.298	0.298	84	43	0.421	--	0.00	0.00
6.0	15	132.3608	0.364	0.364	84	40	0.421	--	0.00	0.00
7.0	15	132.3608	0.430	0.430	84	37	0.421	--	0.00	0.00
8.0	35	139.968	0.498	0.498	119	66	0.421	--	0.00	0.00
9.0	35	139.968	0.568	0.568	119	61	0.421	--	0.00	0.00
10.0	41	139.968	0.638	0.638	123	68	0.421	--	0.00	0.00
11.0	41	139.968	0.708	0.708	123	64	0.421	--	0.00	0.00
12.0	41	139.968	0.778	0.778	123	61	0.421	--	0.00	0.00
13.0	6	129.6464	0.846	0.846		9	0.421	~	0.00	0.00
14.0	6	129.6464	0.910	0.910		9	0.421	~	0.00	0.00
15.0	6	129.6464	0.975	0.975		9	0.421	~	0.00	0.00
16.0	6	129.6464	1.040	1.040		8	0.421	~	0.00	0.00
17.0	6	129.6464	1.105	1.089		8	0.427	~	0.00	0.00
18.0	11	128.3194	1.169	1.123		16	0.439	~	0.00	0.00
19.0	11	128.3194	1.234	1.156		16	0.450	~	0.00	0.00
20.0	11	128.3194	1.298	1.188		16	0.460	~	0.00	0.00
21.5	11	128.3194	1.378	1.230		16	0.472	~	0.00	0.00
22.0	14	122.1536	1.409	1.245		21	0.477	~	0.00	0.00
23.0	14	122.1536	1.486	1.283		21	0.488	~	0.00	0.00
24.0	14	122.1536	1.547	1.313		21	0.497	~	0.00	0.00
25.0	14	122.1536	1.608	1.343		20	0.505	~	0.00	0.00
26.0	14	122.1536	1.669	1.372		20	0.512	~	0.00	0.00
27.0	14	125.2875	1.731	1.403		20	0.520	~	0.00	0.00
28.0	40	125.2875	1.793	1.434	98	59	0.527	Non-Liq.	0.00	0.00
29.0	40	125.2875	1.856	1.466	98	58	0.534	Non-Liq.	0.00	0.00
30.0	40	125.2875	1.919	1.497	98	58	0.540	Non-Liq.	0.00	0.00
31.0	40	125.2875	1.981	1.529	98	57	0.546	Non-Liq.	0.00	0.00
32.0	40	129.5382	2.045	1.561	98	56	0.552	Non-Liq.	0.00	0.00
33.0	40	129.5382	2.110	1.595	98	56	0.557	Non-Liq.	0.00	0.00
34.0	40	129.5382	2.174	1.628	98	55	0.563	Non-Liq.	0.00	0.00
35.0	51	129.5382	2.239	1.662	108	70	0.568	Non-Liq.	0.00	0.00
36.0	51	129.5382	2.304	1.696	108	69	0.573	Non-Liq.	0.00	0.00
37.0	51	129.5382	2.369	1.729	108	68	0.577	Non-Liq.	0.00	0.00
38.0	51	129.5382	2.434	1.763	108	68	0.582	Non-Liq.	0.00	0.00
39.0	51	129.5382	2.498	1.796	108	67	0.586	Non-Liq.	0.00	0.00
40.0	50	129.5382	2.563	1.830	103	65	0.590	Non-Liq.	0.00	0.00
41.0	50	129.5382	2.628	1.863	103	65	0.594	Non-Liq.	0.00	0.00
42.0	50	129.5382	2.693	1.897	103	64	0.598	Non-Liq.	0.00	0.00
43.0	50	129.5382	2.757	1.931	103	63	0.602	Non-Liq.	0.00	0.00
44.0	50	129.5382	2.822	1.964	103	63	0.605	Non-Liq.	0.00	0.00
45.0	54	129.5382	2.887	1.998	104	67	0.609	Non-Liq.	0.00	0.00
46.0	54	129.5382	2.952	2.031	104	67	0.612	Non-Liq.	0.00	0.00
47.0	54	129.5382	3.016	2.065	104	66	0.616	Non-Liq.	0.00	0.00
48.0	54	129.5382	3.081	2.098	104	66	0.619	Non-Liq.	0.00	0.00
49.0	54	129.5382	3.146	2.132	104	65	0.622	Non-Liq.	0.00	0.00
50.0	50	129.5382	3.211	2.166	98	60	0.625	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.0 INCHES

Figure 6

GEOCON

Client : 1774 Zonal Ave
File No. : A9733-06-01
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.63
Peak Horiz. Acceleration PGA_M (g):	0.972
Calculated Mag.Wtg.Factor:	0.733
Historic High Groundwater:	17.0
Groundwater Depth During Exploration:	17.0

By Thomas F. Blake (1994-1996)
ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.15
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf): 62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	132.4	0	15.0	5.0	1	38	84	2.000	45.8	132.4	Infin.	0.998	0.462	--
2.0	132.4	0	15.0	5.0	1	38	84	2.000	45.8	132.4	Infin.	0.993	0.460	--
3.0	132.4	0	15.0	5.0	1	38	84	2.000	45.8	132.4	Infin.	0.989	0.458	--
4.0	132.4	0	15.0	5.0	1	38	84	2.000	45.8	132.4	Infin.	0.984	0.456	--
5.0	132.4	0	15.0	5.0	1	38	84	1.872	43.3	132.4	Infin.	0.979	0.454	--
6.0	132.4	0	15.0	5.0	1	38	84	1.694	39.9	132.4	Infin.	0.975	0.451	--
7.0	132.4	0	15.0	5.0	1	38	84	1.558	37.2	132.4	Infin.	0.970	0.449	--
8.0	140.0	0	35.0	7.5	1		119	1.448	65.5	140.0	Infin.	0.966	0.447	--
9.0	140.0	0	35.0	7.5	1		119	1.355	61.4	140.0	Infin.	0.961	0.445	--
10.0	140.0	0	41.0	10.0	1		123	1.279	67.8	140.0	Infin.	0.957	0.443	--
11.0	140.0	0	41.0	10.0	1		123	1.214	64.4	140.0	Infin.	0.952	0.441	--
12.0	140.0	0	41.0	10.0	1		123	1.158	61.4	140.0	Infin.	0.947	0.439	--
13.0	129.6	0	6.0	15.0	0			1.111	9.3	129.6	~	0.943	0.437	~
14.0	129.6	0	6.0	15.0	0			1.071	8.9	129.6	~	0.938	0.434	~
15.0	129.6	0	6.0	15.0	0			1.035	8.6	129.6	~	0.934	0.432	~
16.0	129.6	0	6.0	15.0	0			1.002	8.4	129.6	~	0.929	0.430	~
17.0	129.6	1	6.0	15.0	0			0.979	8.2	67.2	~	0.925	0.434	~
18.0	128.3	1	11.0	20.0	0			0.964	16.4	65.9	~	0.920	0.444	~
19.0	128.3	1	11.0	20.0	0			0.951	16.1	65.9	~	0.915	0.452	~
20.0	128.3	1	11.0	20.0	0			0.937	15.9	65.9	~	0.911	0.461	~
21.5	128.3	1	11.0	20.0	0			0.921	15.6	65.9	~	0.905	0.470	~
22.0	122.2	1	14.0	25.0	0			0.916	21.1	59.8	~	0.901	0.472	~
23.0	122.2	1	14.0	25.0	0			0.902	20.8	59.8	~	0.897	0.481	~
24.0	122.2	1	14.0	25.0	0			0.892	20.6	59.8	~	0.893	0.487	~
25.0	122.2	1	14.0	25.0	0			0.882	20.3	59.8	~	0.888	0.492	~
26.0	122.2	1	14.0	25.0	0			0.872	20.1	59.8	~	0.883	0.497	~
27.0	125.3	1	14.0	25.0	0			0.863	19.9	62.9	~	0.879	0.502	~
28.0	125.3	1	40.0	30.0	1		98	0.853	58.9	62.9	Infin.	0.874	0.506	Non-Liq.
29.0	125.3	1	40.0	30.0	1		98	0.844	58.2	62.9	Infin.	0.870	0.510	Non-Liq.
30.0	125.3	1	40.0	30.0	1		98	0.835	57.6	62.9	Infin.	0.865	0.513	Non-Liq.
31.0	125.3	1	40.0	30.0	1		98	0.826	57.0	62.9	Infin.	0.861	0.516	Non-Liq.
32.0	129.5	1	40.0	30.0	1		98	0.818	56.4	67.1	Infin.	0.856	0.519	Non-Liq.
33.0	129.5	1	40.0	30.0	1		98	0.809	55.8	67.1	Infin.	0.851	0.521	Non-Liq.
34.0	129.5	1	40.0	30.0	1		98	0.801	55.2	67.1	Infin.	0.847	0.524	Non-Liq.
35.0	129.5	1	51.0	35.0	1		108	0.793	69.7	67.1	Infin.	0.842	0.525	Non-Liq.
36.0	129.5	1	51.0	35.0	1		108	0.785	69.0	67.1	Infin.	0.838	0.527	Non-Liq.
37.0	129.5	1	51.0	35.0	1		108	0.777	68.4	67.1	Infin.	0.833	0.528	Non-Liq.
38.0	129.5	1	51.0	35.0	1		108	0.770	67.7	67.1	Infin.	0.829	0.530	Non-Liq.
39.0	129.5	1	51.0	35.0	1		108	0.762	67.1	67.1	Infin.	0.824	0.531	Non-Liq.
40.0	129.5	1	50.0	40.0	1		103	0.755	65.1	67.1	Infin.	0.819	0.531	Non-Liq.
41.0	129.5	1	50.0	40.0	1		103	0.748	64.6	67.1	Infin.	0.815	0.532	Non-Liq.
42.0	129.5	1	50.0	40.0	1		103	0.742	64.0	67.1	Infin.	0.810	0.533	Non-Liq.
43.0	129.5	1	50.0	40.0	1		103	0.735	63.4	67.1	Infin.	0.806	0.533	Non-Liq.
44.0	129.5	1	50.0	40.0	1		103	0.729	62.9	67.1	Infin.	0.801	0.533	Non-Liq.
45.0	129.5	1	54.0	45.0	1		104	0.723	67.3	67.1	Infin.	0.797	0.533	Non-Liq.
46.0	129.5	1	54.0	45.0	1		104	0.717	66.8	67.1	Infin.	0.792	0.533	Non-Liq.
47.0	129.5	1	54.0	45.0	1		104	0.711	66.2	67.1	Infin.	0.787	0.533	Non-Liq.
48.0	129.5	1	54.0	45.0	1		104	0.705	65.7	67.1	Infin.	0.783	0.532	Non-Liq.
49.0	129.5	1	54.0	45.0	1		104	0.700	65.2	67.1	Infin.	0.778	0.532	Non-Liq.
50.0	129.5	1	50.0	50.0	1		98	0.694	59.9	67.1	Infin.	0.774	0.531	Non-Liq.

Figure 7

GEOCON

Client : 1774 Zonal Ave
File No. : A9733-06-01
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

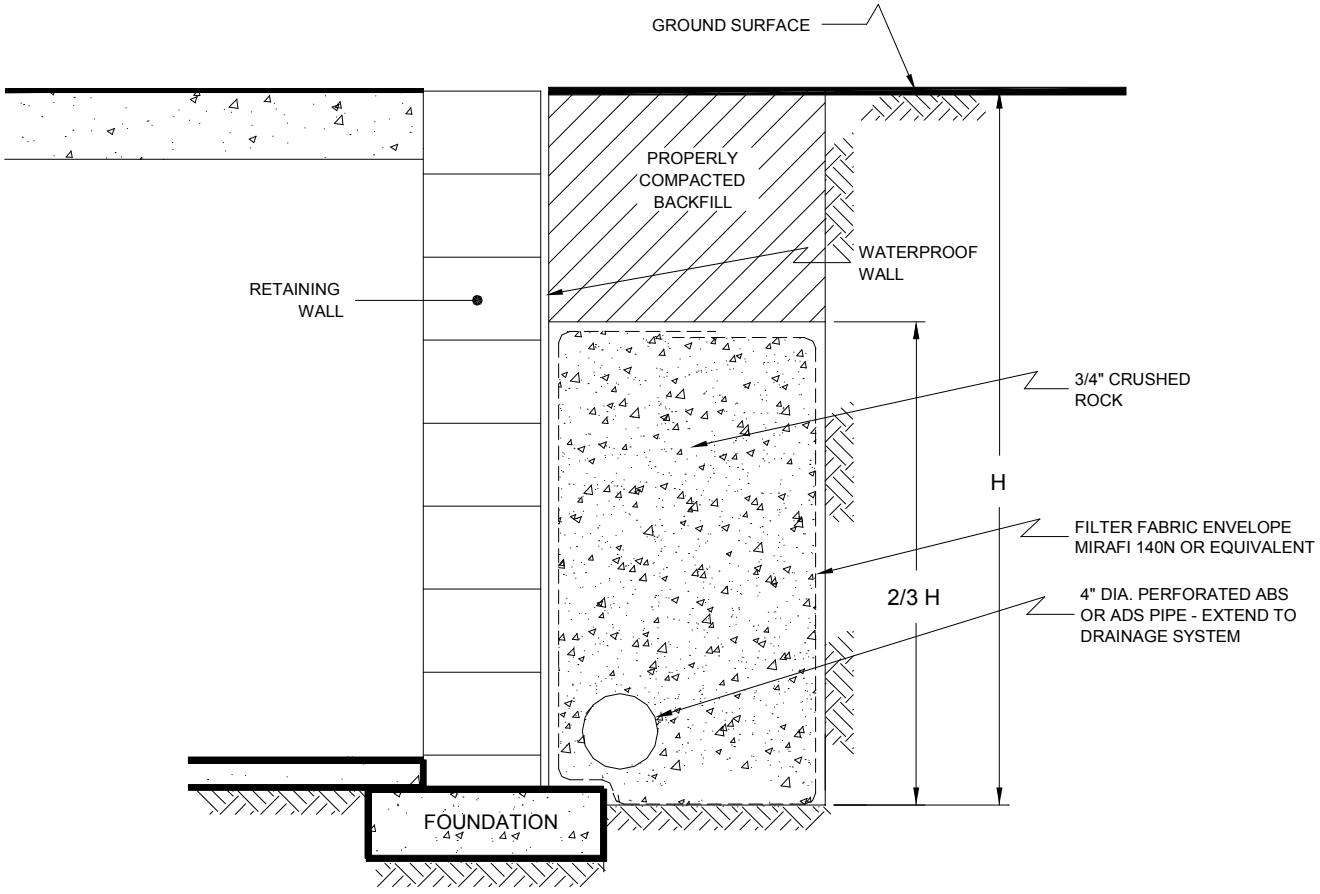
NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.63
PGA_M (g):	0.972
Calculated Mag.Wtg.Factor:	0.733
Historic High Groundwater:	17.0
Groundwater @ Exploration:	17.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	τ_{av}/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain $[e_{15}]$ (%)	EQ. SETTLE. Pe (in.)
1.0	15	132.3608	0.033	0.033	84	46	0.632	--	0.00	0.00
2.0	15	132.3608	0.099	0.099	84	46	0.632	--	0.00	0.00
3.0	15	132.3608	0.165	0.165	84	46	0.632	--	0.00	0.00
4.0	15	132.3608	0.232	0.232	84	46	0.632	--	0.00	0.00
5.0	15	132.3608	0.298	0.298	84	43	0.632	--	0.00	0.00
6.0	15	132.3608	0.364	0.364	84	40	0.632	--	0.00	0.00
7.0	15	132.3608	0.430	0.430	84	37	0.632	--	0.00	0.00
8.0	35	139.968	0.498	0.498	119	66	0.632	--	0.00	0.00
9.0	35	139.968	0.568	0.568	119	61	0.632	--	0.00	0.00
10.0	41	139.968	0.638	0.638	123	68	0.632	--	0.00	0.00
11.0	41	139.968	0.708	0.708	123	64	0.632	--	0.00	0.00
12.0	41	139.968	0.778	0.778	123	61	0.632	--	0.00	0.00
13.0	6	129.6464	0.846	0.846		9	0.632	~	0.00	0.00
14.0	6	129.6464	0.910	0.910		9	0.632	~	0.00	0.00
15.0	6	129.6464	0.975	0.975		9	0.632	~	0.00	0.00
16.0	6	129.6464	1.040	1.040		8	0.632	~	0.00	0.00
17.0	6	129.6464	1.105	1.089		8	0.641	~	0.00	0.00
18.0	11	128.3194	1.169	1.123		16	0.658	~	0.00	0.00
19.0	11	128.3194	1.234	1.156		16	0.674	~	0.00	0.00
20.0	11	128.3194	1.298	1.188		16	0.690	~	0.00	0.00
21.5	11	128.3194	1.378	1.230		16	0.708	~	0.00	0.00
22.0	14	122.1536	1.409	1.245		21	0.715	~	0.00	0.00
23.0	14	122.1536	1.486	1.283		21	0.732	~	0.00	0.00
24.0	14	122.1536	1.547	1.313		21	0.744	~	0.00	0.00
25.0	14	122.1536	1.608	1.343		20	0.757	~	0.00	0.00
26.0	14	122.1536	1.669	1.372		20	0.768	~	0.00	0.00
27.0	14	125.2875	1.731	1.403		20	0.779	~	0.00	0.00
28.0	40	125.2875	1.793	1.434	98	59	0.790	Non-Liq.	0.00	0.00
29.0	40	125.2875	1.856	1.466	98	58	0.800	Non-Liq.	0.00	0.00
30.0	40	125.2875	1.919	1.497	98	58	0.810	Non-Liq.	0.00	0.00
31.0	40	125.2875	1.981	1.529	98	57	0.819	Non-Liq.	0.00	0.00
32.0	40	129.5382	2.045	1.561	98	56	0.827	Non-Liq.	0.00	0.00
33.0	40	129.5382	2.110	1.595	98	56	0.836	Non-Liq.	0.00	0.00
34.0	40	129.5382	2.174	1.628	98	55	0.844	Non-Liq.	0.00	0.00
35.0	51	129.5382	2.239	1.662	108	70	0.851	Non-Liq.	0.00	0.00
36.0	51	129.5382	2.304	1.696	108	69	0.858	Non-Liq.	0.00	0.00
37.0	51	129.5382	2.369	1.729	108	68	0.865	Non-Liq.	0.00	0.00
38.0	51	129.5382	2.434	1.763	108	68	0.872	Non-Liq.	0.00	0.00
39.0	51	129.5382	2.498	1.796	108	67	0.879	Non-Liq.	0.00	0.00
40.0	50	129.5382	2.563	1.830	103	65	0.885	Non-Liq.	0.00	0.00
41.0	50	129.5382	2.628	1.863	103	65	0.891	Non-Liq.	0.00	0.00
42.0	50	129.5382	2.693	1.897	103	64	0.897	Non-Liq.	0.00	0.00
43.0	50	129.5382	2.757	1.931	103	63	0.902	Non-Liq.	0.00	0.00
44.0	50	129.5382	2.822	1.964	103	63	0.908	Non-Liq.	0.00	0.00
45.0	54	129.5382	2.887	1.998	104	67	0.913	Non-Liq.	0.00	0.00
46.0	54	129.5382	2.952	2.031	104	67	0.918	Non-Liq.	0.00	0.00
47.0	54	129.5382	3.016	2.065	104	66	0.923	Non-Liq.	0.00	0.00
48.0	54	129.5382	3.081	2.098	104	66	0.928	Non-Liq.	0.00	0.00
49.0	54	129.5382	3.146	2.132	104	65	0.932	Non-Liq.	0.00	0.00
50.0	50	129.5382	3.211	2.166	98	60	0.937	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.0 INCHES

Figure 8



NO SCALE

GEOCON WEST, INC.

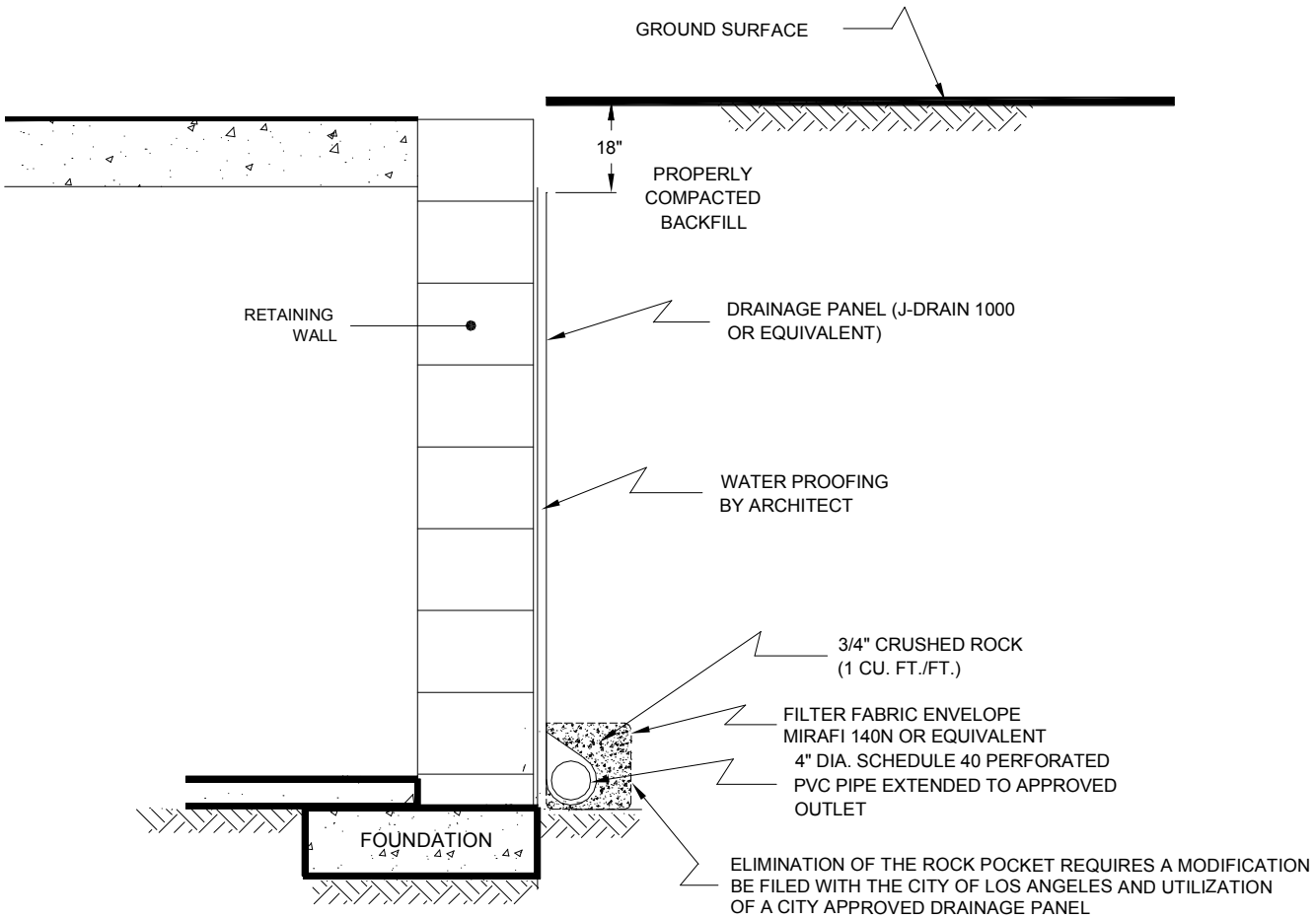
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: PZ	CHECKED BY: HHD

RETAINING WALL DRAIN DETAIL

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018	PROJECT NO. A9733-06-01	FIG. 9



NO SCALE

GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: PZ	CHECKED BY: HHD

RETAINING WALL DRAIN DETAIL

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018	PROJECT NO. A9733-06-01	FIG. 10

APPENDIX A

PROJECT NO. A9733-06-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**GRAVEL** **ARTIFICIAL FILL** Silty Sand, loose, dry, brown, fine- to medium-grained, some coarse-grained, some gravel (to 1"), some brick and asphalt fragments.			
2	B1@2.5'			SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained, some coarse-grained.	51	115.7	14.4
4	B1@5'				- trace coarse-grained	15	--	--
6								
8	B1@7.5'			ML	Sandy Silt, hard, slightly moist, dark brown, fine- to medium-grained, some coarse-grained.	63	121.5	15.2
10	B1@10'			SM	Silty Sand, dense, slightly moist, dark brown, fine- to medium-grained, some coarse-grained, some gravel (to 0.5").	41	--	--
12	B1@12.5'				Silt, firm, slightly moist, brown, trace fine- to coarse-grained sand.	20	108.4	19.6
14								
16	B1@15'		▼	ML	- soft, wet, fine- to medium-grained, some coarse-grained	6	--	--
18	B1@17.5'				- firm, light brown to yellowish brown	15	103.4	24.1
20	B1@20'				- saturated, yellowish brown	11	--	--
22	B1@22.5'				**PUENTE FORMATION** Interbedded Siltstone and Sandstone, yellowish brown, fine- to medium-grained, thinly bedded, soft, highly weathered, heavy oxidation.	24	94.4	29.4
24								
26	B1@25'					14	--	--
28	B1@27.5'				- moderately hard	43	97.5	28.5

A9733-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 1 of 2

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 1** ELEV. (MSL.) -- DATE COMPLETED **3/6/18** EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH** MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30'					40	--	--
32	B1@32.5'				- hard	50 (5")	103.3	25.4
34								
36	B1@35'					51	--	--
38								
40	B1@40'					50 (6")	--	--
42								
44								
46	B1@45'					54	--	--
48								
50	B1@50'					50 (4")	--	--
52								
54								
56					Total depth of boring: 56 feet Fill to 1.5 feet. Groundwater encountered at 17 feet. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

A9733-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 2 of 2

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

BORING 2

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**GRAVEL** **ARTIFICIAL FILL** Silty Sand, loose, dry, brown, fine- to medium-grained, some coarse-grained, some gravel (to 1"), fragments of brick and asphalt.			
2	B2@2'					65	101.2	15.3
4					**ALLUVIUM** Silty Sand, dense, slightly moist, brown, fine- to medium-grained, some coarse-grained.			
6	B2@5'			SM	- decrease in medium- to coarse-grained	65	103.9	12.9
8	B2@7'				- medium dense, increase in silt content	49	112.7	20.8
10	B2@10'			ML	Sandy Silt, stiff, slightly moist, brown, fine- to medium-grained.	33	109.7	18.8
12	B2@12'				- some coarse-grained, increase in sand content	34	100.9	21.1
14	B2@15'			SM	Silty Sand, dense, slightly moist, light brown, fine- to medium-grained, some coarse-grained.	57	110.1	15.7

Total depth of boring: 15.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9733-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

BORING 3

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			**GRAVEL**			
			ARTIFICIAL FILL			
2	B3@2'		Silty Sand, loose, dry, brown, fine- to medium-grained, some coarse-grained, some gravel (to 1").	20	104.6	19.2
		ML	**ALLUVIUM**			
4			Sandy Silt, firm, slightly moist, dark brown, fine- to coarse-grained.			
	B3@5'		- stiff, increase in sand content	28	101.4	15.8
6			Silty Sand, medium dense, slightly moist, light brown, fine-grained, some medium-grained, trace gravel (to 0.5").			
	B3@7'			28	111.9	16.4
8		SM				
10	B3@10'		- some oxidation staining	39	109.5	20.1
12	B3@12'		Sandy Silt, stiff, slightly moist, olive brown, fine-grained, trace medium- to coarse-grained.	32	94.7	27.6
		ML				
14						
	B3@15'		- decrease in sand content	40	114.9	18.0

Total depth of boring: 15.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9733-06-01 BORING LOGS.GPJ

Figure A3,
Log of Boring 3, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

BORING 4

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT**			
					ARTIFICIAL FILL			
2	B4@2'			ML	Silty Sand, loose, dry, dark brown, fine- to medium-grained, some coarse-grained, some gravel (to 1").	18	104.0	16.2
					ALLUVIUM			
4					Sandy Silt, firm, slightly moist, brown, fine- to coarse-grained.			
					Silty Sand, loose, slightly moist, light yellowish brown, fine-grained.			
6	B4@5'			SM		14	112.4	19.8
	B4@7'				- some bedrock framents	14	100.8	23.7
8								
10					Sandy Silt, firm, slightly moist, dark brown, fine- to medium-grained.			
	B4@10'					17	103.7	24.5
12	B4@12'			ML	- soft, moist, some coarse-grained	8	98.7	26.7
14								
	B4@15'				- firm, brown to yellowish brown, fine- to coarse-grained	13	97.1	26.6

Total depth of boring: 15.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9733-06-01 BORING LOGS.GPJ

Figure A4,
Log of Boring 4, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

BORING 5

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ASPHALT**			
					ARTIFICIAL FILL Silty Sand, loose, dry, brown, fine- to medium-grained, some coarse-grained, some gravel (to 1").			
2	B5@2'					12	99.9	21.0
					ALLUVIUM Sandy Silt, firm, slightly moist, light brown, fine- to coarse-grained.			
4								
	B5@5'				- yellowish brown, fine- to medium-grained	14	100.9	22.1
6				ML				
	B5@7'				- dark brown	18	110.5	16.5
8								
10	B5@10'				- soft, dark yellowish brown, trace coarse-grained	9	86.7	16.8
					Sand with Silt, well-graded, medium dense, slightly moist, brown to yellowish brown, fine- to coarse-grained, some gravel (to 0.5").			
12	B5@12'			SW-SM		27	119.0	10.3
14					**PUENTE FORMATION** Interbedded Siltstone and Sandstone, yellowish brown, fine- to medium-grained, thinly bedded, soft to moderately hard, highly weathered, oxidation.			
	B5@15'					53	100.1	23.7

Total depth of boring: 15.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9733-06-01 BORING LOGS.GPJ

Figure A5,
Log of Boring 5, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

APPENDIX A

FIELD INVESTIGATION

The site was explored on March 6, 2018, by excavating five 8-inch-diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths of approximately 15½ feet and 56 feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2 3/8-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained. Standard Penetration Tests (SPTs) were performed in boring B1.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A5. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the boring logs were revised based on subsequent laboratory testing. The location of the borings are shown on Figure 2.

APPENDIX B

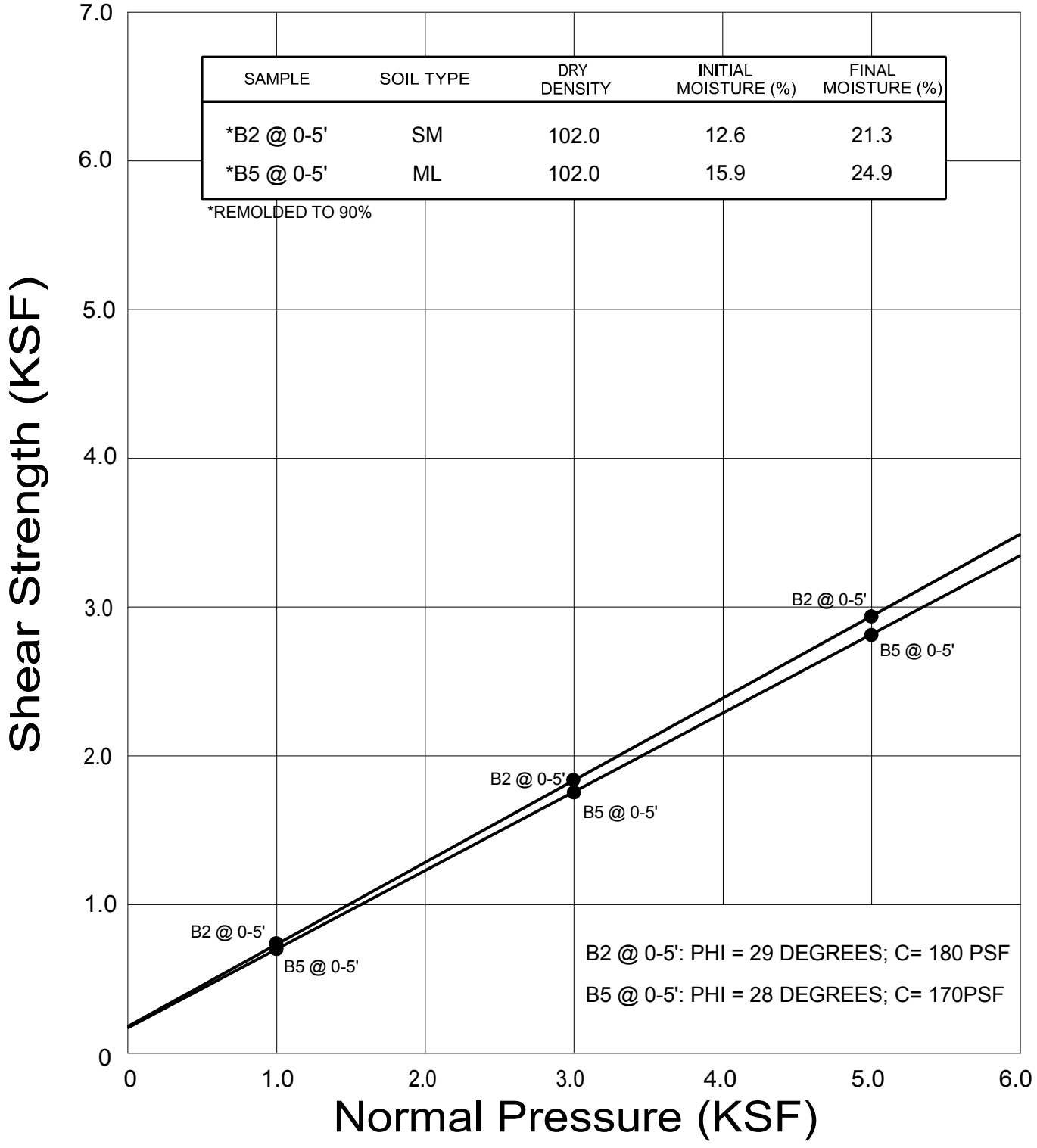
APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, plasticity indices, grain size, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B8. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.

SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
*B2 @ 0-5'	SM	102.0	12.6	21.3
*B5 @ 0-5'	ML	102.0	15.9	24.9

*REMOLDED TO 90%



● Direct Shear, Saturated

GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

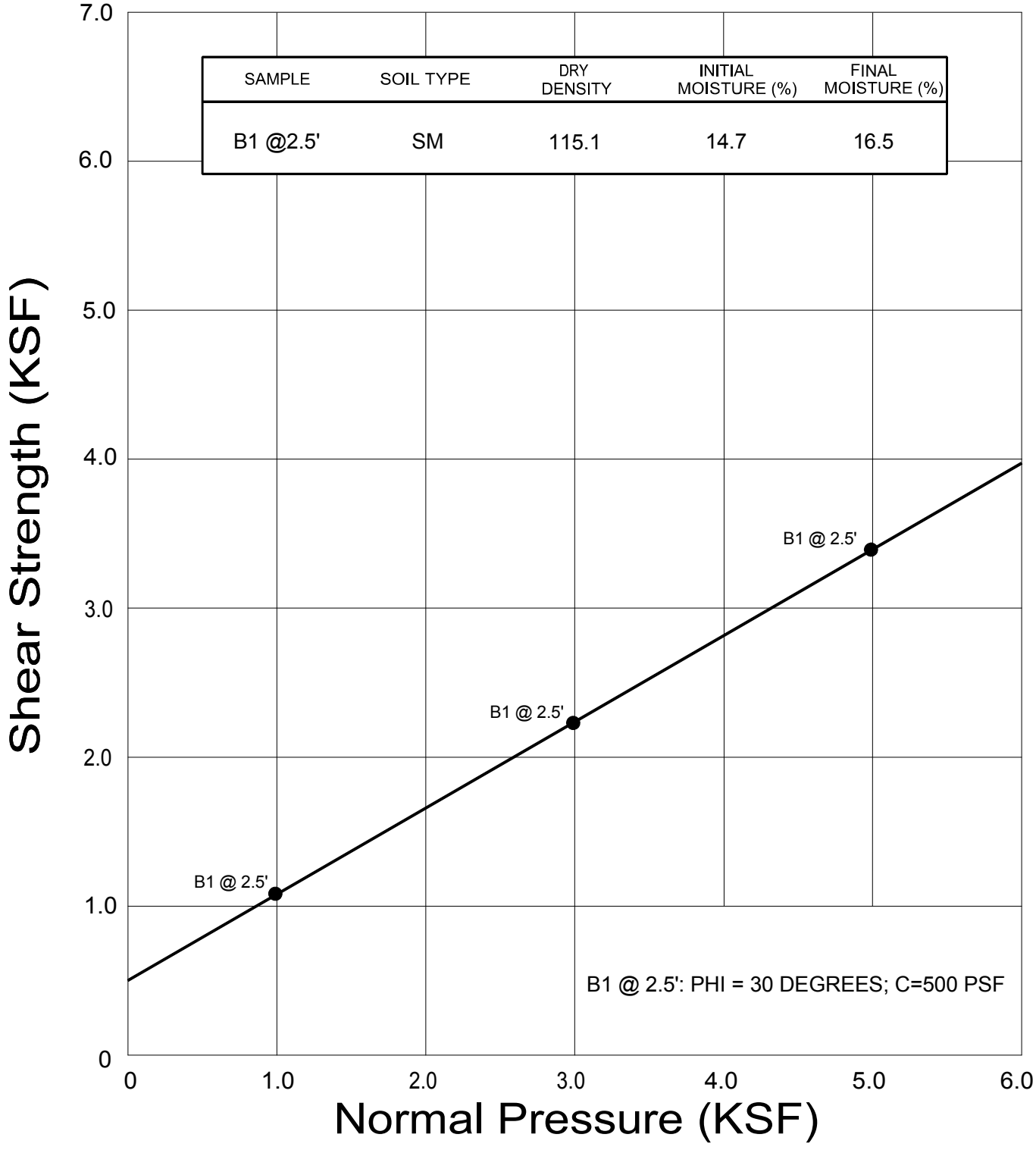
DRAFTED BY: RP | CHECKED BY: HHD

DIRECT SHEAR TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. B1

SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @2.5'	SM	115.1	14.7	16.5



● Direct Shear, Saturated

GEOCON
WEST, INC.

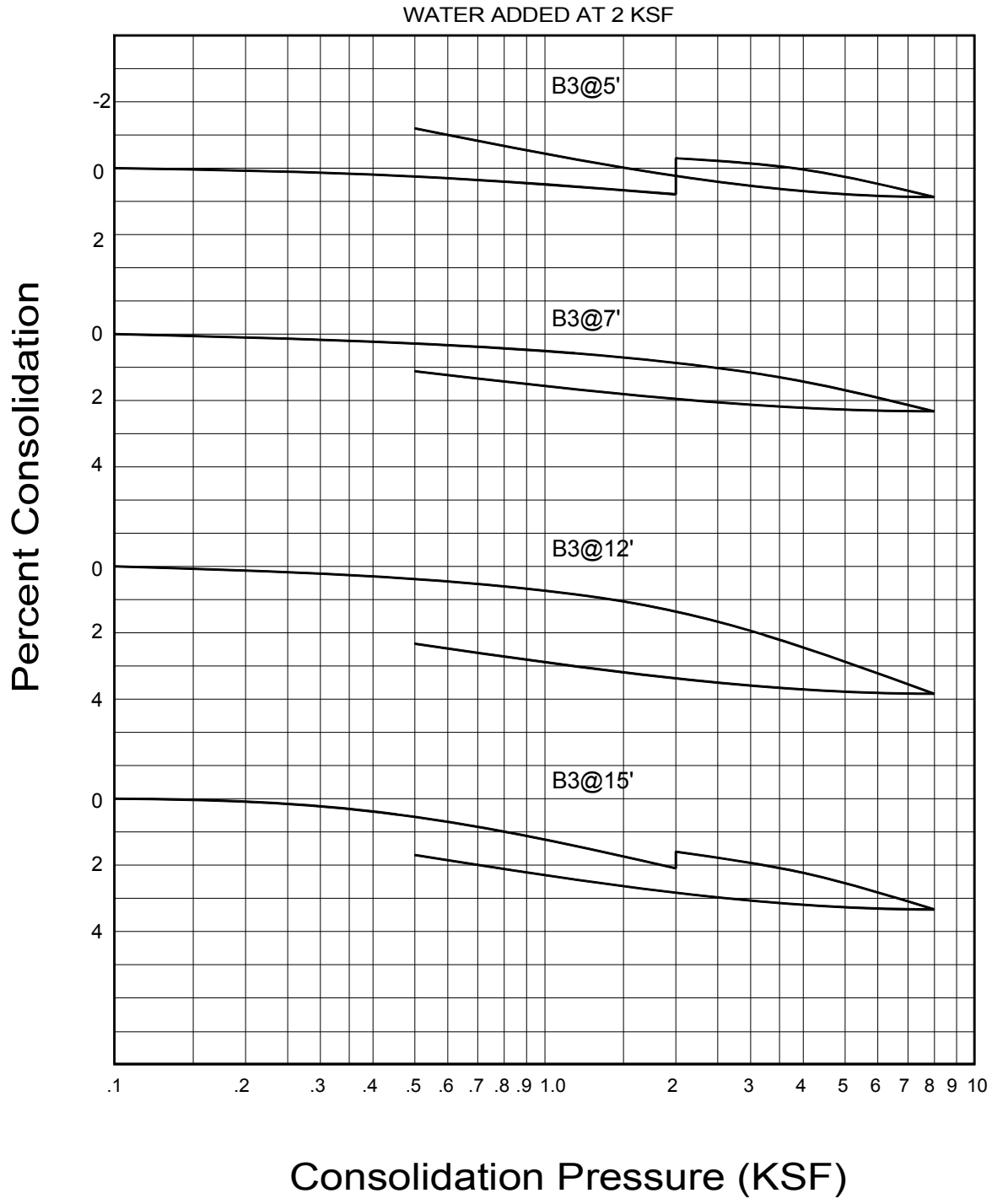
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMH | CHECKED BY: HHD

DIRECT SHEAR TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. B2



GEOCON
WEST, INC.

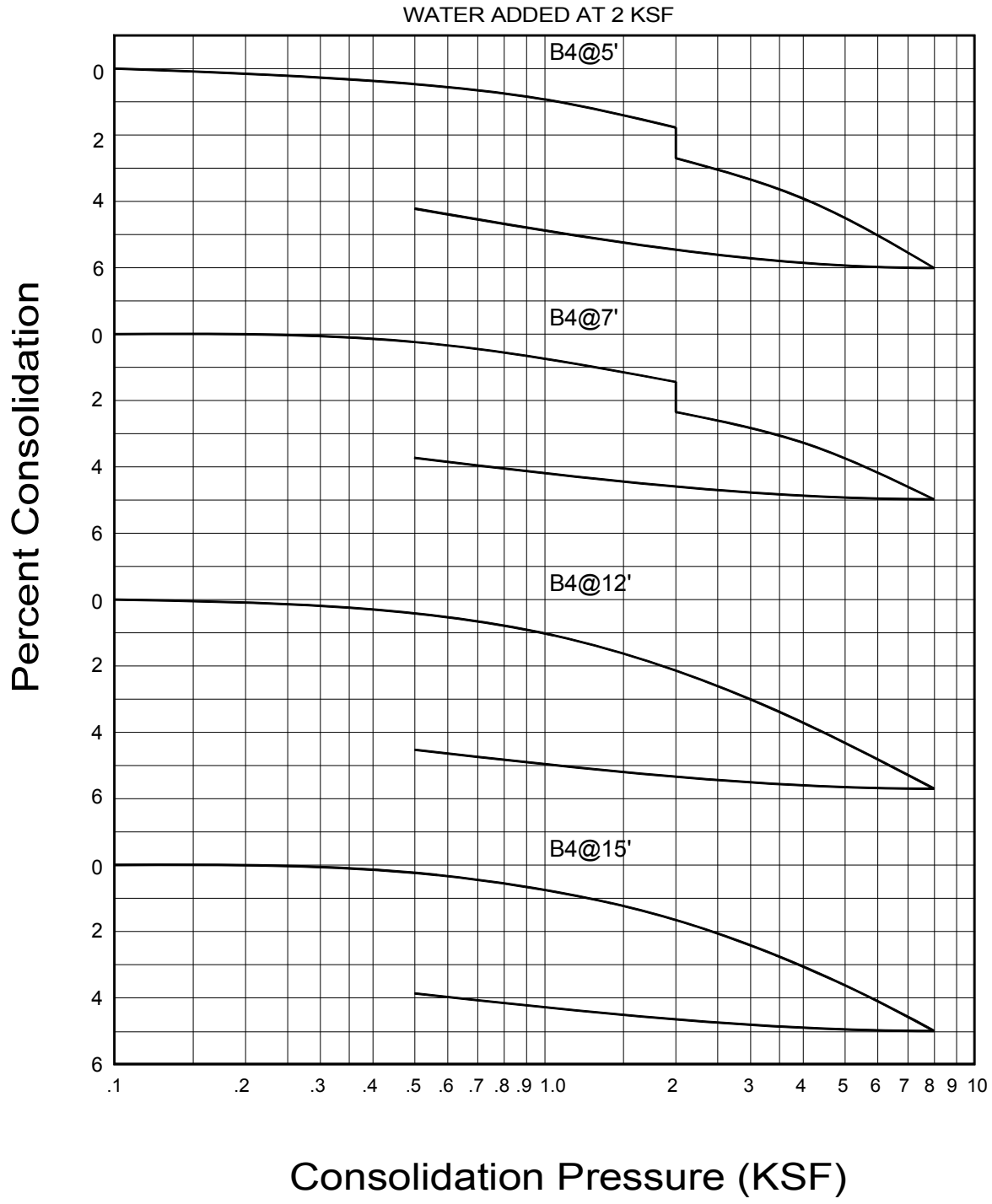
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RP	CHECKED BY: HHD

CONSOLIDATION TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018	PROJECT NO. A9733-06-01	FIG. B3



GEOCON
WEST, INC.

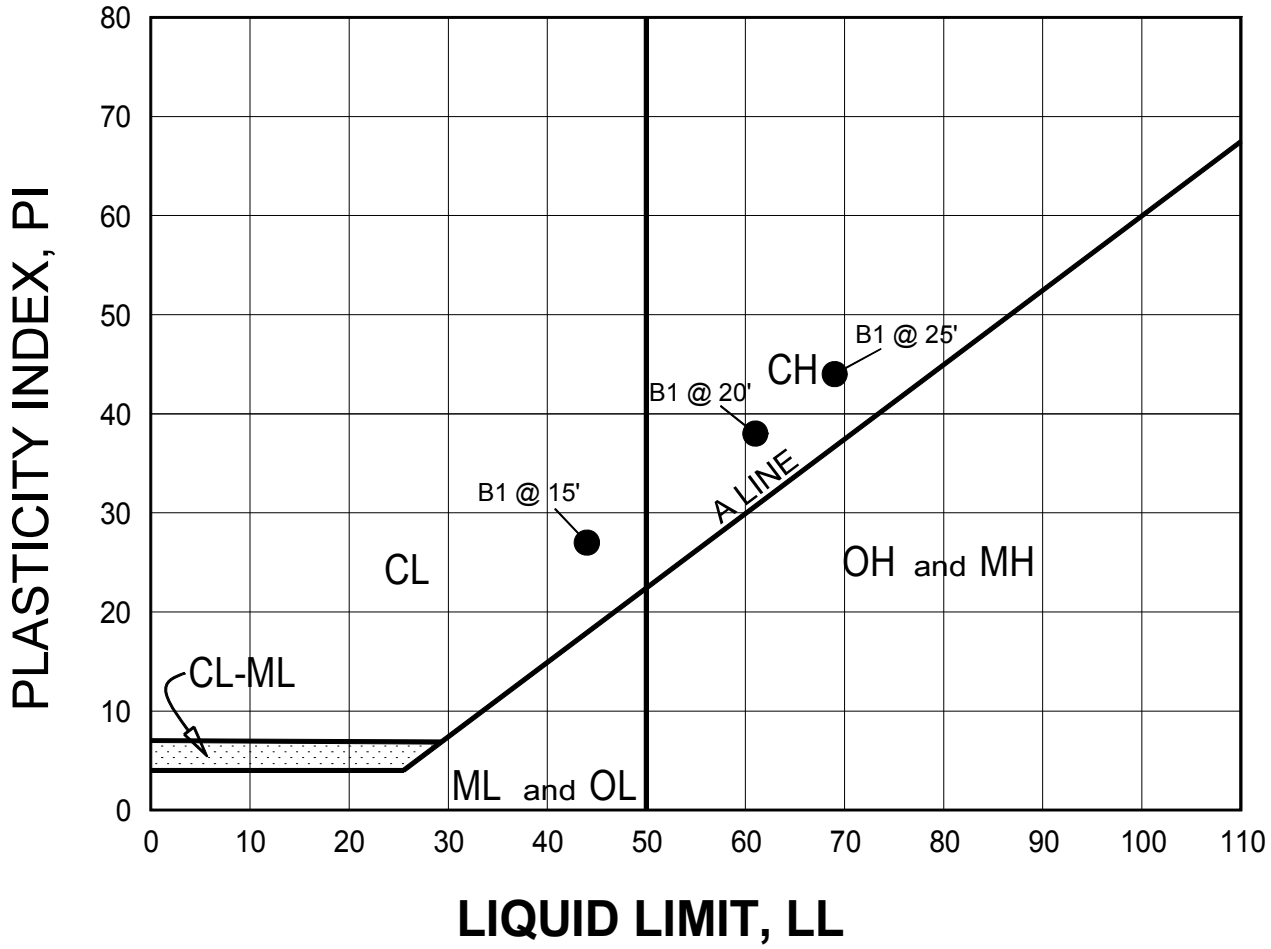
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RP	CHECKED BY: HHD

CONSOLIDATION TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018	PROJECT NO. A9733-06-01	FIG. B4



BORING NUMBER	DEPTH (FEET)	LL	PL	PI	MOISTURE CONTENT AT SATURATION (%)	SOIL BEHAVIOR
B1	5.0	24.0	18.0	6.0	---	*N/P
B1	15.0	44.0	17.0	27.0	15.1	CL
B1	20.0	61.0	23.0	38.0	16.1	CH
B1	25.0	69.0	25.0	44.0	19.1	CH

*N/P indicates Non-Plastic

GEOCON WEST, INC.

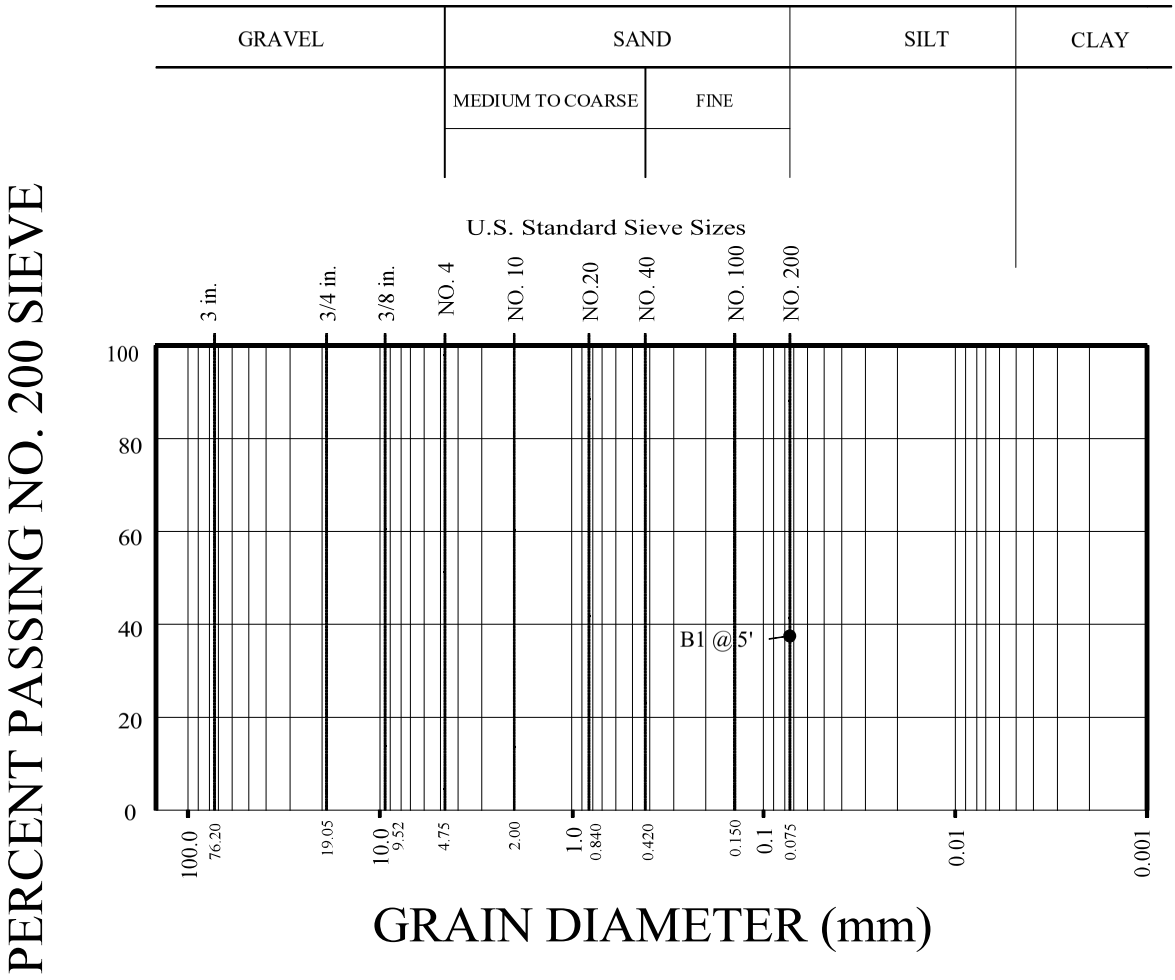
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMH | CHECKED BY: HHD

ATTERBERG LIMIT

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. B5



SAMPLE	PERCENT PASSING NO. 200 SIEVE
B1 @ 5'	37.6

GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RP	CHECKED BY: HHD

GRAIN SIZE ANALYSIS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018	PROJECT NO. A9733-06-01	FIG. B6

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829-11

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification	**CBC Classification
	Before	After				
B2 @ 0-5'	10.4	24.1	106.1	81	Moderate	Expansive
B5 @ 0-5'	13.1	29.1	100.0	101	High	Expansive

* Reference: 1997 Uniform Building Code, Table 18-I-B.

** Reference: 2016 California Building Code, Section 1803.5.3

SUMMARY OF LABORATORY MAXIMUM DENSITY AND AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-12

Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B2 @ 0-5'	Brown Silty Sand	120.0	12.0
B5 @ 0-5'	Yellowish Brown Sandy SIlt	113.0	16.0

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMH | CHECKED BY: HHD

LABORATORY TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. B7

SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS CALIFORNIA TEST NO. 643

Sample No.	pH	Resistivity (Ohm Centimeters)
B2 @ 0-5'	7.55	740 (Severely Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B2 @ 0-5'	0.013

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS CALIFORNIA TEST NO. 417

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B2 @ 0-5'	0.498	Severe

* Reference: 2016 California Building Code, Section 1904.3 and ACI 318-11 Section 4.3.

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMH | CHECKED BY: HHD

CORROSIVITY TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. B8

APPENDIX F.3

GEOTECHNICAL REPORT, LAC+USC MEDICAL CENTER, WOMEN'S AND CHILDREN'S HOSPITAL DEMOLITION PROJECT, LOS ANGELES, CALIFORNIA

PRELIMINARY GEOTECHNICAL INVESTIGATION

PROPOSED HEALTH AGENCY RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

TRACT: 2082, LOT: LT1

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR
COUNTY OF LOS ANGELES DEPARTMENT OF PUBLIC WORKS
ALHAMBRA, CALIFORNIA

PROJECT NO. A8559-06-74

MAY 20, 2019

Project No. A8559-06-74
May 20, 2019

Mr. Juan Sarda
County of Los Angeles Dept. of Public Works
900 South Freemont Avenue, 5th Floor
Alhambra, California 91803

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED LAC+USC RECUPERATIVE CARE VILLAGE
1744 ZONAL AVENUE, LOS ANGELES, CALIFORNIA
TRACT: TR 2082 LOT: LT 1

Reference: Geotechnical Investigation, Proposed Health Agency Recuperative Campus, 1774 Zonal Avenue, Los Angeles, California, Tract: TR 2082, Lot: LT 1, prepared for KFA, LP, Geocon Project Number A7933-06-01, report dated April 9, 2018.

Dear Mr. Sarda:

In accordance with your authorization of our proposal dated April 2, 2019, we have performed a preliminary geotechnical investigation for the proposed LAC+USC Recuperative Care Village (RCV) project located at 1774 Zonal Avenue in the City of Los Angeles, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed building design and construction.

This investigation is not intended to be submitted to the building official at this time. Additional investigation and analyses will be required, including the evaluation of slope stability, in order to generate a more comprehensive report in accordance with the Building Code requirements.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned.

Very truly yours,

GEOCON WEST, INC.

REGISTERED PROFESSIONAL ENGINEER
NEAL D. BERLINER
2576
GEOTECHNICAL
STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST
GERALD KASMAN
NO. 2251
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Rex Panoy
Staff Engineer

Neal D. Berliner
GE 2576

Gerald A. Kasman
CEG 2251

(EMAIL) Addressee

TABLE OF CONTENTS

1. PURPOSE AND SCOPE 1
2. SITE AND PROJECT DESCRIPTION 2
3. GEOLOGIC SETTING 3
4. SOIL AND GEOLOGIC CONDITIONS 3
   - 4.1 Artificial Fill 3
   - 4.2 Colluvium 4
   - 4.3 Alluvium 4
   - 4.4 Puente Formation 4
5. GROUNDWATER 4
6. GEOLOGIC HAZARDS 5
   - 6.1 Surface Fault Rupture 5
   - 6.2 Seismicity 6
   - 6.3 Seismic Design Criteria 6
   - 6.4 Liquefaction Potential 8
   - 6.5 Slope Stability 9
   - 6.6 Earthquake-Induced Flooding 10
   - 6.7 Tsunamis, Seiches, and Flooding 10
   - 6.8 Oil Fields & Methane Potential 10
   - 6.9 Subsidence 11
7. CONCLUSIONS AND RECOMMENDATIONS 12
   - 7.1 General 12
   - 7.2 Temporary Dewatering 15
   - 7.3 Soil and Excavation Characteristics 15
   - 7.4 Minimum Resistivity, pH, and Water-Soluble Sulfate 16
   - 7.5 Grading 17
   - 7.6 Shrinkage 20
   - 7.7 General Foundation Design 20
   - 7.8 Conventional Foundation Design – RCC and RTP Structures 21
   - 7.9 Grade Beam (Waffle-Slab) and Mat Foundation Design – RTP Structures 22
   - 7.10 Miscellaneous Foundations 24
   - 7.11 Lateral Design 24
   - 7.12 Concrete Slabs-on-Grade 24
   - 7.13 Preliminary Pavement Recommendations 26
   - 7.14 Retaining Wall and Elevator Pit Design 27
   - 7.15 Retaining Wall Drainage 28
   - 7.16 Elevator Piston 29
   - 7.17 Temporary Excavations 29
   - 7.18 Surface Drainage 30
   - 7.19 Plan Review 31

LIMITATIONS AND UNIFORMITY OF CONDITIONS

LIST OF REFERENCES

MAPS, TABLES, AND ILLUSTRATIONS
- Figure 1, Vicinity Map
- Figure 2, Site Plan
- Figure 3, Test Pit Cross Section
- Figure 4, Regional Fault Map
- Figure 5, Regional Seismicity Map
- Figures 6 and 7, Retaining Wall Drain Detail

TABLE OF CONTENTS (Continued)

APPENDIX A
FIELD INVESTIGATION
Figures A1 through A10, Boring Logs

APPENDIX B
LABORATORY TESTING
Figures B1 through B3, Direct Shear Test Results
Figures B4 and B21, Consolidation Test Results
Figure B22, Expansion Index Test Results
Figures B23 and B24, Modified Compaction Test of Soils
Figure B25, Corrosivity Test Results

APPENDIX C
GEOTECHNICAL INVESTIGATION DATED APRIL 9, 2018

GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a preliminary geotechnical investigation for the proposed Recuperative Care Village (RCV) project located at the LAC + USC medical campus located at 1744 Zonal Avenue in the City of Los Angeles, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of proposed foundation design and construction.

This investigation is not intended to be submitted to the building official at this time. Additional investigation and analyses will be required, including the evaluation of slope stability, in order to generate a more comprehensive report in accordance with the Building Code requirements.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was previously explored on March 6, 2018 (Geocon Report dated April 9, 2018) by excavating five 8-inch diameter borings to depths between approximately 15½ and 56 feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. Additional site exploration was performed on April 22 and 23, 2019, by manually excavating seven borings (B1 through B7) and one test pit using hand auger equipment and digging tools. The test pit was excavated for the purpose of verifying footing depth and dimension, as well as to verify bearing materials of the existing retaining wall bounding the Imaging Center (MRI) building and parking area. However, the retaining wall footing was not encountered in the test pit excavated to a depth of 5 feet below the parking lot surface; therefore, a series of hand auger borings were conducted within the test pit and every few feet perpendicular the the wall to find the approximate the depth of the existing footing. Additionally, on April 30, 2019, one 24-inch and two 18-inch diameter borings were drilled using a bucket auger drilling machine. The bucket auger borings were advanced to a maximum depth of 25½ feet below the existing ground surface. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A cross-section of the existing retaining wall is depicted on Figure 3. A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

Geocon previously performed a geotechnical investigation for the proposed health agency recuperative campus and prepared a report dated April 9, 2018. Logs of prior explorations and laboratory testing were considered in this investigation and are presented in Appendix C.

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The subject site consists of an irregularly shaped parcel bounded by Zonal Avenue to the north, by Circle Drive and the Women's and Children's Hospital campus to the west, by a pedestrian walkway/stairway to the south, and by North State Street to the east/southeast. The subject area of improvements currently consists of an approximate 42,000 square-foot unpaved area that is currently occupied by a temporary trailer, as well as the MRI building and parking area located at 1744 Zonal Avenue in the City of Los Angeles, California. The MRI building and parking area is situated next to an approximate 40-foot-high graded slope that ascends from the south/southeast corner of the site to the entrance drive for the University of Southern California Health Sciences Campus located east-southeast. The slope gradient is approximately 2:1 (H:V) or flatter. With the exception of the graded slope, the topography of the area of improvementsis relatively level, with no pronounced highs or lows. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to onsite drains and the city streets. Vegetation on the site consists of native grasses in localized areas.

Based on the information provided by the Client, it is our understanding that the proposed Restorative Care Village project will consist of one 4-story recuperative care center (RCC) building and four 3-story residential treatment program (RTP) structures to be constructed at or near the present grade of the parking lot, see Site Plan, Figure 2.

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structures will be up to 400 kips, and wall loads will be up to 4.5 kips per linear foot.

The scope of the proposed development has changed significantly since the time of our original proposed and authorization to prepare a geotechnical investigation report. The current design for the project now requires the demolition of the existing site retaining walls, which retain the existing 40-foot-high ascending slope bounding the southern portion of the site, and the construction of new retaining walls pushed further into the existing slope.

The demolition of existing site retaining walls and the construction of new walls requires the evaluation of the gross stability of the existing 40-foot-high ascending slope that bounds the southern portion of the site. This level of study and design was not included in our original contracted scope of services and will be required by the building official. Additional exploration and analysis of the slope stability will be required in order to provide recommendations for demolition of existing walls, as well as construction of new walls. These recommendations will be provided under a separate cover after additional investigation and analyses are performed.

3. GEOLOGIC SETTING

The site is located in the northeastern portion of the Los Angeles Basin, along the southwestern flank of the Repetto Hills. The Los Angeles Basins is a coastal plain bounded by the Santa Monica Mountains on the north, the Elysian Hills and Repetto Hills on the northeast, the Puente Hills and Whittier Fault on the east, the Palos Verdes Peninsula and Pacific Ocean on the west and south, and the Santa Ana Mountains and San Joaquin Hills on the southeast. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits underlain by a basement complex of igneous and metamorphic composition. Regionally, the site is located within the northern portion of the Peninsular Ranges geomorphic province. This geomorphic province is characterized by northwest-trending physiographic and geologic features such as the Newport-Inglewood Fault Zone and the Whittier Fault Zone located approximately 9.5 miles to the southwest and the 9.8 miles to the southeast.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Quaternary age alluvial deposits consisting of sand, silt, and gravel (Dibblee, 1989; California Geological Survey, 2012). The Quaternary age sediments are in turn underlain by sedimentary bedrock of the Miocene age Puente Formation (Lamar, 1970; Dibblee, 1989). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A and Appendix C.

4.1 Artificial Fill

Artificial fill was encountered in all current and prior field explorations to a maximum depth of 10 feet below existing ground surface. The artificial fill generally consists of brown to dark brown, fine- to medium-grained silt with clay and silty sand with some fine to coarse gravel. The artificial fill is characterized as dry to slightly moist and loose or firm. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Colluvium

Colluvial deposits were encountered underlying the artificial fill in borings B-7 through B-10. The colluvium is derived from in-place weathering of the underlying bedrock materials and subject to slow downhill movement due to gravitational forces. The colluvium generally consists of fine-grained clay and silt and can be characterized as moist and firm to stiff.

4.3 Alluvium

Quaternary age alluvium was encountered in all previous borings beneath the fill in the northwestern portion of the property. The alluvium generally consists of light yellowish brown to dark brown silty sand and sandy silt. The alluvial soils are primarily fine- to medium-grained, slightly moist to wet and medium dense to dense or soft to hard.

4.4 Puente Formation

Sedimentary bedrock units of the Miocene age Puente Formation was encountered in prior borings B1 and B5 and borings B-4, B-5, and B7 through B-10 of the current investigation at depths of 10 to 20 feet beneath the existing ground surface. The depth to bedrock, where encountered in our site explorations, is indicated on Figure 2. The bedrock consists of siltstone and fine-grained sandstone ranges from poorly- to well-bedded, thinly bedded to massive, soft to hard, moderately weathered to completely weathered.

5. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Los Angeles Quadrangle (California Division of Mines and Geology [CDMG], 1998) indicates the historically highest groundwater level in the area is approximately 20 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was encountered in several of our borings at depths between 10 and 18 feet beneath the existing ground surface. The depth to groundwater, where encountered in our site explorations, is indicated on Figure 2. Based on the depth to groundwater encountered in our borings and the depth of proposed construction, groundwater will likely be encountered during construction if excavations exceed 10 feet in depth below the ground surface. It is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.17).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (CGS, 2018a). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2018b; CGS, 2017) for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Raymond Fault located approximately 4.0 miles to the north (Ziony and Jones, 1989). Other nearby active faults include the Hollywood Fault, the Verdugo Fault, the Newport-Inglewood Fault Zone, and the Whittier Fault Zone located approximately 4.2 miles north-northwest, 5.6 miles north, 9.5 miles southwest, and 9.8 miles southeast of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 33 miles north of the site.

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987 M_w 5.9 Whittier Narrows earthquake and the January 17, 1994 M_w 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. These thrust faults and others in the Los Angeles area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 4, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Near Redlands	July 23, 1923	6.3	55	E
Long Beach	March 10, 1933	6.4	34	SSE
Tehachapi	July 21, 1952	7.5	79	NW
San Fernando	February 9, 1971	6.6	26	NNW
Whittier Narrows	October 1, 1987	5.9	7	E
Sierra Madre	June 28, 1991	5.8	18	NE
Landers	June 28, 1992	7.3	102	E
Big Bear	June 28, 1992	6.4	80	E
Northridge	January 17, 1994	6.7	21	WNW
Hector Mine	October 16, 1999	7.1	117	ENE

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in Southern California and the effects of ground shaking can be mitigated if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

6.3 Seismic Design Criteria

The following table summarizes summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the computer program *U.S. Seismic Design Maps*, provided by the USGS. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented on the following page are for the risk-targeted maximum considered earthquake (MCE_R).

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.556g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.891g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.556g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.336g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.704g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.891g	Section 1613.3.4 (Eqn 16-40)

The table below presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10.

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.972g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.972g	Section 11.8.3 (Eqn 11.8-1)

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2016 California Building Code and ASCE 7-10, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain "Life Safety" during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2008 Conterminous U.S. Dynamic edition. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.92 magnitude event occurring at a hypocentral distance of 8.02 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.79 magnitude occurring at a hypocentral distance of 11.3 kilometers from the site.

Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

6.4 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

The current standard of practice, as outlined in the "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California" and "Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California" requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

The State of California Seismic Hazard Zone Map for the Los Angeles Quadrangle (CDMG, 1999) indicates that the site is located in an area identified as having a potential for liquefaction. Also, according to the Los Angeles County Safety Element (1990), the site is located within an area identified as having a potential for liquefaction.

Liquefaction analysis of the soils underlying the site was performed using an updated version of the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance and field performance data.

Screening criteria developed by Bray and Sancio (2006) characterize fine-grained soils which are not susceptible to liquefaction as soils with a plasticity index (PI) that is greater than 18 or with a saturated moisture content that is less than 80 percent of the liquid limit. In order to apply the screening criteria, laboratory testing was performed to evaluate the Atterberg Limits and saturated moisture content of select soil samples. Laboratory test results used for the screening criteria are presented as Figure B5.

The liquefaction analysis performed during the previous investigation was performed for a Design Earthquake level by using a historic high groundwater table of 17 feet below the ground surface, a magnitude 6.64 earthquake, and a peak horizontal acceleration of 0.648g ($⅔PGA_M$). The enclosed liquefaction analysis, included herein for prior boring B1, indicates that the alluvial soils below the historic high groundwater level would not be susceptible to liquefaction settlement during Design Earthquake ground motion (see enclosed calculation sheets, Figures 5 and 6 in Appendix C).

It is our understanding that the intent of the Building Code is to maintain "Life Safety" during Maximum Considered Earthquake level events. Therefore, additional analysis was performed to evaluate the potential for liquefaction during a MCE event. The structural engineer should evaluate the proposed structure for the anticipated MCE liquefaction induced settlements and verify that anticipated deformations would not cause the foundation system to lose the ability to support the gravity loads and/or cause collapse of the structure.

The liquefaction analysis was also performed for the Maximum Considered Earthquake level by using a historic high groundwater table of 17 feet below the ground surface, a magnitude 6.63 earthquake, and a peak horizontal acceleration of 0.972g (PGA_M). The enclosed liquefaction analysis, included herein for boring B1, indicates that the alluvial soils below the historic high groundwater level would not be susceptible to liquefaction settlement during Maximum Considered Earthquake ground motion (see Appendix C enclosed calculation sheets, Figures 7 and 8).

6.5 Slope Stability

The topography at the site is relatively level and the topography in the immediate site vicinity slopes gently to the west or southwest. The site is located within a City of Los Angeles Hillside Grading Area and a Hillside Ordinance Area (City of Los Angeles, 2018). However, the site and adjacent sites are not located within an area identified as having a potential for seismic slope instability (CDMG, 1999; City of Los Angeles, 2018). As previously indicated, an approximately 40-foot-high, 2:1 (H:V gradient) graded slope ascends from the southeast corner of the site. Based on the 2:1 gradient of the slope, it is not considered a stability hazard and is not anticipated to adversely impact the site. There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The Los Angeles County Safety Element (Leighton, 1990) indicates that the site is located within the Ascot Reservoir inundation area. However, this reservoir, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design, construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

6.7 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area. Therefore, tsunamis are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Therefore, flooding resulting from a seismically-induced seiche is considered unlikely.

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2019; LACDPW, 2019b). Therefore, flooding is not anticipated to adversely impact the site.

6.8 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Well Finder Website, the site is not located within the limits of an oilfield and oil or gas wells are not located within 1,200 feet of the site. However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the DOGGR.

The site is located within the boundaries of a city-designated Methane Zone (City of Los Angeles, 2018). Therefore, there is a potential for methane or other volatile gases to occur at the site. The City of Los Angeles will require a methane study be performed for the proposed development. It is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 This investigation is not intended to be submitted to the building official at this time. Additional investigation and analyses will be required, including the evaluation of slope stability, in order to generate a more comprehensive report in accordance with the Building Code requirements. The purpose of this report was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of proposed foundation design and construction.

7.1.2 The demolition of existing site retaining walls and the construction of new walls requires the evaluation of the gross stability of the existing 40-foot-high ascending slope that bounds the southern portion of the site. A shoring system will be required in order to demolish the existing walls and the orientation of the bedrock in the slope will have to be characterized and analyzed. This level of study and design was not included in our original contracted scope of services. Additional exploration and analysis of the slope stability will be required in order to provide recommendations for demolition of existing walls, as well as construction of new walls. These recommendations will be provided under a separate cover after additional investigation and analyses are performed.

7.1.3 The enclosed seismically-induced settlement analyses performed during the previous investigation dated April 9, 2018, indicate that the alluvial soils underlying the site would not be susceptible to liquefaction as a result of the Design Earthquake peak ground acceleration ($\frac{2}{3}PGA_M$).

7.1.4 During the site explorations, up to 1½ feet of existing artificial fill was encountered within the boundary of the proposed RCC structure. Additionally, approximately 10 feet of existing artificial fill was encountered within the footprint areas of the proposed RTP structures. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. Demolition of existing structures and improvements will likely disturb the upper site soils. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.4).

7.1.5 Laboratory test results indicate that the existing fill within the footprint areas of the proposed RTP structures is marginally compacted. A comparison of the in-situ density of the existing fill with the laboratory maximum dry density indicates that the majority of the exiting fill is below the required 90 percent relative compaction.

7.1.6 Provided that all existing fill is excavated and properly compacted in the building pad area, a conventional foundation system deriving support in the newly placed engineered fill may be utilized for support of the proposed RCC and RTP structures. As a minimum, foundations for the proposed RCC and RTP structures should be underlain by at least 3 feet of newly placed engineered fill. Excavations should be conducted as necessary to completely remove all artificial fill at the direction of the Geotechnical Engineer (a representative of Geocon). Where excavation and compaction are to be conducted, the excavation should extend laterally a minimum distance of 3 feet beyond the building footprint area or for a distance equal to the depth of fill below the foundation, whichever is greater. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.4).

7.1.7 As an alternative to removing all existing fill, the proposed RTP structures may be supported on a reinforced concrete grade beam foundation system or a reinforced concrete mat foundation system deriving support on a minimum of 5 feet of newly placed engineered fill. The use of the structural foundation system precludes the need for the complete removal of all existing artificial fill in the building footprint area. The 5-foot-thick engineered fill blanket in combination with the structural foundation system is capable of minimizing the affects if differential settlement on the proposed structures. Recommendations for the design of grade beam and mat foundation systems are provided in Sections 7.8 and 7.9, respectively.

7.1.8 The optimum moisture content of the upper site soils varies from approximately 14 to 26 percent. Currently, site soils are generally above the optimum moisture content and will require spreading and drying efforts in order to achieve required compaction. The contractor should be prepared for this as it will add a time delay to grading activities. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.4).

7.1.9 Groundwater was encountered in several of our borings at depths between 10 and 18 feet beneath the existing ground surface. The depth to groundwater, where encountered in our site explorations, is indicated on Figure 2. Based on the depth to groundwater encountered in our borings and the depth of proposed construction, groundwater will likely be encountered during construction if excavations exceed 10 feet in depth below the ground surface. If groundwater is encountered, temporary dewatering measures and subgrade stabilization will be required during excavation and construction. Recommendations for temporary dewatering are discussed in Section 7.2 of this report.

7.1.10 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the upper 12 inches of the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.1.11 It is anticipated that stable excavations for the recommended grading associated with the proposed structure can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.16).

7.1.12 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

7.1.13 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft alluvial soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of subgrade soil should be scarified and properly compacted for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.13).

7.1.14 Due to the shallow existing groundwater elevation at the site a stormwater infiltration system is not recommended for this development. It is suggested that stormwater be retained, filtered and discharged in accordance with the requirements of the local governing agency.

7.1.15 Once the design and foundation loading configuration for the proposed structure proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be re-evaluated by this office.

7.1.16 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

7.2 Temporary Dewatering

7.2.1 Groundwater was encountered in several of our borings at depths between 10 and 18 feet beneath the existing ground surface. Based on the conditions encountered at the time of exploration, it is anticipated that groundwater will be encountered during excavations and construction activities that extend to these depths. If groundwater is present, temporary dewatering will be necessary to maintain a safe working environment during excavation and construction activities.

7.2.2 It is recommended that a qualified dewatering consultant be retained to design the temporary dewatering system and determine the design flow rates for dewatering. Temporary dewatering may consist of perimeter wells with interior well points, as well as gravel filled trenches (French drains) that are strategically placed to intercept and control water. The number and locations of the wells or French drains can be adjusted during excavation activities as necessary to collect and control any encountered seepage. The French drains will then direct the collected seepage to a sump where it will be pumped out of the excavation.

7.2.3 If a temporary French drain is to remain on a permanent basis, it must be lined with filter fabric to prevent soil migration into the gravel.

7.3 Soil and Excavation Characteristics

7.3.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular soils are encountered.

7.3.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of existing adjacent improvements.

7.3.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.16).

7.3.4 The upper 5 feet of existing site soils encountered during this investigation are considered to have a "moderate" to "high" expansive potential (EI = 62, 81, & 101); and are classified as "expansive" based on the 2016 California Building Code (CBC) Section 1803.5.3. Recommendations presented herein assume that the building foundations and slabs will derive support in these materials.

7.4 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.4.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "corrosive to severely corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B8) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that ABS pipes be considered in lieu of cast-iron for subdrains and retaining wall drains in direct contact with the site soils.

7.4.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B8), as well as from the previous investigation (Appendix C), indicate that the on-site materials possess "negligible (S0) to severe (S2)" sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3. *The table below presents a summary of concrete requirements set forth by 2016 CBC Section 1904 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.*

REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Exposure	Exposure Class	Water-Soluble Sulfate Percent by Weight	Cement Type (ASTM C150)	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
Not Applicable	S0	SO_4<0.10	--	--	2,500
Moderate	S1	0.10≤SO_4<0.20	II	0.50	4,000
Severe	**S2**	**0.20≤SO_4≤2.00**	**V**	**0.45**	**4,500**
Very Severe	S3	SO_4>2.00	V+Pozzolan or Slag	0.45	4,500

7.4.3 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

7.5 Grading

7.5.1 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and alluvial soil encountered during exploration is suitable for re-use as engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris are removed.

7.5.2 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer, geotechnical engineer, and building official in attendance. Special soil handling requirements can be discussed at that time.

7.5.3 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.5.4 Where a conventional spread foundation system will be utilized, as a minimum, it is recommended that the upper 5 feet of existing earth materials within the proposed RCC and RTP building footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as necessary to remove all existing artificial fill or soft alluvial soil at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint area, including building appurtenances, or a distance equal to the depth of engineered fill below the foundation, whichever is greater. Proposed building foundations should be underlain by a minimum of 3 feet of newly placed engineered fill. The limits of existing fill and/or soft alluvial soils removal will be verified by the Geocon representative during site grading activities.

7.5.5 As an alternative to removing all existing fill, the proposed RTP structures may be supported on a reinforced concrete grade beam foundation system or a reinforced concrete mat foundation system deriving support on a minimum of 5 feet of newly placed engineered fill. The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of engineered fill below the foundation, whichever is greater. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities.

7.5.6 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the upper 12 inches of the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.5.7 The optimum moisture content of the upper site soils varies from approximately 14 to 26 percent. The grading contractor should be aware that currently, site soils are generally above the optimum moisture content and will require spreading and drying efforts in order to achieve required compaction.

7.5.8 Due to the potential for high-moisture content soils at the excavation bottom, or if construction is performed during the rainy season and the excavation bottom becomes saturated, stabilization measures may have to be implemented to prevent excessive disturbance to the excavation bottom. Should this condition exist, rubber tire equipment should not be allowed in the excavation bottom until it is stabilized, or extensive soil disturbance could result. Track mounted equipment should be considered to minimize disturbance to the soils.

7.5.9 If the excavation bottom is soft, subgrade stabilization may be performed by introducing a thin lift of 3 to 6-inch diameter crushed angular rock into the soft excavation bottom. The use of crushed concrete will also be acceptable. The crushed rock should be spread thinly across the excavation bottom and pressed into the soils by drum rolling, track rolling or wheel rolling. It is very important that voids between the rock fragments are not created so the rock must be thoroughly pressed or blended into the soils. All subgrade soils must be properly compacted and proof-rolled in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.5.10 If a temporary French drain is used for dewatering purposes and is to remain on a permanent basis or be abandoned in-place, it must be lined and covered with filter fabric to prevent soil migration into the gravel.

7.5.11 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to at least 2 percent above optimum moisture content, and properly compacted to a minimum of 90 percent of the maximum dry density per ASTM D 1557 (latest edition).

7.5.12. Where new paving is to be placed, it is recommended that all existing fill and soft alluvium be excavated and properly compacted for paving support. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to 2 percent above optimum moisture content, and compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.13).

7.5.13 It is anticipated that stable excavations for the recommended grading can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of the existing offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.16).

7.5.14 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

7.5.15 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry is also acceptable as backfill. Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

7.5.16 All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 50 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B8 and Appendix C). Import soils placed in the building area should be placed uniformly across the building pad or in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).

7.5.17 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding materials, fill, steel, gravel, or concrete.

7.6 Shrinkage

7.6.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 10 and 15 percent should be anticipated when excavating and compacting the upper 5 feet of existing earth materials on the site to an average relative compaction of 92 percent.

7.4.2 If import soils will be utilized in the building pad, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.). Soils can be borrowed from non-building pad areas and later replaced with imported soils.

7.7 General Foundation Design

7.7.1 Provided that all existing fill is excavated and properly compacted in the building pad area, a conventional foundation system deriving support in the newly placed engineered fill may be utilized for support of the proposed RCC and RTP structures. As a minimum, foundations for the proposed RCC and RTP structures should be underlain by at least 3 feet of newly placed engineered fill.

7.7.2 As an alternative to removing all existing fill, the proposed RTP structures may be supported on a reinforced concrete grade beam foundation system or a reinforced concrete mat foundation system deriving support on a minimum of 5 feet of newly placed engineered fill. The 5-foot-thick engineered fill blanket in combination with the structural foundation system is capable of minimizing the affects if differential settlement on the proposed structures.

7.7.3 Due to the expansive potential of the subgrade soils, the moisture content foundation subgrade should be maintained at 2 percent above optimum moisture content prior to and at the time of concrete placement.

7.7.4 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

7.7.5 This office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

7.8 Conventional Foundation Design – RCC and RTP Structures

7.8.1 Subsequent to the recommended grading, a conventional shallow spread foundation system may be utilized for support of the proposed RCC and RTP structures.

7.8.2 Continuous footings may be designed for an allowable bearing capacity of 2,200 pounds per square foot (psf), and should be a minimum of 12 inches in width, 30 inches in depth below the lowest adjacent grade, and 12 inches into the newly placed engineered fill.

7.8.3 Isolated spread foundations may be designed for an allowable bearing capacity of 2,600 psf, and should be a minimum of 24 inches in width, 30 inches in depth below the lowest adjacent grade, and 12 inches into the newly placed engineered fill.

7.8.4 The allowable soil bearing pressure above may be increased by 200 psf and 400 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 4,000 psf.

7.8.5 The allowable bearing pressures may be increased by one-third for transient loads due to wind or seismic forces.

7.8.6 If depth increases are utilized for the perimeter foundations, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary. Additional grading should be conducted as-needed in order to maintain the required 3-foot-thick blanket of engineered fill below proposed foundations.

7.8.7 Continuous footings should be reinforced with four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. Reinforcement for spread footings should be designed by the project structural engineer.

7.8.8 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only, and are not intended to be used in lieu of those required for structural purposes.

7.8.9 The maximum expected static settlement for a structure supported on a conventional foundation system deriving support in the newly placed engineered fill and designed with a maximum bearing pressure of 4,000 psf is estimated to be less than 1 inch and occur below the heaviest loaded structural element. A majority of the settlement of the foundation system is expected to occur on initial application of loading with minor additional settlements within the first 12 months. Differential settlement is not expected to exceed ¾ inch over a distance of 20 feet.

7.9 Grade Beam (Waffle-Slab) and Mat Foundation Design – RTP Structures

7.9.1 As an alternative to removing all existing fill, the proposed RTP structures may be supported on a reinforced concrete grade beam foundation system or a reinforced concrete mat foundation system deriving support on a minimum of 5 feet of newly placed engineered fill.

7.9.2 The grade beam foundation system consists of a continuous perimeter reinforced concrete grade beam foundation, that is interconnected with interior grade beams and a concrete slab. The system of grade beams, in conjunction with the slab, provides a stiff foundation system capable of distributing building loads and providing greater resistance to differential settlements. The grade beams and slab should be poured monolithically where possible. The thickness of and reinforcement for the slab should be designed by the project structural engineer.

7.9.3 Continuous grade beam foundations may be designed for a bearing value of 2,200 pounds per square foot (psf) and should be a minimum of 12 inches in width, 30 inches in depth below the lowest adjacent grade, and 12 inches into the newly placed engineered fill. The allowable soil bearing pressure above may be increased by 200 psf and 400 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 4,000 psf. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.9.4 The maximum expected total settlement for a structure supported on a grade beam foundation system designed with the maximum allowable bearing value of 4,000 psf and deriving support in the newly placed engineered fill is estimated to be less than 1 inch and occur below the heaviest loaded structural element. Differential settlement is not expected to exceed ½ inch over a distance of 20 feet. A majority of the foundation settlement is expected to occur on initial application of loading with minor additional settlements within the first 12 months.

7.9.5 It is anticipated that the mat foundation will impart an average pressure of less than 2,000 psf. The recommended maximum allowable bearing value for the mat foundation system is 3,000 psf. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.9.6 The thickness of and reinforcement for the mat foundation should be designed by the project structural engineer.

7.9.7 The maximum expected total settlement for a structure supported on a mat foundation system designed with the maximum allowable bearing value of 3,000 psf and deriving support in the newly placed engineered fill is estimated to be less than ¾ inch. and occur below the heaviest loaded structural element. Differential settlement between the center and corner of the mat is not expected to exceed ½ inch. A majority of the foundation settlement is expected to occur on initial application of loading with minor additional settlements within the first 12 months.

7.9.8 For seismic design purposes, a coefficient of friction of 0.30 may be utilized between concrete slab and recommended bearing material without a moisture barrier, and 0.15 for slabs underlain by a vapor retarder or methane barrier.

7.9.9 It is recommended that a modulus of subgrade reaction of 150 pounds per cubic inch (pci) be utilized for the design of the foundation bearing in newly placed engineered fill. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

$$K_R = K\left[\frac{B+1}{2B}\right]^2$$

where:

K_R = reduced subgrade modulus
K = unit subgrade modulus
B = foundation width (in feet)

7.10 Miscellaneous Foundations

7.10.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures which will not be tied to the proposed structure may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils and should be deepened as necessary to maintain a minimum 12 inch embedment into the recommended bearing materials.

7.10.2 If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 30 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.10.3 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated.

7.11 Lateral Design

7.11.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.30 may be used with the dead load forces in the competent alluvial soils or in properly compacted engineered fill.

7.11.2 Passive earth pressure for the sides of foundations and slabs poured against properly compacted engineered fill or competent alluvial soils may be computed as an equivalent fluid having a density of 180 pcf with a maximum earth pressure of 1,800 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.12 Concrete Slabs-on-Grade

7.12.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Pavement Recommendations* section of this report (Section 7.13).

7.12.2 Unless specifically designed and evaluated by the project structural engineer, concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 4 inches thick and minimum slab reinforcement should consist of No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.

7.12.3 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the Los Angeles Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Los Angeles Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

7.12.4 For seismic design purposes, a coefficient of friction of 0.30 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.12.5 Exterior slabs for walkways or flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to 2 percent above optimum moisture content and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 8 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

7.12.6 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

7.13 Preliminary Pavement Recommendations

7.13.1 Where new paving is to be placed, it is recommended that all existing fill and soft or unsuitable alluvial materials be excavated and properly recompacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing unsuitable material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to 2 percent above optimum moisture content, and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.13.2 The following pavement sections are based on an assumed R-Value of 20. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.

7.13.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking And Driveways	4.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	12.0

7.13.4 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction"* (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation"* (Caltrans). The use of Crushed Miscellaneous Base in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction"* (Green Book).

7.13.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 92 and 95 percent relative compactions, respectively, determined by ASTM Test Method D 1557 (latest edition).

7.13.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

7.14 Retaining Wall and Elevator Pit Design

7.14.1 The construction of new retaining walls associated with the ascending slope that bounds the southern portion of the site will be provided under a separate cover after additional investigation and analyses are performed. The recommendations presented herein are for small site retaining walls as well as elevator pit wall with minor backfills.

7.14.2 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 5 feet. In the event that walls significantly higher than 5 feet are planned, Geocon should be contacted for additional recommendations.

7.14.3 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* sections of this report (see Sections 7.6 through 7.9).

7.14.4 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 35 pcf.

7.14.5 Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 50 pcf.

7.14.6 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvial soils or engineered fill derived from onsite soils. If import soil will be used to backfill proposed retaining walls, revised earth pressures may be required to account for the geotechnical properties of the import soil used as engineered fill. This should be evaluated once the use of import soil is established. All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site.

7.14.7 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, typical of elevator pits, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf.

7.14.8 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

7.15 Retaining Wall Drainage

7.15.1 Retaining walls should be provided with a drainage system. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 6). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.15.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 7). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

7.15.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

7.15.4 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

7.16 Elevator Piston

7.16.1 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to a foundation or pile, or the drilled excavation could compromise the existing foundation or pile support, especially if the drilling is performed subsequent to the foundation construction.

7.16.2 Casing will be required since groundwater and some caving is expected in the drilled excavation and the contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. The contractor should also be prepared to mitigate buoyant forces during installation of the piston casing. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer (a representative of Geocon West, Inc.) is required.

7.16.3 The annular space between the piston casing and drilled excavation wall should be filled with a minimum of 1½-sack slurry pumped from the bottom up. As an alternative, pea gravel may be utilized. The use of soil to backfill the annular space is not acceptable.

7.17 Temporary Excavations

7.17.1 Excavations on the order of 10 feet in height may be required during grading operations. The excavations are expected to expose artificial fill and alluvial soils with possible bedrock at the excavation bottom near the slope. Vertical excavations up to 5 feet in height may be attempted where loose fill soils or caving sands are not present, and where not surcharged by adjacent traffic or structures.

7.17.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to maximum height of 10 feet. A uniform slope does not have a vertical portion.

7.17.3 Performing continuous vertical excavations along property lines or adjacent to an existing structure could remove support from the property and/or structure which is not acceptable. If excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such as slot-cutting or shoring may be necessary in order to maintain lateral support of offsite improvements. Recommendations for special temporary excavation measures can be provided under separate cover once the proposed building layout is established.

7.17.4 Where sloped embankments are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. Geocon personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

7.18 Surface Drainage

7.18.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

7.18.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2016 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.18.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond.

7.18.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or an impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

7.19 Plan Review

7.19.1 Grading, foundation, and shoring plans should be reviewed by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon West, Inc.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

LIST OF REFERENCES

California Department of Water Resources, 2018, *Groundwater Level Data by Township, Range, and Section,* http://www.water.ca.gov/waterdatalibrary/groundwater/ hydrographs/index_trs.cfm.

California Department of Water Resources, 1961, *Planned Utilization of Groundwater Basins of the Coastal Plain of Los Angeles County*, Bulletin 104, Appendix A.

California Division of Mines and Geology, 1999; *State of California Seismic Hazard Zones, Los Angeles Quadrangle,* Official Map, Released: March 25, 1999.

California Division of Mines and Geology, 1998, *Seismic Hazard Evaluation of the Los Angeles 7.5-Minute Quadrangle, Los Angeles County, California,* Open File Report 98-20.

California Division of Oil, Gas and Geothermal Resources, 2019, Division of Oil, Gas, and Geothermal Resources Well Finder, http://maps.conservation.ca.gov.doggr/index.html#close. Accessed March 12, 2018.

California Geological Survey, 2018a, *Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California,* Special Publication 42, Revised 2018.

California Geological Survey, 2018b, CGS Information Warehouse, Regulatory Map Portal, http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps.

California Geological Survey, 2017, *Earthquake Zones of Required Investigation, Los Angeles Quadrangle, Los Angeles County, California*, *dated June 15, 2017*.

California Geological Survey, 2012, *Geologic Compilation of Quaternary Surficial Deposits in Southern California, Los Angeles 30' X 60' Quadrangle*, A Project for the Department of Water Resources by the California Geological Survey, Compiled from existing sources by Trinda L. Bedrossian, CEG and Peter D. Roffers, CGS Special Report 217, Plate 9, Scale 1:100,000.

Dibblee, T. W., Jr., 1989, *Geologic Map of the Los Angeles Quadrangle, Los Angeles County, California,* Dibblee Foundation Map # DF-22.

FEMA, 2019, Online Flood Hazard Maps, *Flood Insurance Rate Map, Los Angeles County, California and Unincorporated Areas, Map Number 06037C1636F, Date Accessed: March 12, 2018,* http://www.esri.com/hazards/index.html.

Geocon West, Inc., 2018, *Geotechnical Investigation, Proposed Health Agency Recuperative Campus, 1774 Zonal Avenue, Los Angeles, California, Tract: TR 2082, Lot: LT 1*, prepared for KFA, LP, Geocon Project Number A7933-06-01, report dated April 9, 2018.

Jennings, C. W. and Bryant, W. A., 2010, *Fault Activity Map of California,* California Geological Survey Geologic Data Map No. 6.

Lamar, D.L., 1970, *Geology of the Elysian Park-Repetto Hills Area, Los Angeles County, California,* California Division of Mines and Geology Special Report 101.

LIST OF REFERENCES (Continued)

Leighton and Associates, Inc., 1990, *Technical Appendix to the Safety Element of the Los Angeles County General Plan, Hazard Reduction in Los Angeles County*.

Los Angeles, City of, 2018, NavigateLA website, http://navigatela.lacity.org.

Los Angeles, City of, 1996, *Safety Element of the Los Angeles City General Plan*.

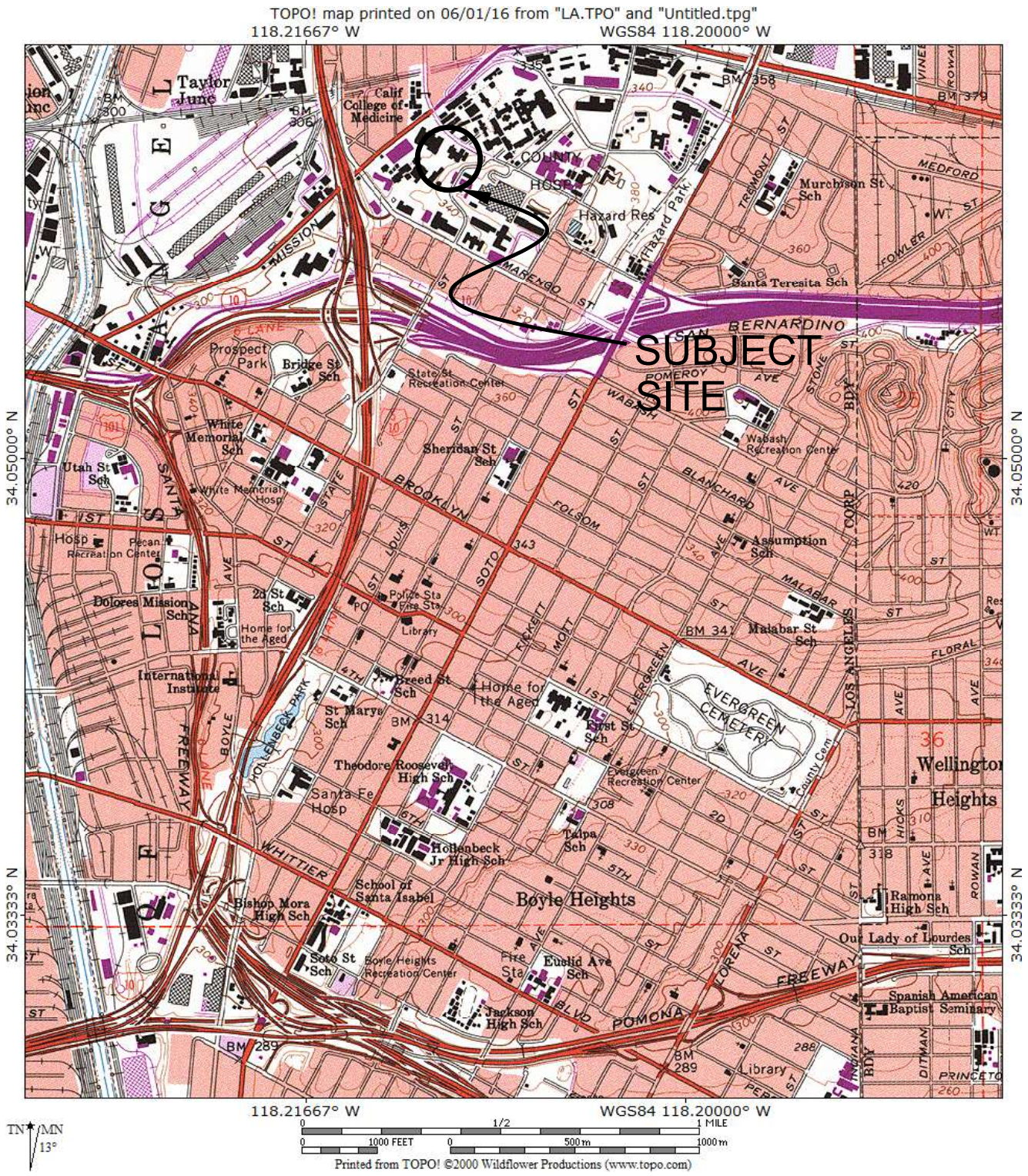
Los Angeles County Department of Public Works, 2019a, Ground Water Wells Website, http://dpw2.co.la.ca.us/website/wells/viewer.asp.

Los Angeles County Department of Public Works, 2019b, Flood Zone Determination Website, http://dpw.lacounty.gov/apps/wmd/floodzone/map.htm.

Toppozada, T., Branum, D., Petersen, M, Hallstrom, C., and Reichle, M., 2000, *Epicenters and Areas Damaged by M> 5 California Earthquakes, 1800 – 1999,* California Geological Survey, Map Sheet 49.

U.S. Geological Survey Seismic Design Maps Web Application, 2016, http://earthquake.usgs.gov/designmaps/us/application.php.

Ziony, J. I., and Jones, L. M., 1989, *Map Showing Late Quaternary Faults and 1978–1984 Seismicity of the Los Angeles Region, California*, U.S. Geological Survey Miscellaneous Field Studies Map MF-1964.



REFERENCE:U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, LOS ANGELES, CA QUADRANGLE

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RA	CHECKED BY: SFK

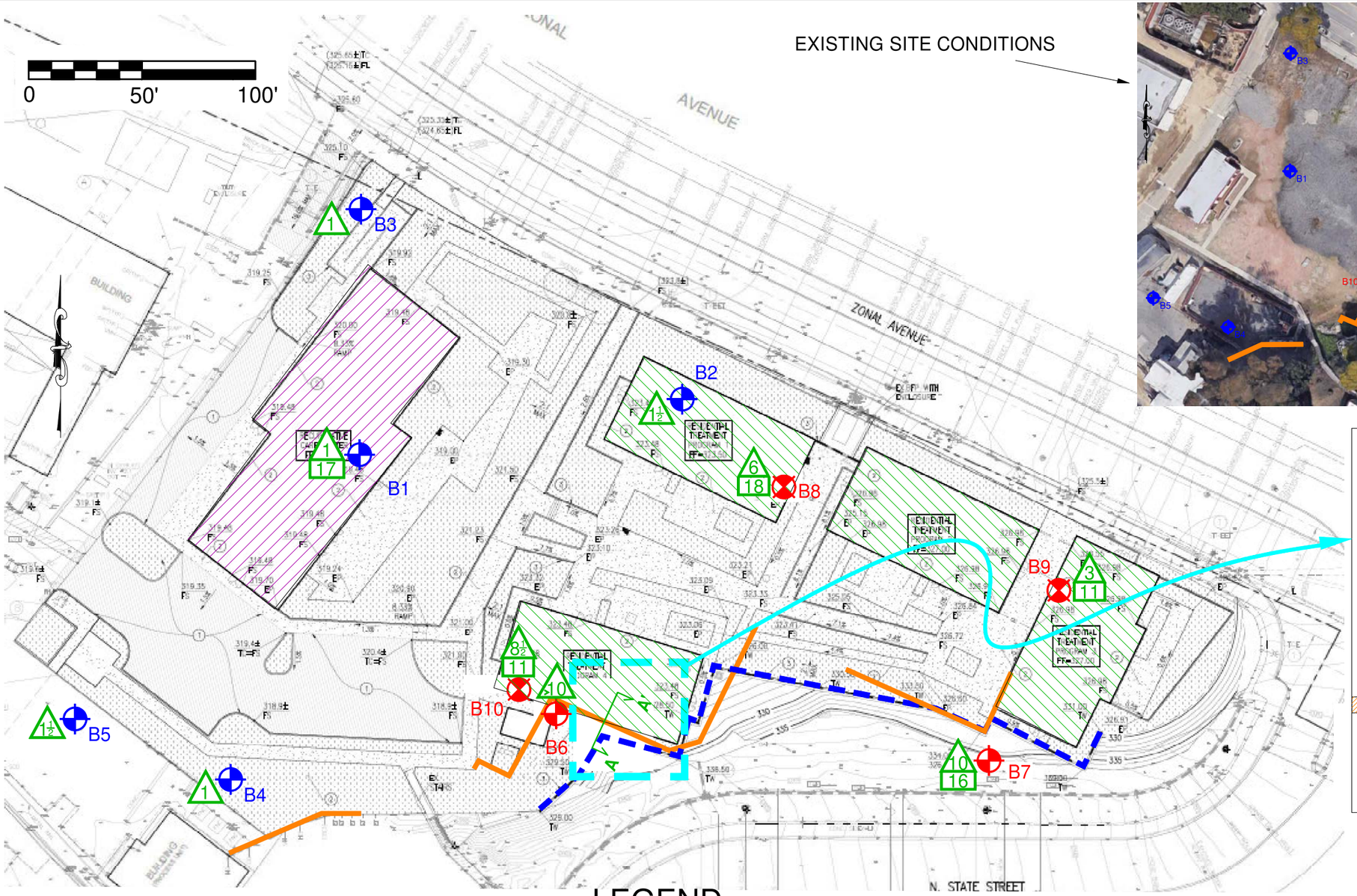
VICINITY MAP

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY 2019	PROJECT NO. A8559-06-74	FIG. 1

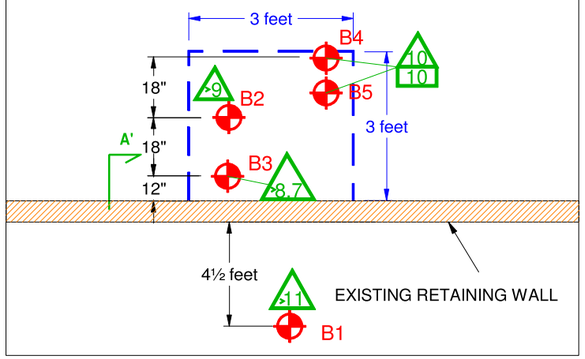
EXISTING SITE CONDITIONS





ENHANCED VIEW

NO SCALE - ILLUSTRATION ONLY



LEGEND

- B7 — Location of Manual Auger Boring
- B10 — Location of Bucket Auger Boring
- B5 — Location of Prior Boring (A9733-06-01)
- 11 / 17 — Depth of Fill (Feet) / Depth of GroundWater (Feet)
- A A' — Test Pit Cross Section (See Figure 3)
- Existing Retaining Walls
- Proposed RCC Structure
- Proposed RTP Structures
- Proposed Retaining Walls

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

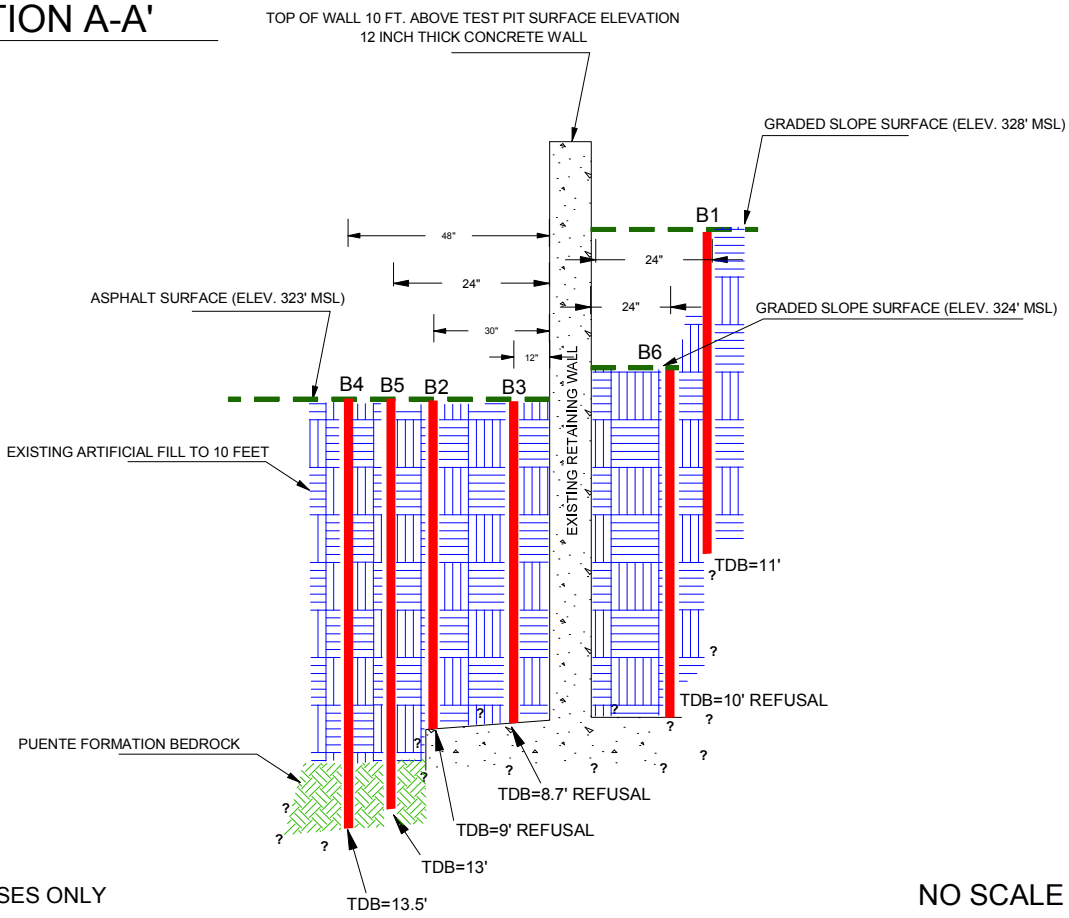
DRAFTED BY: RP	CHECKED BY: HHD

SITE PLAN

1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

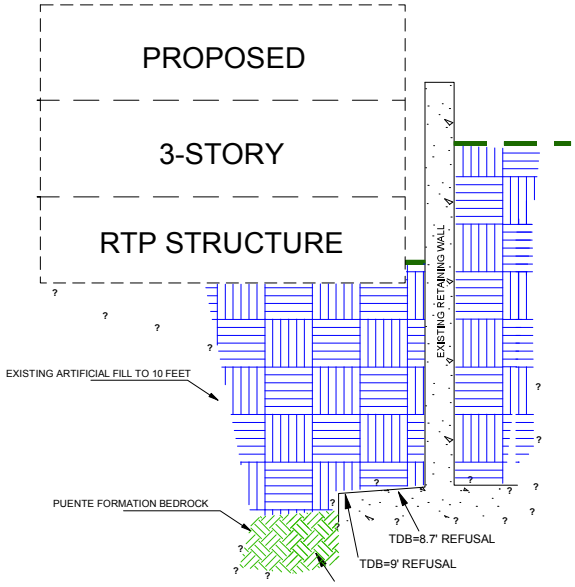
MAY 2019	PROJECT NO. A8559-06-74	FIG. 2

CROSS SECTION A-A'



FOR ILLUSTRATION PURPOSES ONLY

NO SCALE



GEOCON
W E S T, I N C.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

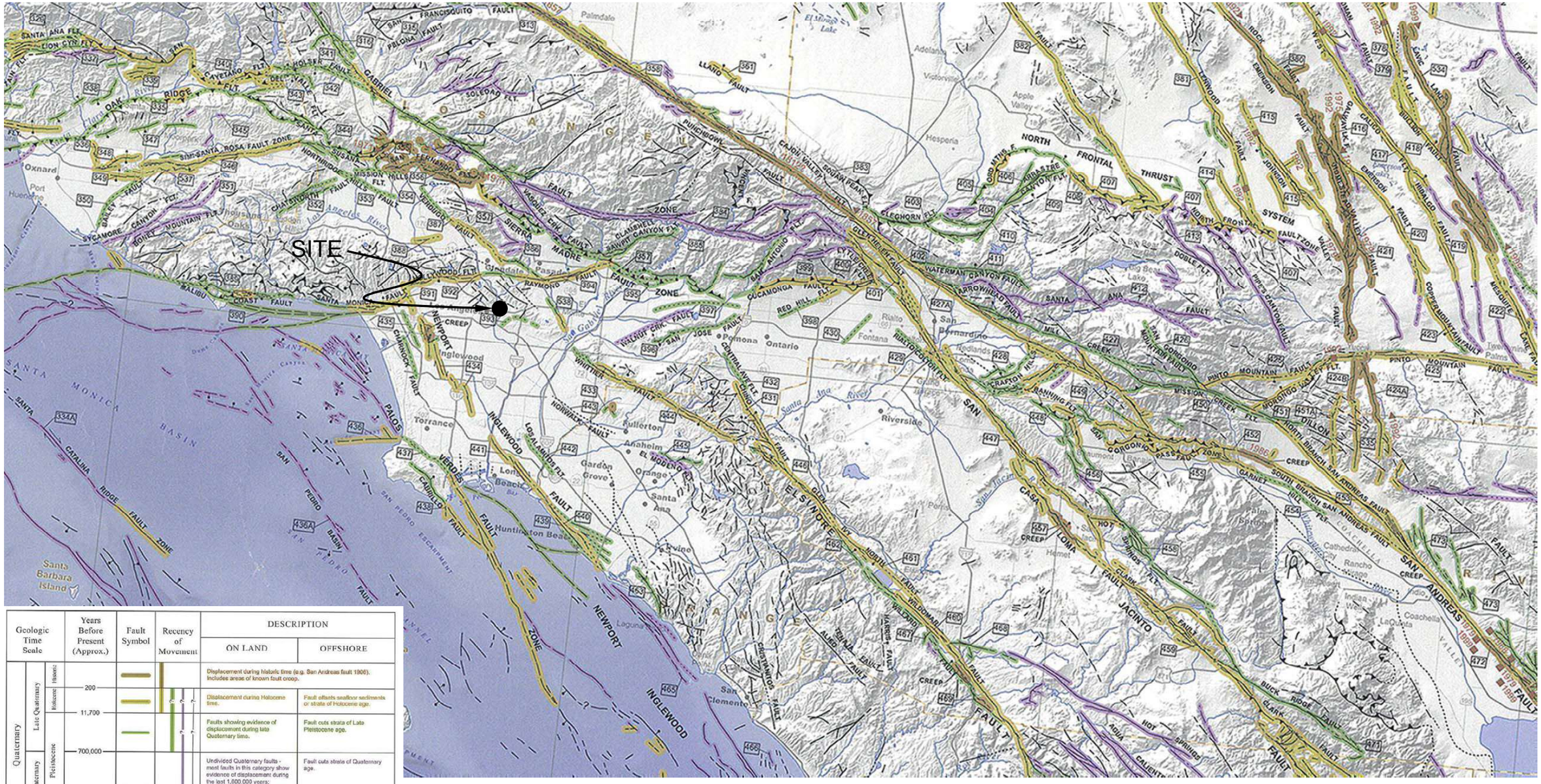
DRAFTED BY: RP	CHECKED BY: HHD

TEST PIT CROSS SECTION

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY 2019	PROJECT NO. A8559-06-74	FIG. 3

Reference: Jennings, C.W. and Bryant, W. A., 2010, Fault Activity Map of California, California Geological Survey Geologic Data Map No. 6.



Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION: ON LAND	DESCRIPTION: OFFSHORE
Quaternary	Late Quaternary	Historic				Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
		Holocene	200			Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
		Pleistocene	11,700			Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary		700,000			Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary			1,600,000*			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.
			4.5 billion (Age of Earth)				

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.8 Ma criterion.

N

0 12 24 Miles

GEOCON WEST, INC.

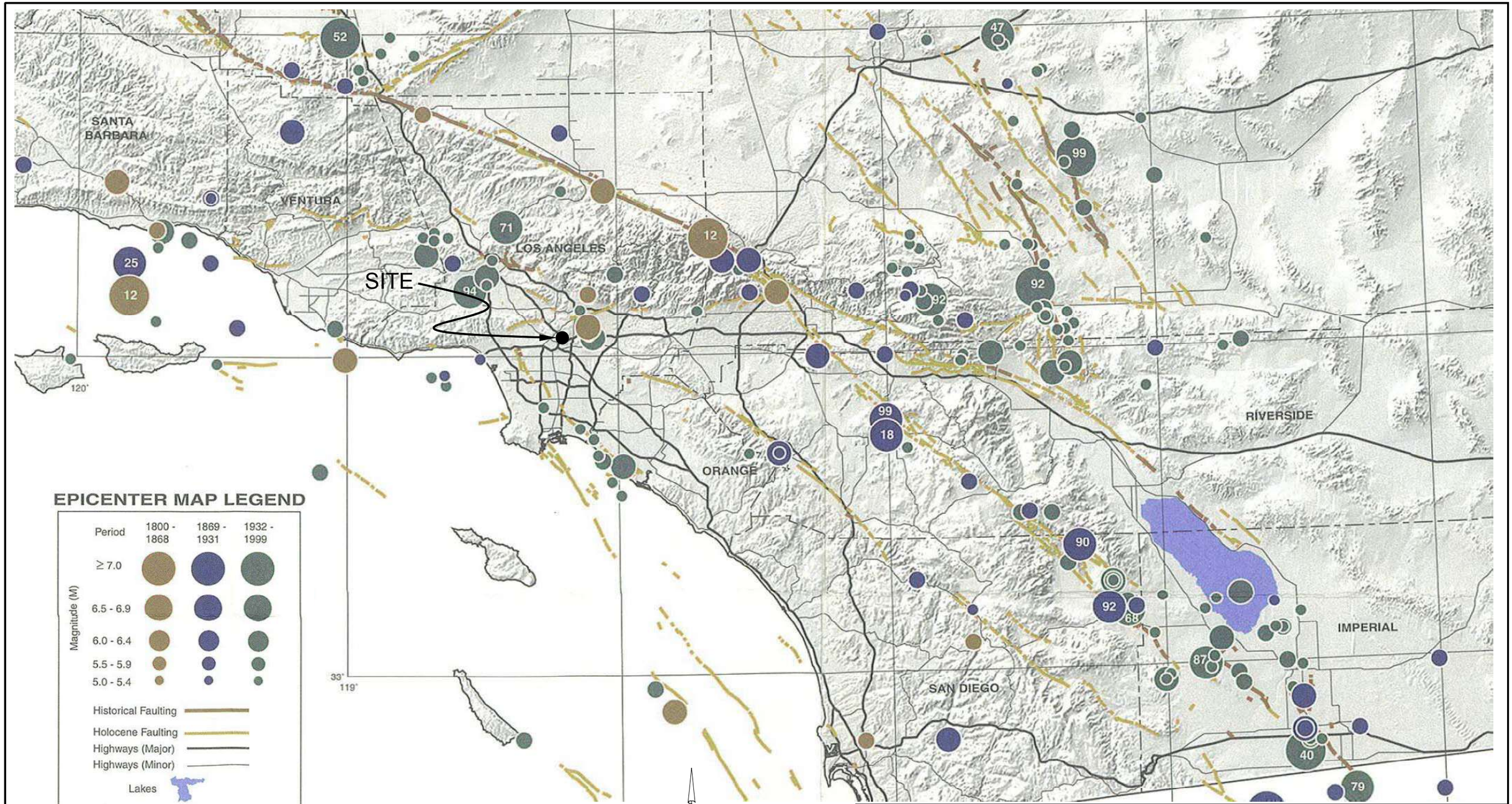
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RA | CHECKED BY: SFK

REGIONAL FAULT MAP

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY 2019 | PROJECT NO. A8559-06-74 | FIG. 4



Reference: Toppozada, T., Branum, D., Petersen, M., Hallstrom, C., Cramer, C., and Reichle, M., 2000, Epicenters and Areas Damaged by M≥5 California Earthquakes, 1800 - 1999, California Geological Survey, Map Sheet 49.

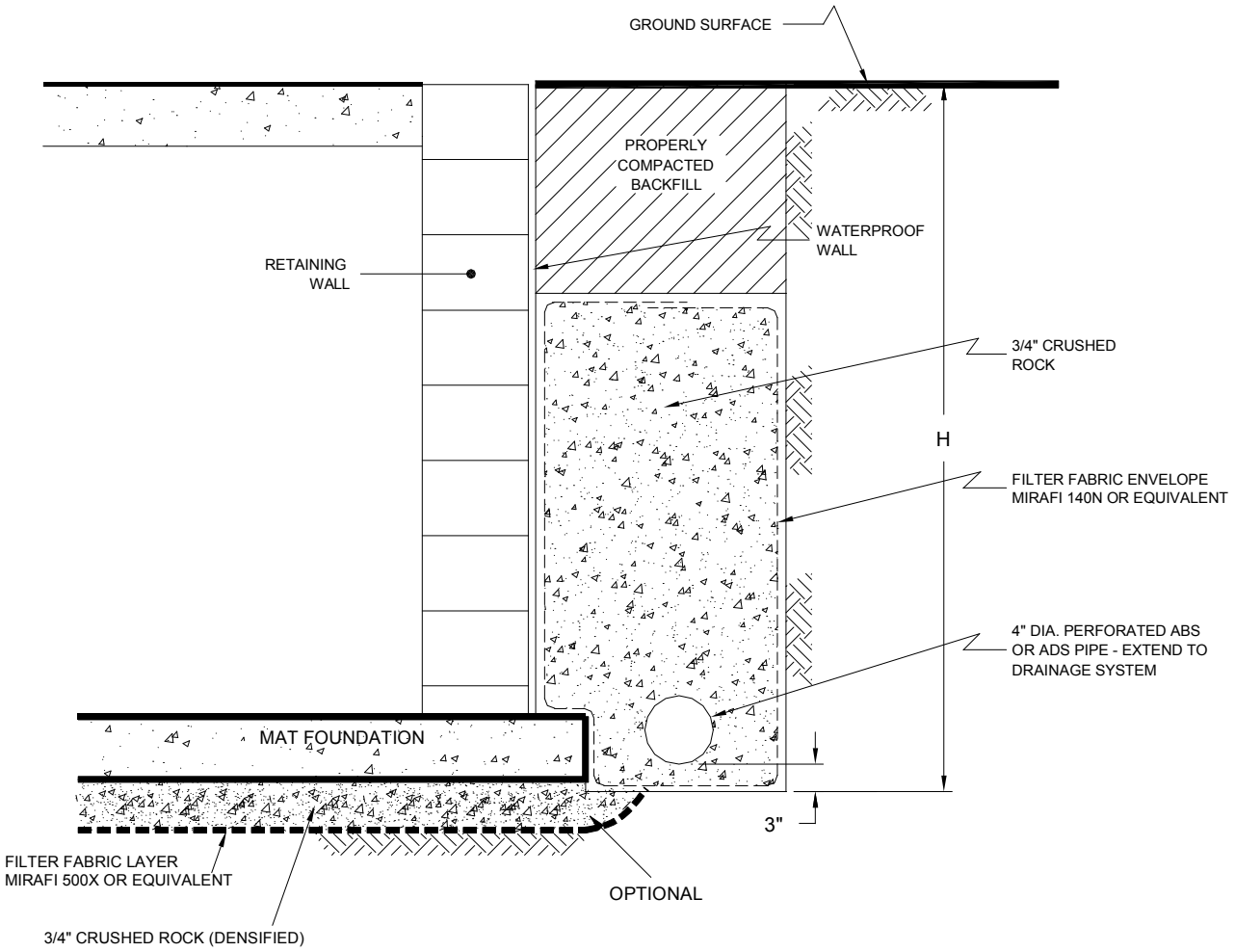
GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RA	CHECKED BY: SFK

REGIONAL SEISMICITY MAP

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY 2019	PROJECT NO. A8559-06-74	FIG. 4



NO SCALE

GEOCON WEST, INC.

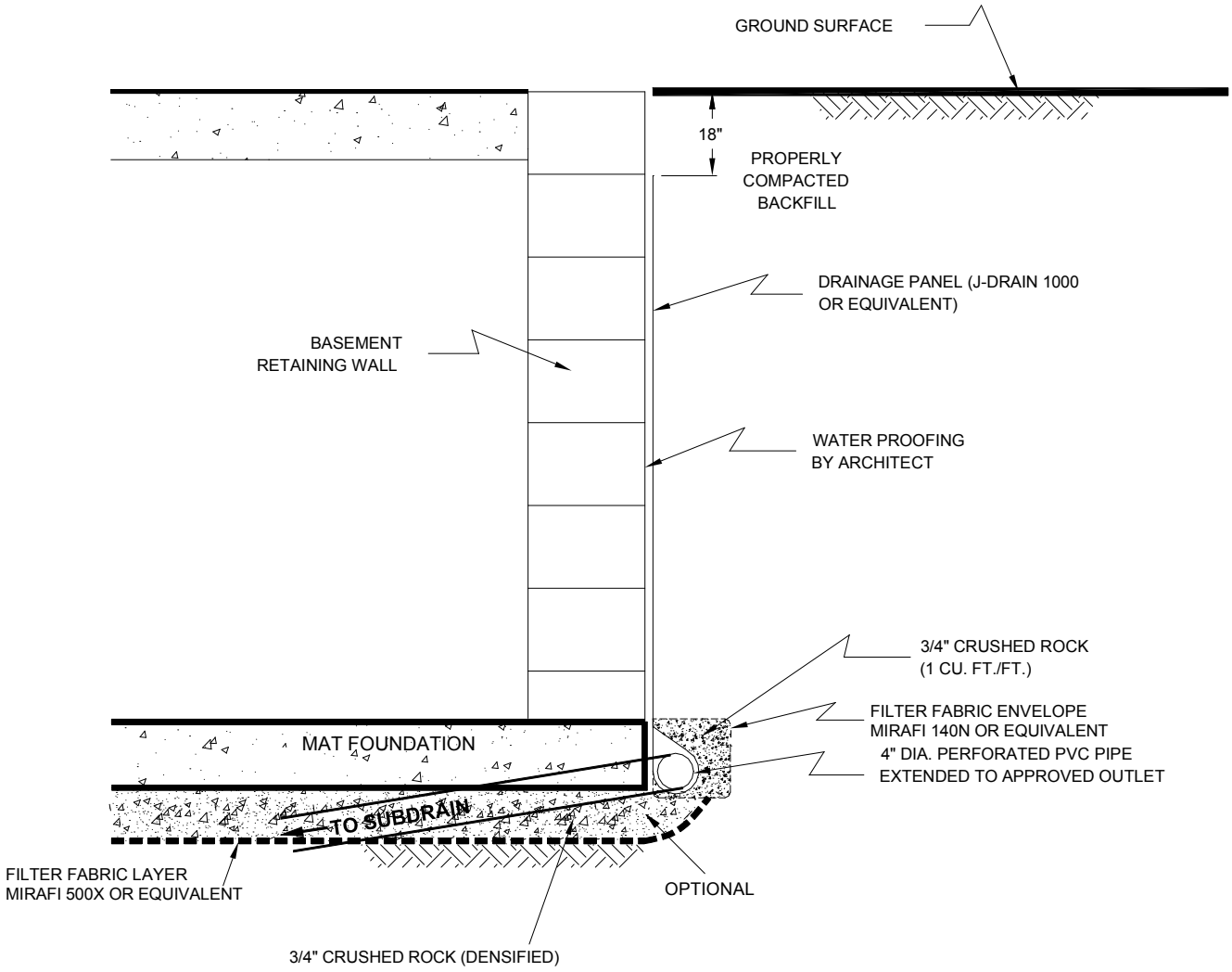
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RP	CHECKED BY: JTA/NDB

RETAINING WALL DRAIN DETAIL

1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY 2019	PROJECT NO. A8559-06-74	FIG. 6



NOTE: TOP OF DRAINAGE PANEL NOT MORE THAN 18 INCHES FROM GROUND SURFACE

NO SCALE

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RP	CHECKED BY: JTA/NDB

RETAINING WALL DRAIN DETAIL

1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY 2019	PROJECT NO. A8559-06-74	FIG. 7

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was previously explored on March 6, 2018, by excavating five 8-inch diameter borings to depths of approximately 15½ and 56 feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. Additional site exploration was performed on April 22 and 23, 2019, by manually excavating seven borings (B1 through B7) and one test pit using hand auger equipment and digging tools. Additionally, on April 30, 2019, one 24-inch and two 18-inch diameter borings were drilled using a bucket auger drilling machine. The bucket auger borings were advanced to a maximum depth of 25½ feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2⅜-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A10. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the boring logs were revised based on subsequent laboratory testing. The locations of the borings are indicated on Figure 2.

PROJECT NO. A8559-06-74

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 1** ELEV. (MSL.) **328** DATE COMPLETED **4/22/19** EQUIPMENT **HAND AUGER** BY: **RP** MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silt with Clay, soft to firm, slightly moist, dark brown with yellowish brown mottles, fine-grained, plastic, some sandstone/siltstone fragments, some rootlets.			
2	B1@2.5'				- yellowish brown with olive brown mottles		92.3	23.8
4	B1@5'				- increase in bedrock fragments, trace quartz fragments		101.7	19.1
6								
8	B1@7.5'				- trace rootlets, slight oxidation staining, trace carbon deposits		96.0	23.4
10	B1@10'						94.5	27.7
					- refusal on rock Total depth of boring: 11 feet All fill. No groundwater encountered. Backfilled with soil cuttings and tamped.			

A8559-06-74 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 2** ELEV. (MSL.) **323** DATE COMPLETED **4/22/19** EQUIPMENT **HAND AUGER** BY: **RP** MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 4" BASE: 12"**			
					ARTIFICIAL FILL			
2					Silt with Clay, firm, slightly moist, yellowish brown with brown mottles, some bedrock fragments, plastic, trace rootlets.			
	B2@3'						102.5	23.1
4					- yellowish brown with olive brown mottles, trace diatomaceous content, some oxidation.			
6	B2@6'						96.1	26.8
8					- soft, brown with yellowish brown mottles, increase in moisture - refusal on hard surface			
					Total depth of boring: 9 feet All fill. No groundwater encountered. Backfilled with soil cuttings and tamped. Surface patched with asphalt.			

A8559-06-74 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 3** ELEV. (MSL.) **323** DATE COMPLETED **4/22/19** EQUIPMENT **HAND AUGER** BY: **RP**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**AC: 4" BASE: 12"** **ARTIFICIAL FILL** Silt with Clay, firm, slightly moist, yellowish brown with brown mottles, some bedrock fragments, plastic, trace rootlets.			
2								
4					- yellowish brown with olive brown mottles, trace diatomaceous content, some oxidation.			
6								
8					- soft, moist, brown with yellowish brown mottles - refusal on hard surface Total depth of boring: 8.7 feet All fill. No groundwater encountered. Backfilled with soil cuttings and tamped. Surface patched with asphalt.			

A8559-06-74 BORING LOGS.GPJ

Figure A3,
Log of Boring 3, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

BORING 4

ELEV. (MSL.) **323** DATE COMPLETED **4/23/19**

EQUIPMENT **HAND AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 4" BASE: 12"**			
					ARTIFICIAL FILL			
2					Silt with Clay, slightly moist, yellowish brown with brown mottles, some bedrock fragments, plastic, trace rootlets.			
4					- yellowish brown with olive brown mottles, trace diatomaceous content			
6								
8								
10			▼		- soft, brown			
					PUENTE FORMATION			
12	B4@11.5'				Sandy Siltstone, brown with olive brown mottles, fine-grained, poorly bedded, massive, hard, moderately weathered.		---	---
	B4@12.5'				- no recovery		94.4	29.0
	B4@13'						---	---
					Total depth of boring; 13.5 feet			
					Fill to 10 feet.			
					Perched groundwater encountered at 10 feet.			
					Backfilled with soil cuttings and tamped.			
					Surface patched with asphalt.			

A8559-06-74 BORING LOGS.GPJ

Figure A4,
Log of Boring 4, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 5** ELEV. (MSL.) **323** DATE COMPLETED **4/23/19** EQUIPMENT **HAND AUGER** BY: **RP**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**AC: 4" BASE: 12"** **ARTIFICIAL FILL** Silt with Clay, slightly moist, yellowish brown with brown mottles, some bedrock fragments, plastic, trace rootlets.			
2								
4					- yellowish brown with olive brown mottles, trace diatomaceous content			
6								
8								
10					- soft, brown **PUENTE FORMATION** Sandy Siltstone, brown with olive brown mottles, fine-grained, poorly bedded, massive, hard, moderately weathered.			
12								
					Total depth of boring; 13 feet Fill to 10 feet. Perched groundwater encountered at 10 feet. Backfilled with soil cuttings and tamped. Surface patched with asphalt.			

A8559-06-74 BORING LOGS.GPJ

Figure A5,
Log of Boring 5, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 6** ELEV. (MSL.) **324** DATE COMPLETED **4/23/19** EQUIPMENT **HAND AUGER** BY: **RP**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**GRADED SLOPE** **ARTIFICIAL FILL** Silt with Clay, slightly moist, yellowish brown with brown mottles, some bedrock fragments, plastic, trace rootlets.			
2								
4					- yellowish brown with olive brown mottles, trace diatomaceous content, some oxidation			
6								
8								
10					- refusal on hard surface Total depth of boring: 10 feet All fill. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A6,
Log of Boring 6, Page 1 of 1

A8559-06-74 BORING LOGS.GPJ

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

BORING 7

ELEV. (MSL.) **335** DATE COMPLETED **4/23/19**

EQUIPMENT **HAND AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silt with Clay, firm, slightly moist, yellowish brown with olive brown mottles, some bedrock fragments.			
2	B7@2.5'						90.2	27.2
4								
6	B7@5'				- brown with dark brown mottles, slight increase in moisture, plastic		104.5	17.6
8	B7@7.5'				- brown to yellowish brown, increase in silt some rootlets		101.1	19.5
10	B7@10'				**COLLUVIUM** Silt with Clay, firm, slightly moist, brown with olive brown mottles, some carbon deposits.		106.4	21.7
12	B7@12'				- grayish brown, decrease in silt, plastic, increase in moisture - reddish brown with olive brown mottles, increased carbon deposits, some fine-grained sand		108.4	19.3
14				CL-ML				
16			▼					
18	B7@17.5'				**PUENTE FORMATION** Sandy Siltstone, yellowish brown with olive brown mottles, fine-grained, poorly bedded, massive, hard, highly weathered.		109.3	18.5

Total depth of boring: 18 feet
Fill to 10 feet.
Perched groundwater enountered at 16 feet.
Backfilled with soil cuttings and tamped.

A8559-06-74 BORING LOGS.GPJ

Figure A7,
Log of Boring 7, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

BORING 8

ELEV. (MSL.) **323** DATE COMPLETED **4/30/19**

EQUIPMENT **BUCKET AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK @0-5'				**AC: 3.5" BASE: 12"** **ARTIFICIAL FILL** Silt with Clay, firm, slightly moist, yellowish brown with olive brown mottles, fine-grained, some bedrock fragments, plastic.			
2								
4	B8@3'				- dark brown	1	97.2	26.0
6	B8@6'				**COLLUVIUM** Silt with Clay, firm to hard, slightly moist, reddish brown, some caliche.	5	110.6	18.8
8					- abundant diatomaceous content			
10	B8@9'			CL-ML	- hard	10	110.0	17.4
12	B8@12'				- brown to reddish brown	6	110.7	19.7
14					Silty Sand, medium dense, slightly moist, olive brown, fine-grained, some clay, some oxidation staining.			
16	B8@15'			SM		7	112.4	19.5
18			▼		- moist, groundwater encountered			
20	B8@20'				**PUENTE FORMATION** Sandy Siltstone, brown with olive brown mottles, poorly bedded, massive, medium dense, moderately weathered to highly weathered.	6	100.7	24.5
22								
24								
	B8@25'					13	107.9	19.0
					Total depth of boring: 25.5 feet Fill to 6 feet. Groundwater encountered at 18 feet. Backfilled with soil cuttings and tamped. Surface patched with asphalt. *Kelley Bar Weights: 0-24' 2,400 lbs 25-44' 1,550 lbs			

A8559-06-74 BORING LOGS.GPJ

Figure A8,
Log of Boring 8, Page 1 of 1

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

⊠ ... DISTURBED OR BAG SAMPLE ◪ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

BORING 9

ELEV. (MSL.) **326** DATE COMPLETED **4/30/19**

EQUIPMENT **BUCKET AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK @0-5'				**AC: 3" BASE: 12"** **ARTIFICIAL FILL** Silt with Clay, firm, slightly moist, dark brown with olive brown mottles, trace gravel, trace coarse-grained sand.			
2								
4	B9@3'				**COLLUVIUM** Silt with Clay, firm, slightly moist, dark brown, plastic, some oxidation mottling.	2	106.4	19.0
6	B9@6'				- hard, some silt	5	112.0	14.4
8				CL-ML	- brown, increase in silt - firm, trace bedrock fragments			
10	B9@9'		▼		- olive brown to yellowish brown - groundwater encountered	10	109.2	18.8
12	B9@12'				- some oxidation staining, trace Silty Sand interbeds	1	108.7	19.1
14								
16	B9@15'				**PUENTE FORMATION** Siltstone, yellowish brown with olive brown mottles, poorly bedded, massive, hard, highly to completely weathered, some oxidation.	2	113.7	18.5
18								
20	B9@20'				- olive brown with brown mottles, some calcium carbonate	5	111.1	19.2
					Total depth of boring: 20.5 feet Fill to 3 feet. Perched groundwater encountered at 11 feet. Backfilled with soil cuttings and tamped. Surface patched with asphalt. *Kelley Bar Weights: 0-24' 2,400 lbs 25-44' 1,550 lbs			

A8559-06-74 BORING LOGS.GPJ

Figure A9,
Log of Boring 9, Page 1 of 1

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

⊠ ... DISTURBED OR BAG SAMPLE ◪ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

BORING 10

ELEV. (MSL.) **321** DATE COMPLETED **4/30/19**

EQUIPMENT **BUCKET AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK @0-5'				**ARTIFICIAL FILL** Silt with Clay, firm, slightly moist, brown with dark brown mottles, fine- to medium-grained, plastic, some fine gravel, trace glass fragments.			
2					- some interbedded bedrock fragments			
4	B10@3'					7	110.0	14.9
6	BULK @5'-10'				- yellowish brown with olive brown mottles			
8	B10@6'				- trace concrete	4	96.7	20.2
10	B10@9'		▼		**COLLUVIUM** Silt with Clay, firm, slightly moist, dark brown, plastic.	PUSH	106.8	22.7
12	B10@12'				- groundwater encountered - brown, some caliche	2	108.2	20.5
14				CL-ML				
16	B10@15'				- some sandstone fragments	3	104.6	18.2
18								
20	B10@20'				**PUENTE FORMATION** Siltstone, brown with olive brown mottles, poorly bedded, massive, hard, moderately weathered.	5	98.5	26.1
22								
24	B10@24.5					10	103.5	25.3
	B10@25'				Total depth of boring: 25.5 feet Fill to 8.5 feet. Groundwater encountered at 11 feet. Backfilled with soil cuttings and tamped. *Kelley Bar Weights: 0-24' 2,400 lbs 25-44' 1,550 lbs	19	100.2	25.5

A8559-06-74 BORING LOGS.GPJ

Figure A10,
Log of Boring 10, Page 1 of 1

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

⊠ ... DISTURBED OR BAG SAMPLE ◥ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

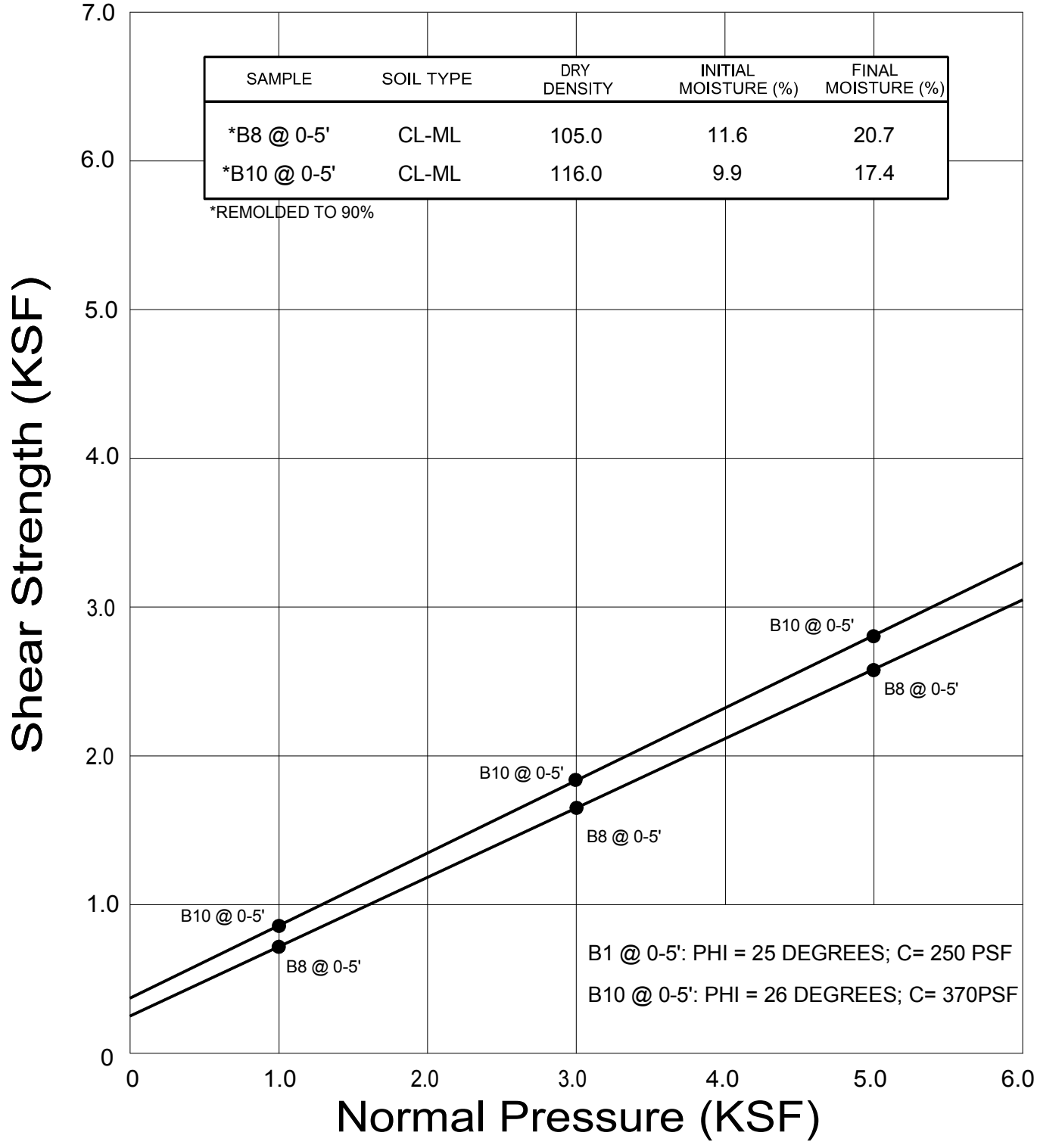
GEOCON

APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, grain size, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized herein. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

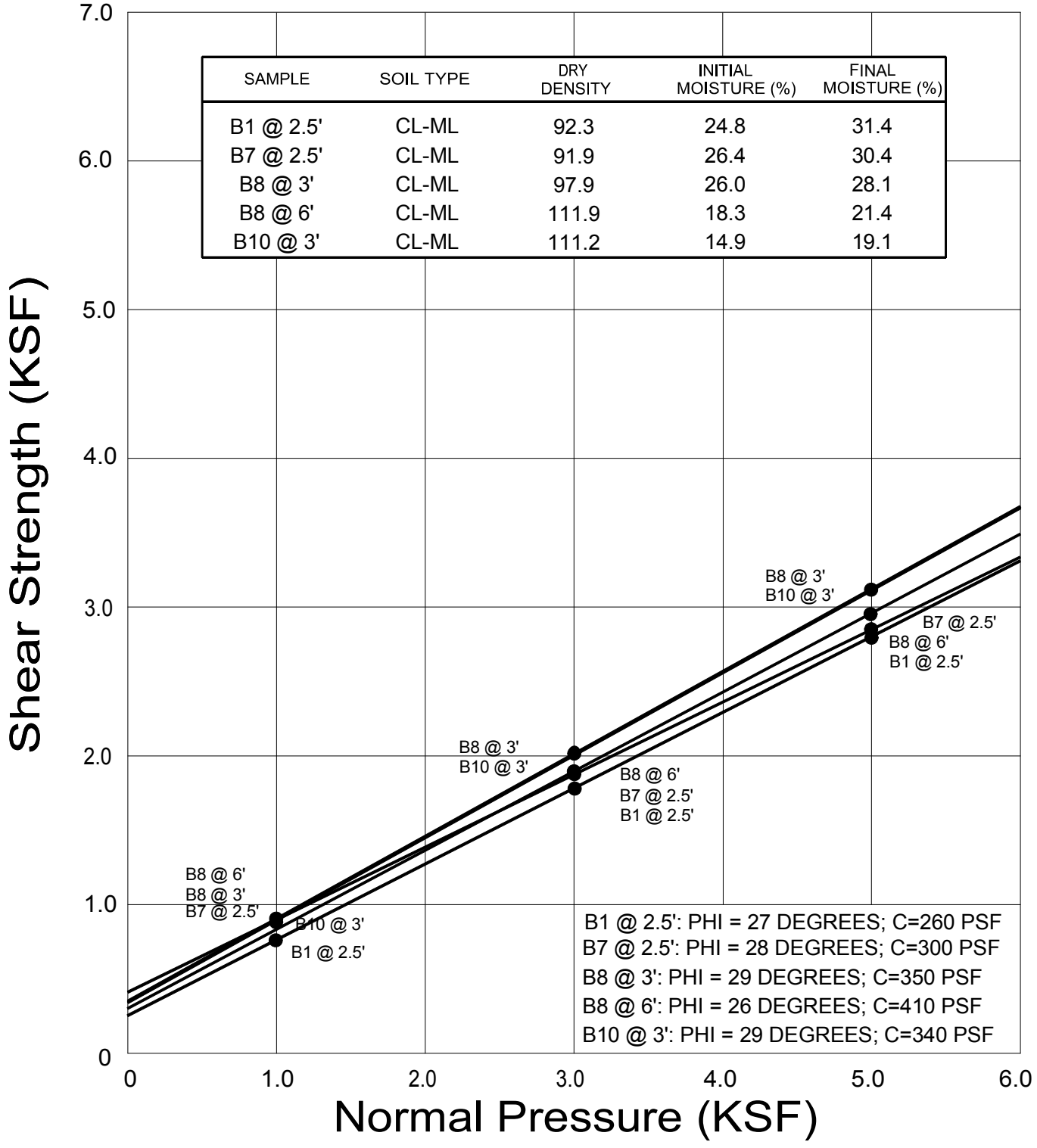
DRAFTED BY: RP | CHECKED BY: HHD

DIRECT SHEAR TEST RESULTS

1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY 2019 | PROJECT NO. A8559-06-74 | FIG. B1

SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @ 2.5'	CL-ML	92.3	24.8	31.4
B7 @ 2.5'	CL-ML	91.9	26.4	30.4
B8 @ 3'	CL-ML	97.9	26.0	28.1
B8 @ 6'	CL-ML	111.9	18.3	21.4
B10 @ 3'	CL-ML	111.2	14.9	19.1



● Direct Shear, Saturated

GEOCON
WEST, INC.

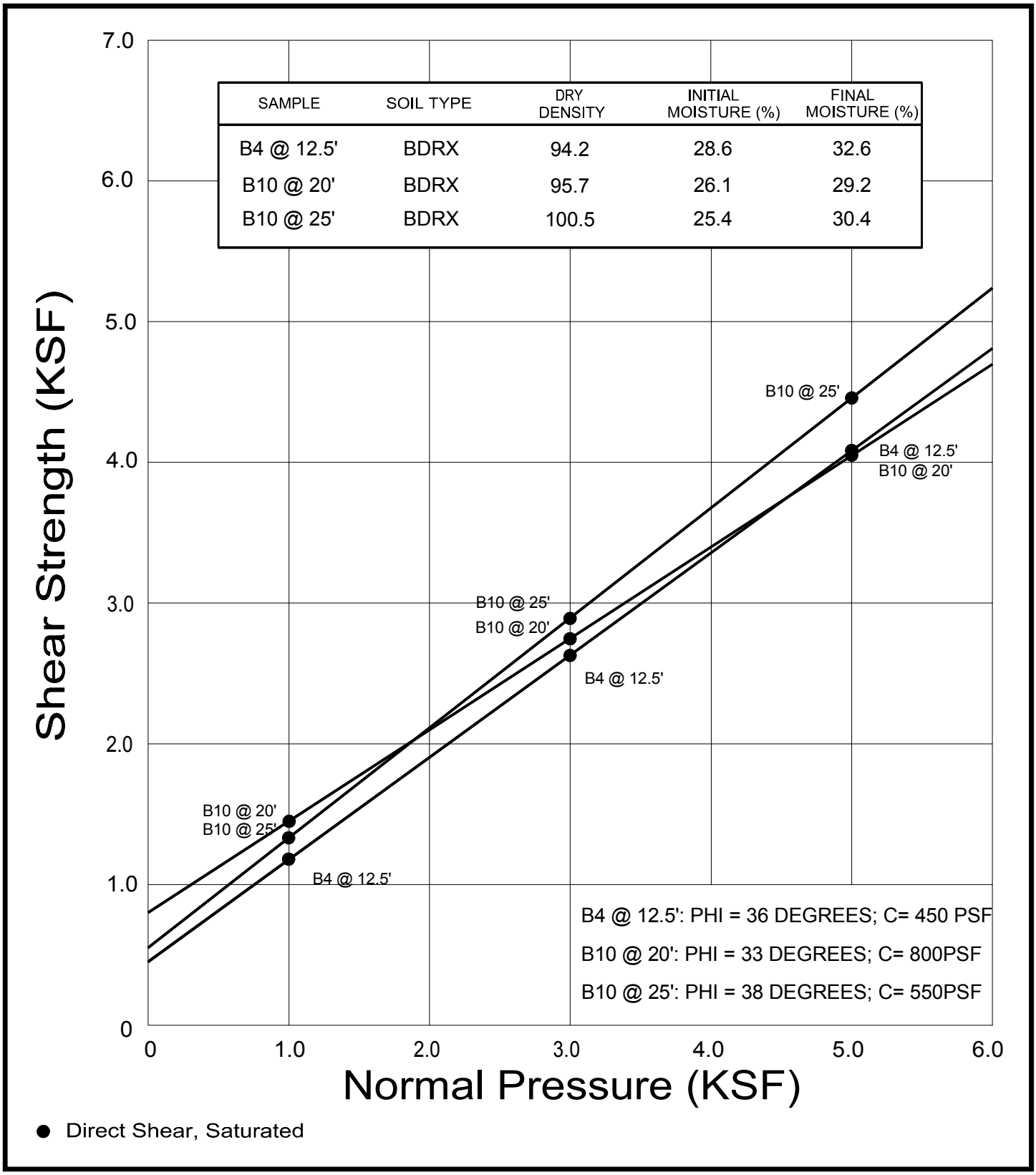
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMH	CHECKED BY: HHD

DIRECT SHEAR TEST RESULTS

1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY 2019	PROJECT NO. A8559-06-74	FIG. B2



GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

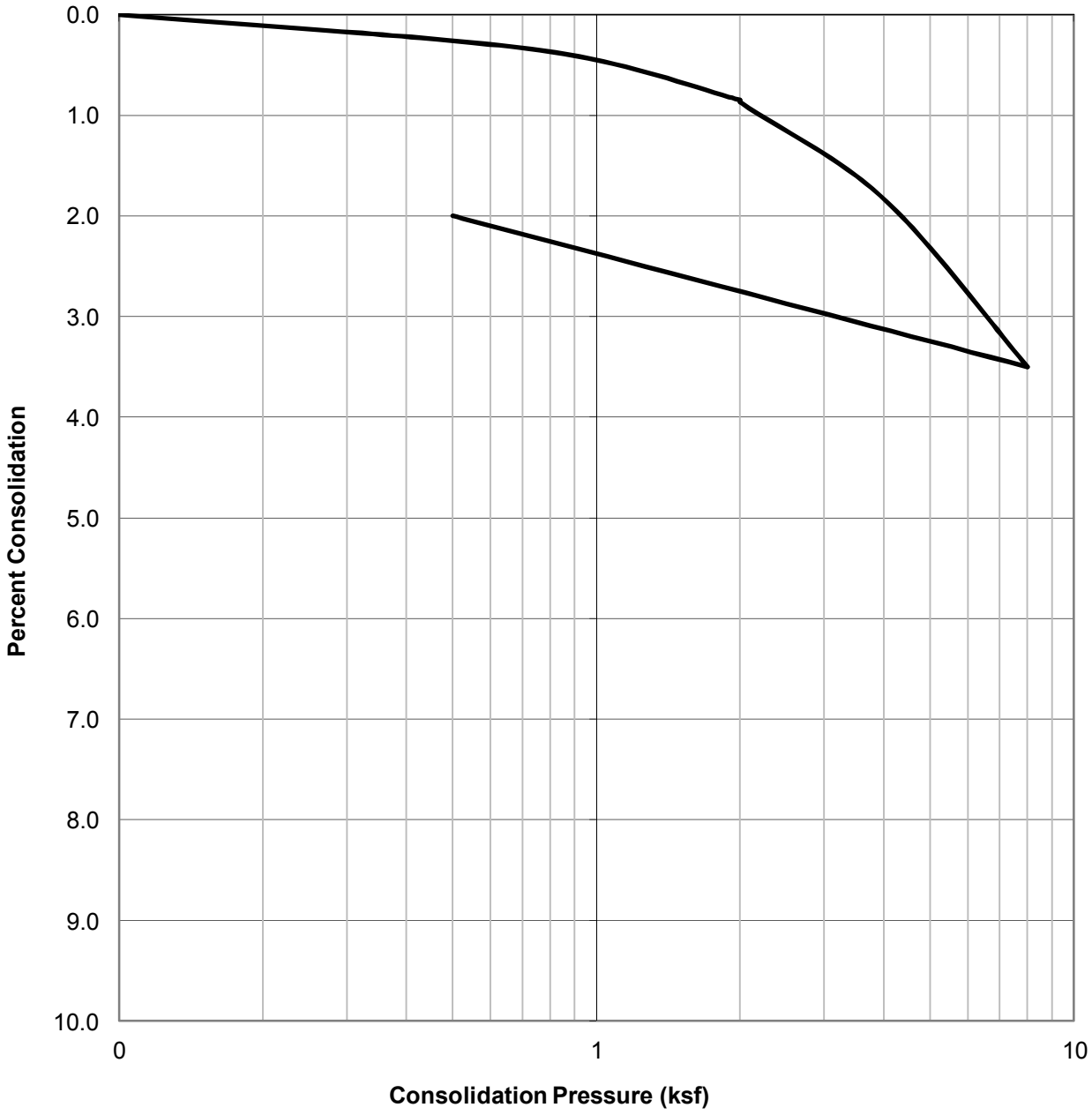
DRAFTED BY: RP | CHECKED BY: HHD

DIRECT SHEAR TEST RESULTS

1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY 2019 | PROJECT NO. A8559-06-74 | FIG. B3

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@3	Silt with Clay (CL-ML)	105.2	21.9	22.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: RP

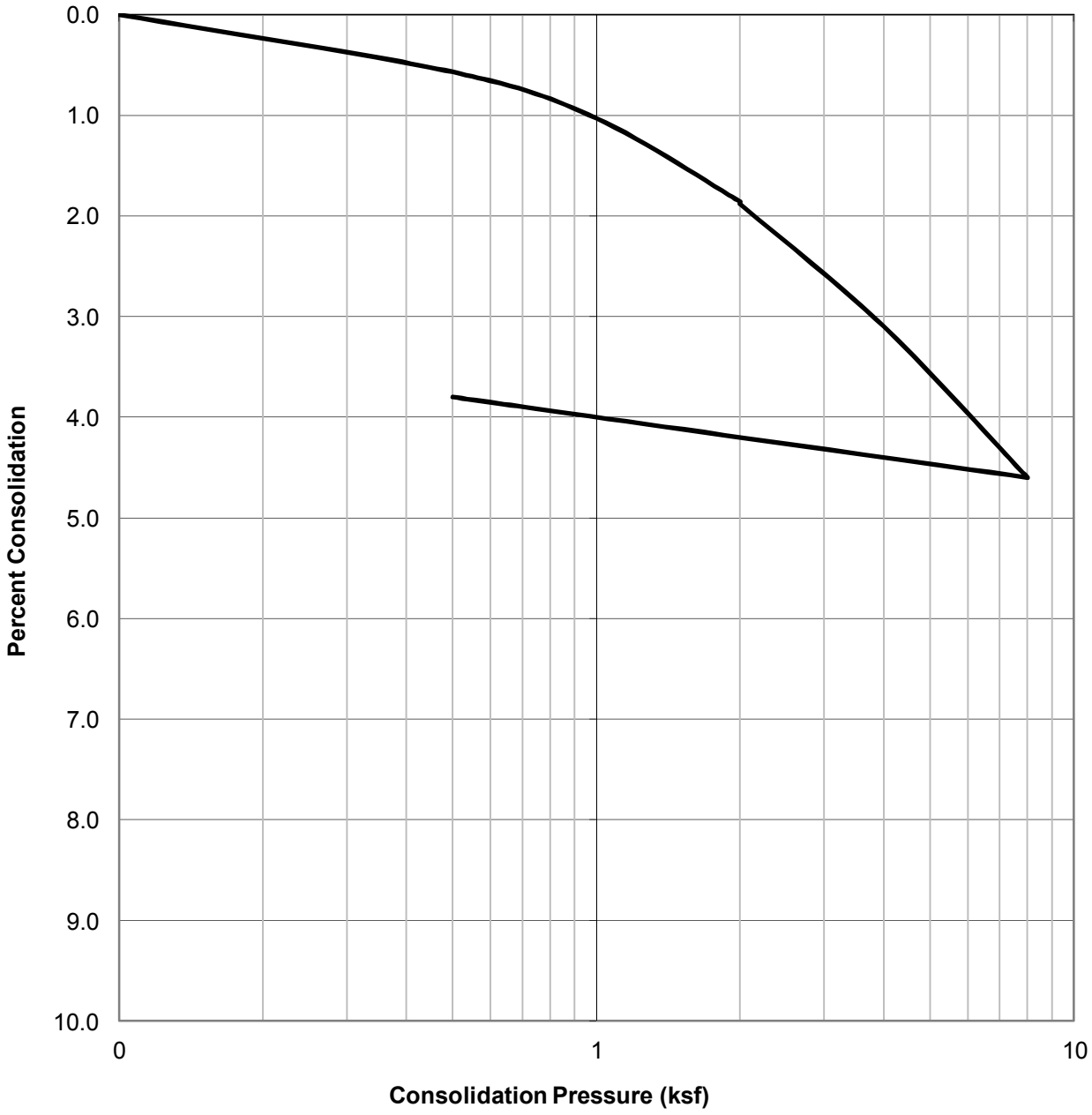
Project No.: A8559-06-74

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY

Figure B4

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B2@6	Silt with Clay (CL-ML)	91.8	29.4	27.6

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: RP

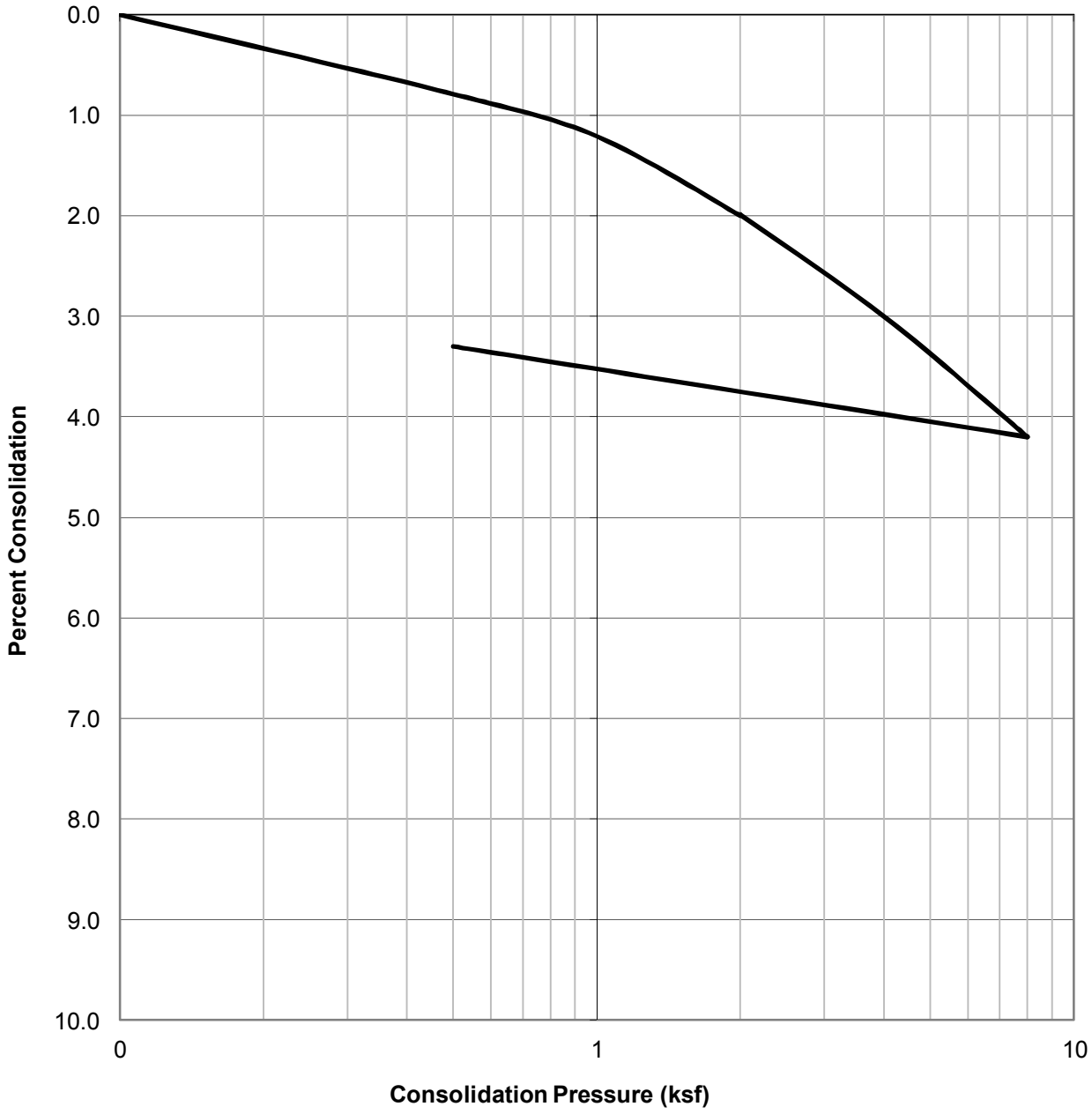
Project No.: A8559-06-74

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY

Figure B5

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B7@10	Silt with Clay (CL-ML)	102.0	23.2	22.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: RP

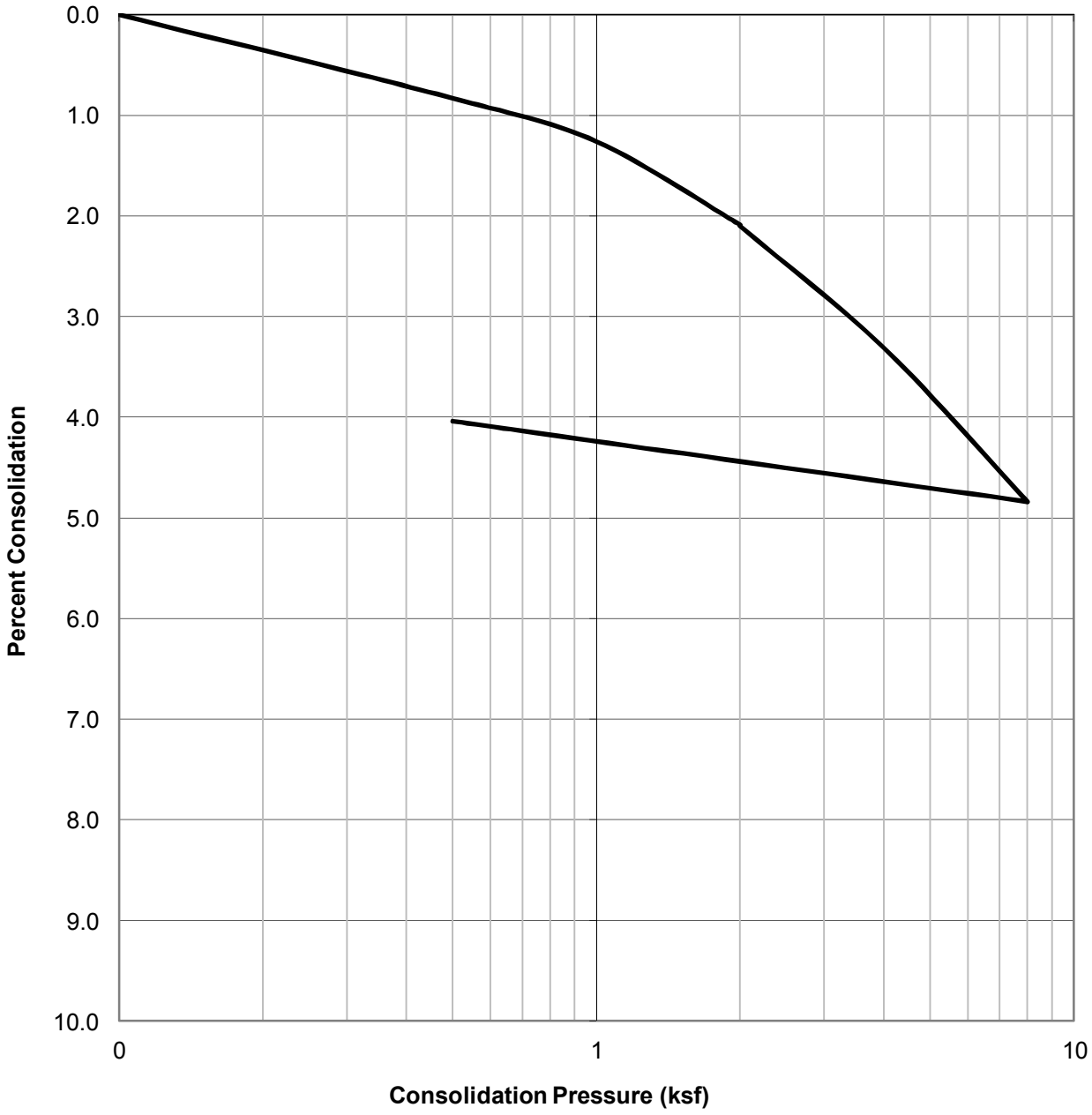
Project No.: A8559-06-74

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY

Figure B6

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B7@12.5	Silt with Clay (CL-ML)	104.0	21.7	19.9

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: RP

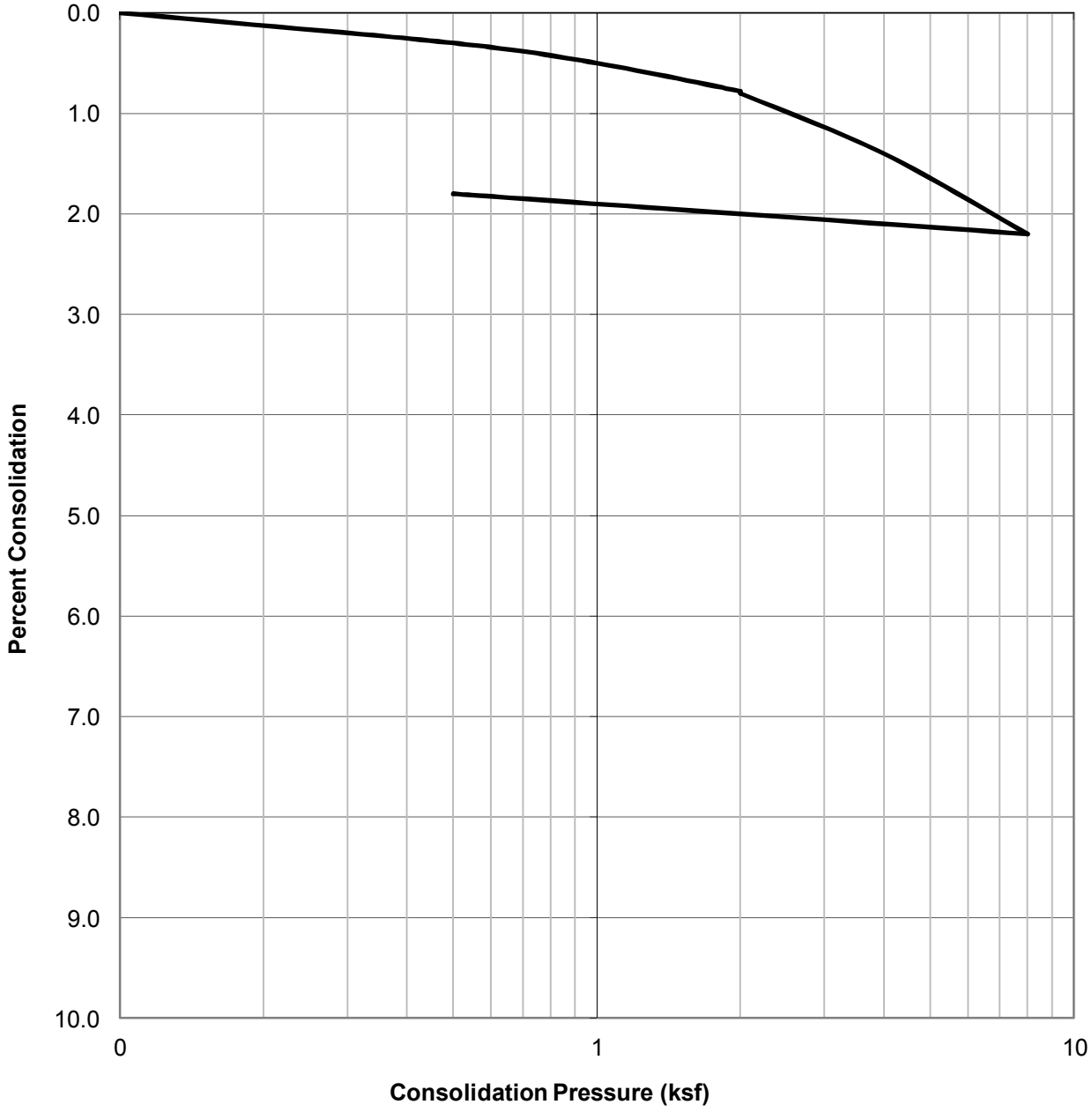
Project No.: A8559-06-74

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY

Figure B7

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B7@17.5	Siltstone	112.1	19.3	18.8

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: RP

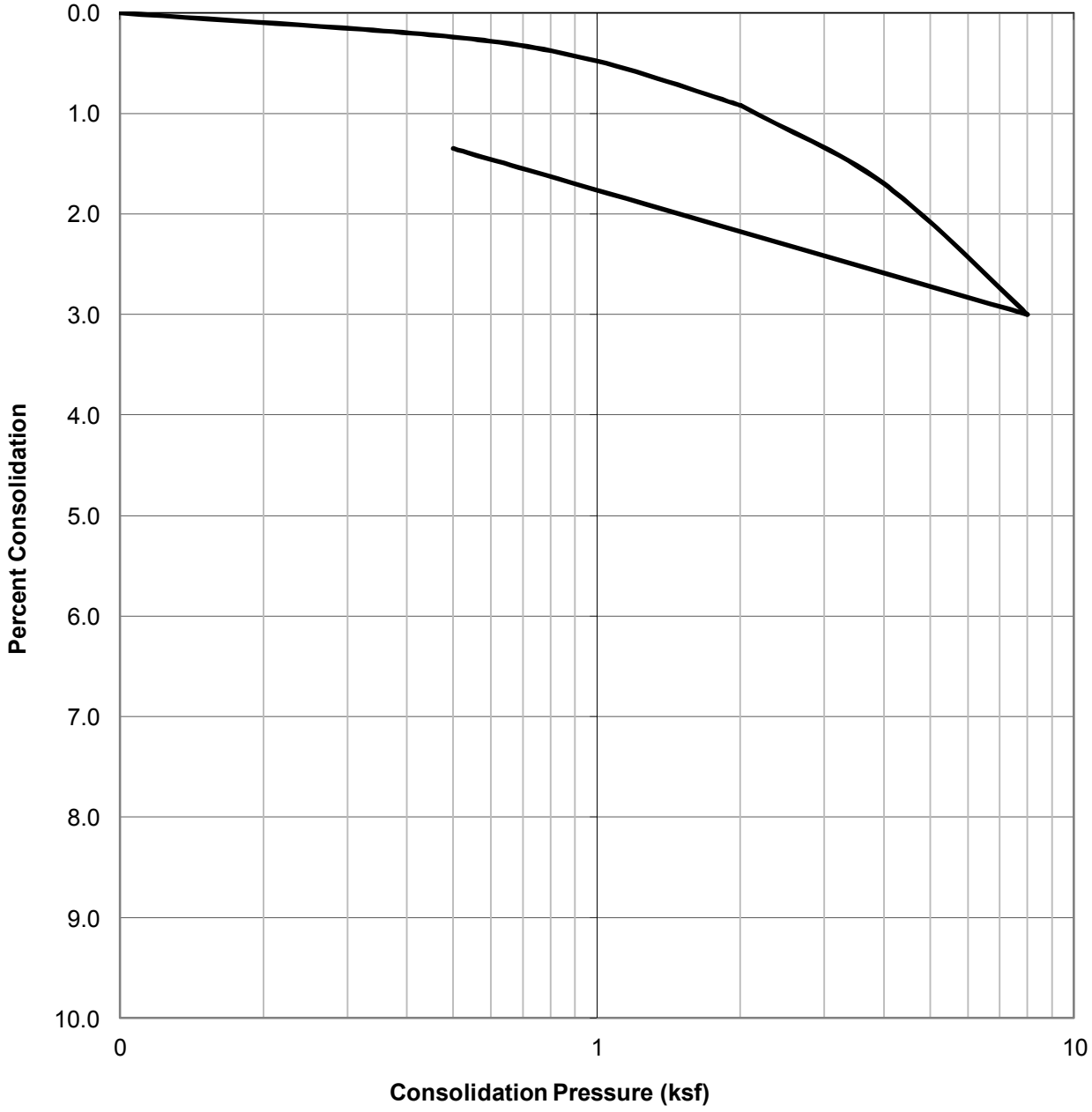
Project No.: A8559-06-74

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY

Figure B8

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B8@6	Silt with Clay (CL-ML)	110.0	18.2	19.3

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

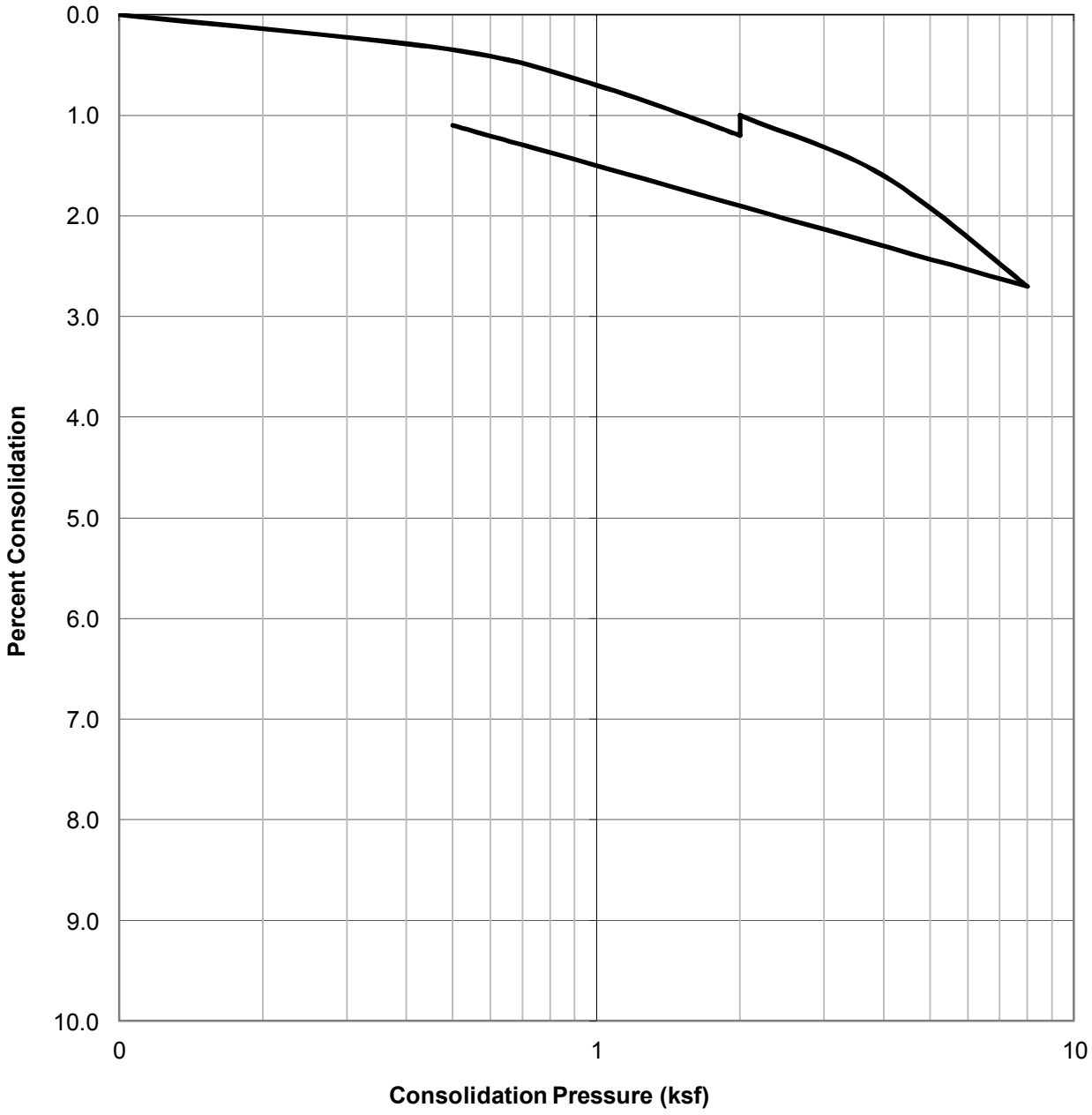
Checked by: RP

Project No.: A8559-06-74

1744 Zonal Avenue
Los Angeles, California

May 19 Figure B9

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B8@9	Silt with Clay (CL-ML)	110.2	18.6	20.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: RP

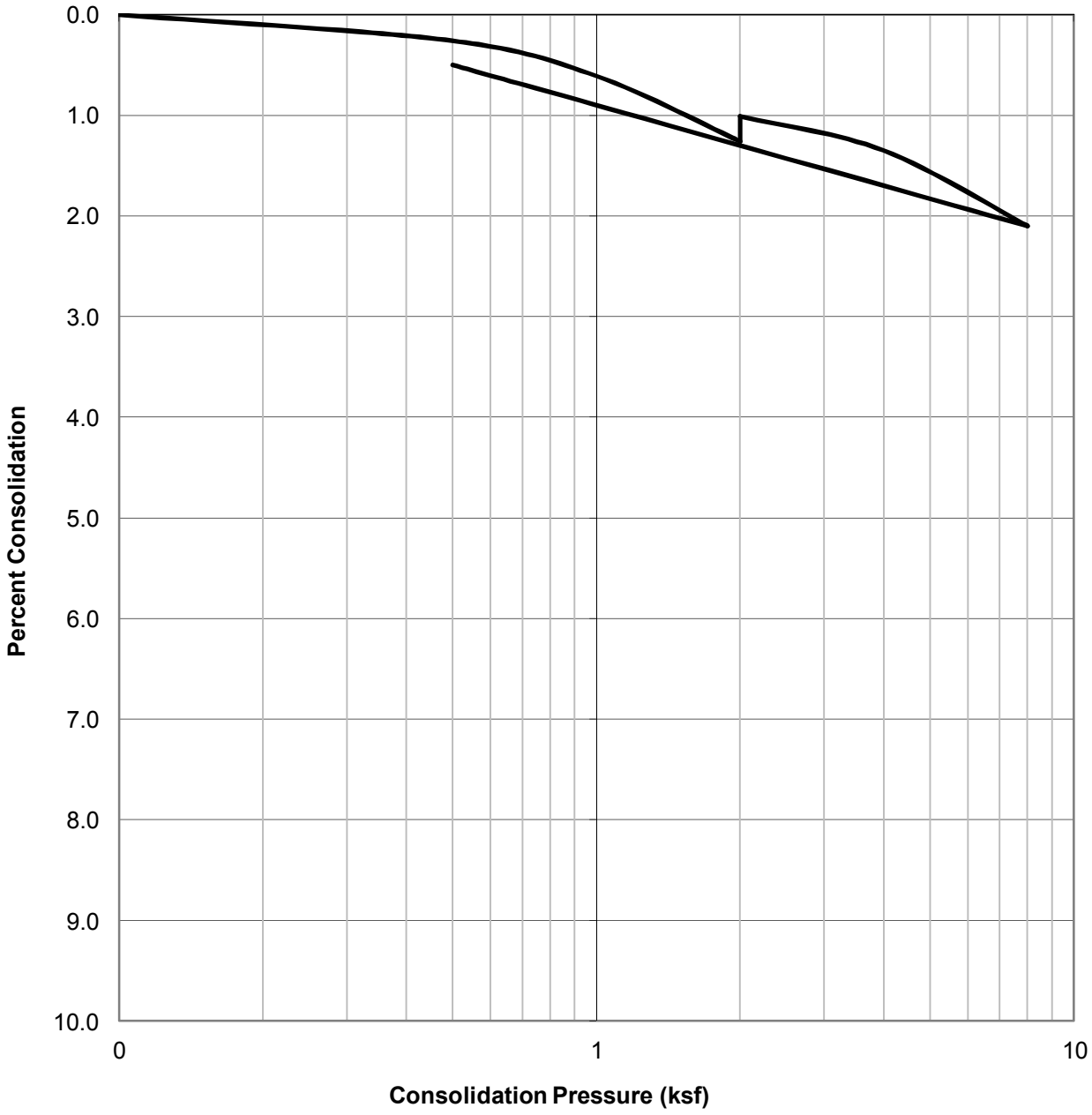
Project No.: A8559-06-74

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY

Figure B10

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
BA1@12	Silt with Clay (CL-ML)	105.7	22.0	23.5

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

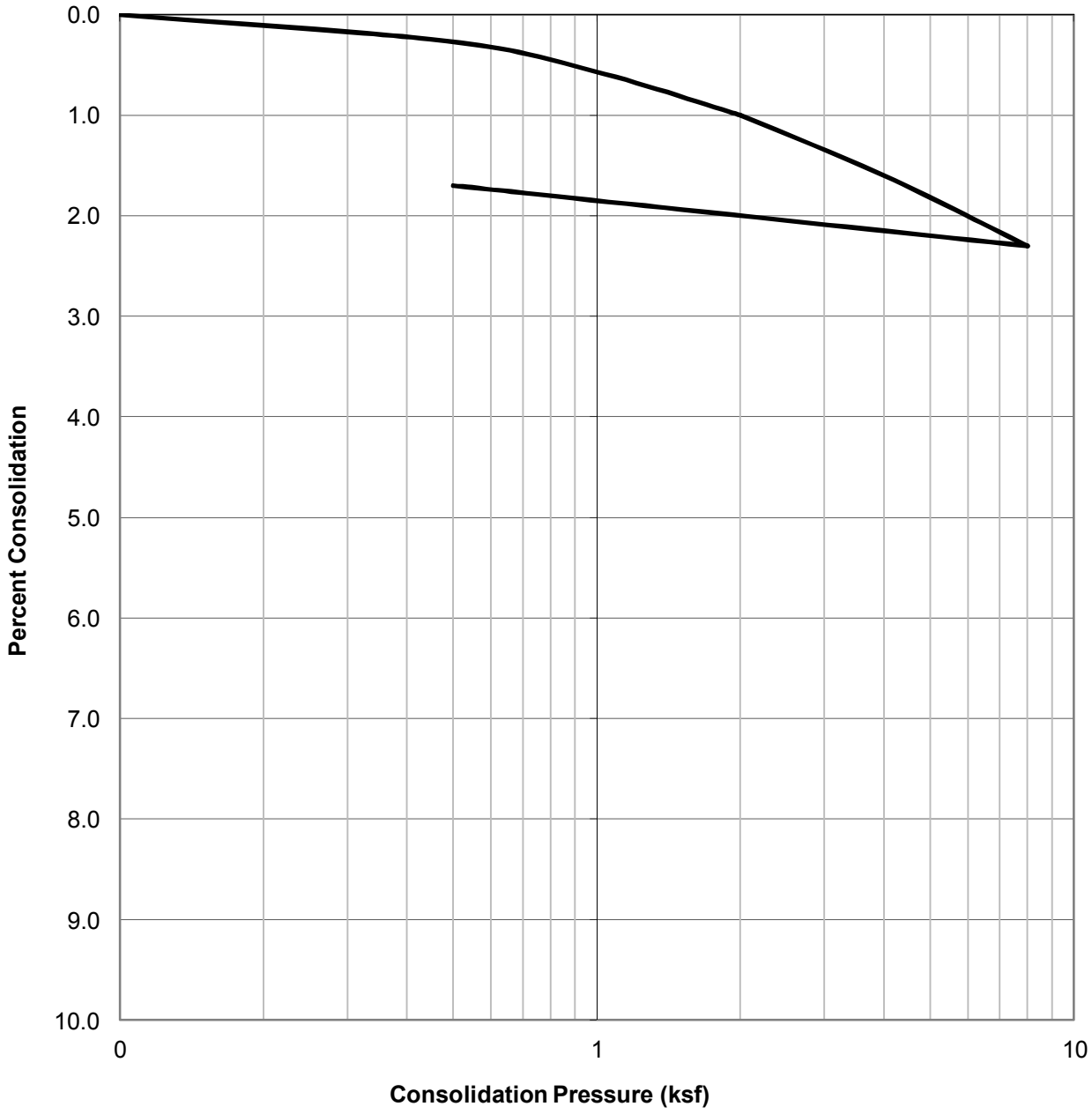
Checked by: JJK

Project No.: A8559-06-74

1744 Zonal Ave
Los Angeles, California

May Figure B11

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B8@15	Silty Sand (SM)	104.5	20.5	20.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

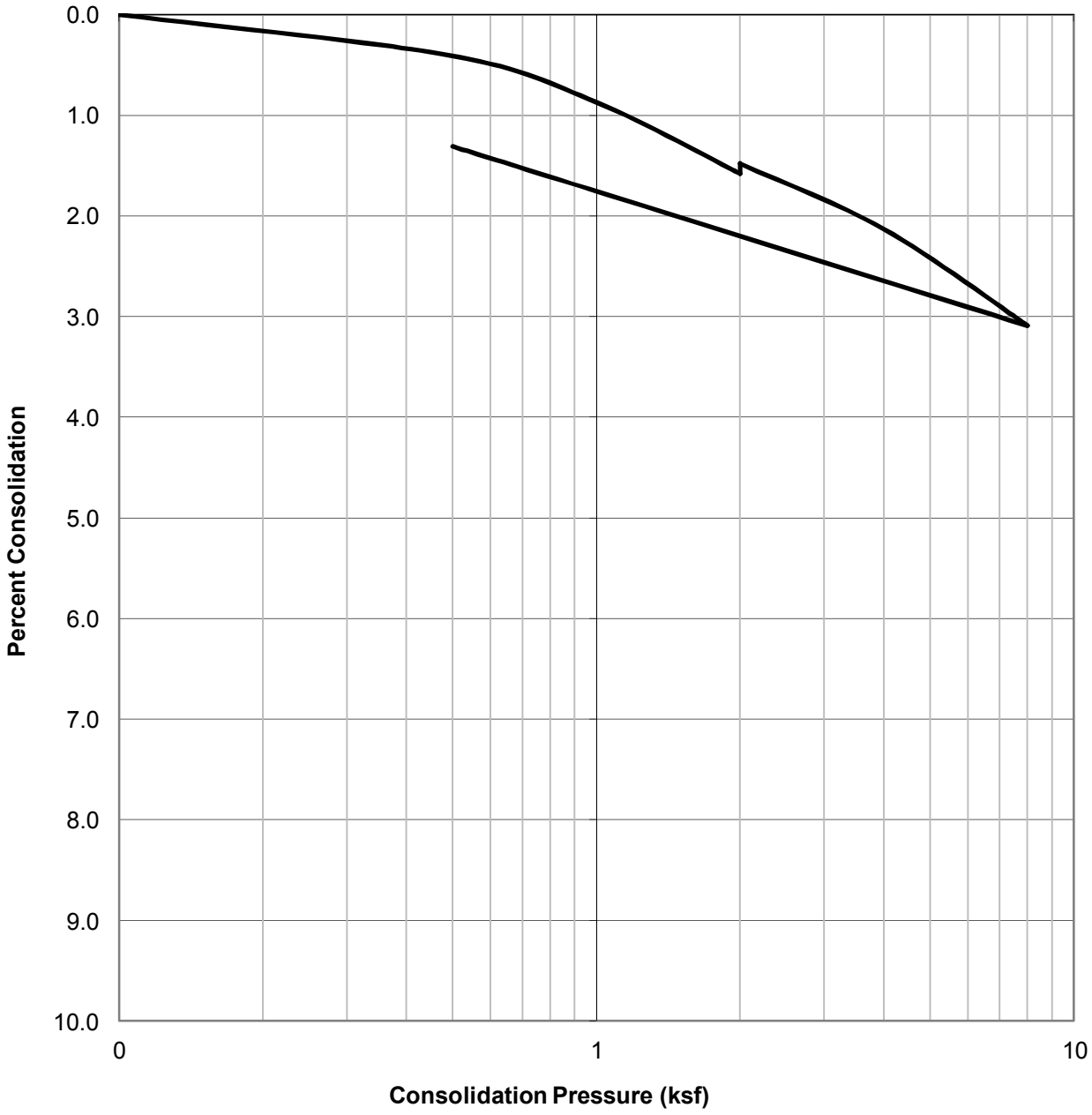
Checked by: RP

Project No.: A8559-06-74

1744 Zonal Avenue
Los Angeles, California

May Figure B12

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B9@6	Silt with Clay (CL-ML)	108.8	17.1	19.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

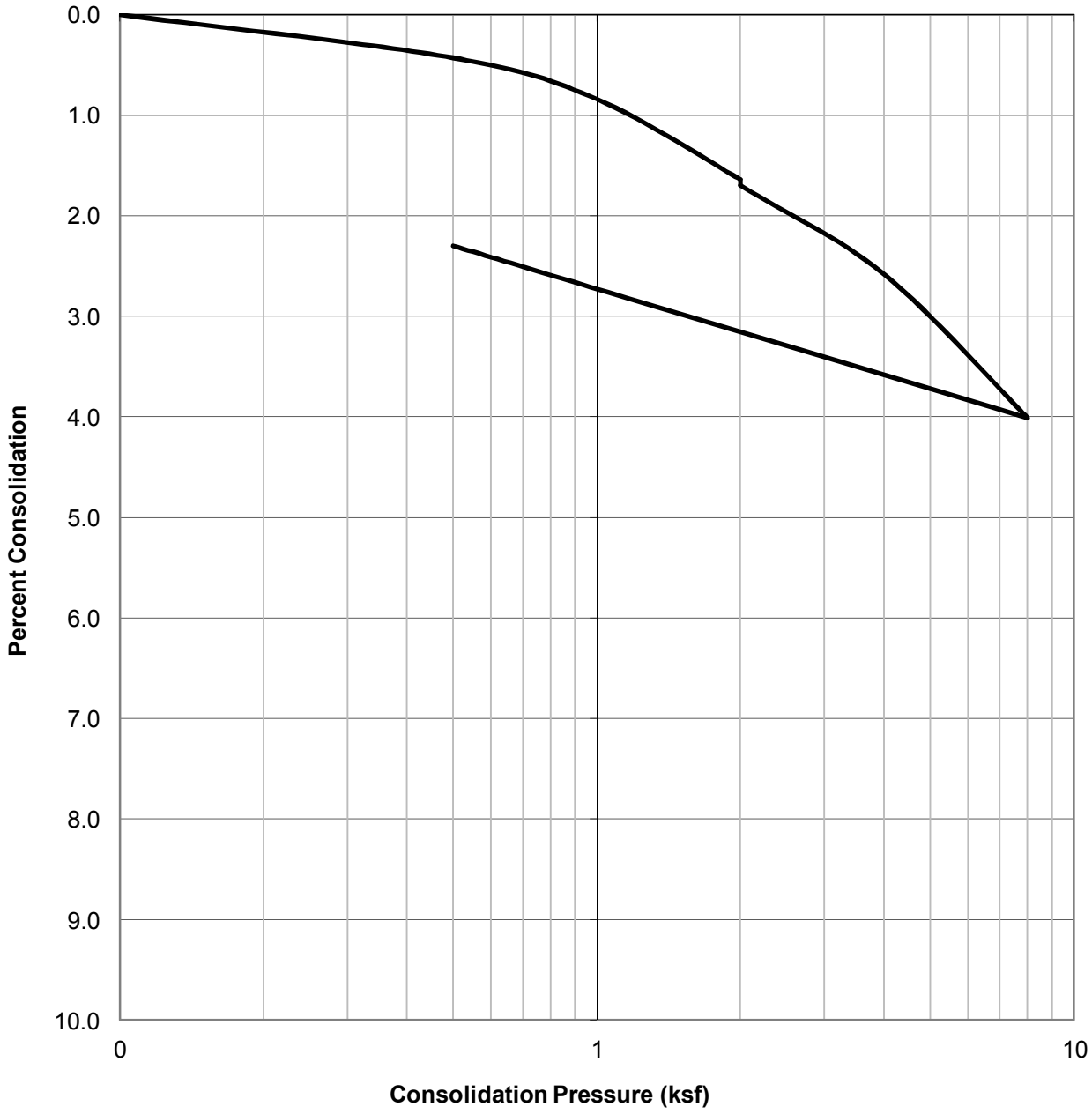
Checked by: JJK

Project No.: A8559-06-74

1744 Zonal Avenue
Los Angeles, California

May Figure B13

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B9@9	Silt with Clay (CL-ML)	106.6	20.9	20.6

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

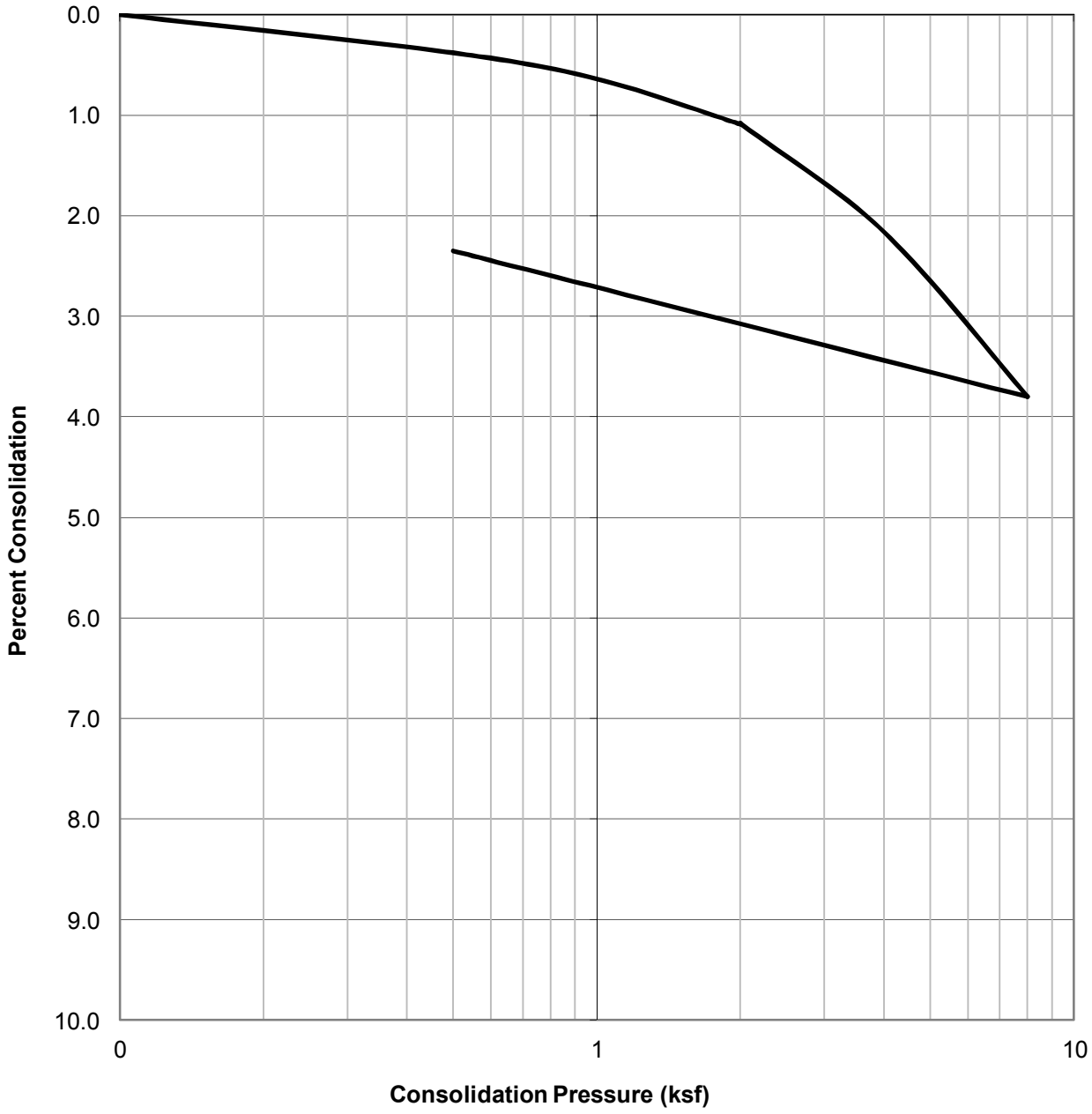
Checked by: JJK

Project No.: A8559-06-74

1744 Zonal Avenue
Los Angeles, California

May Figure B14

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B9@12	Silt with Clay (CL-ML)	103.9	23.3	22.8

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

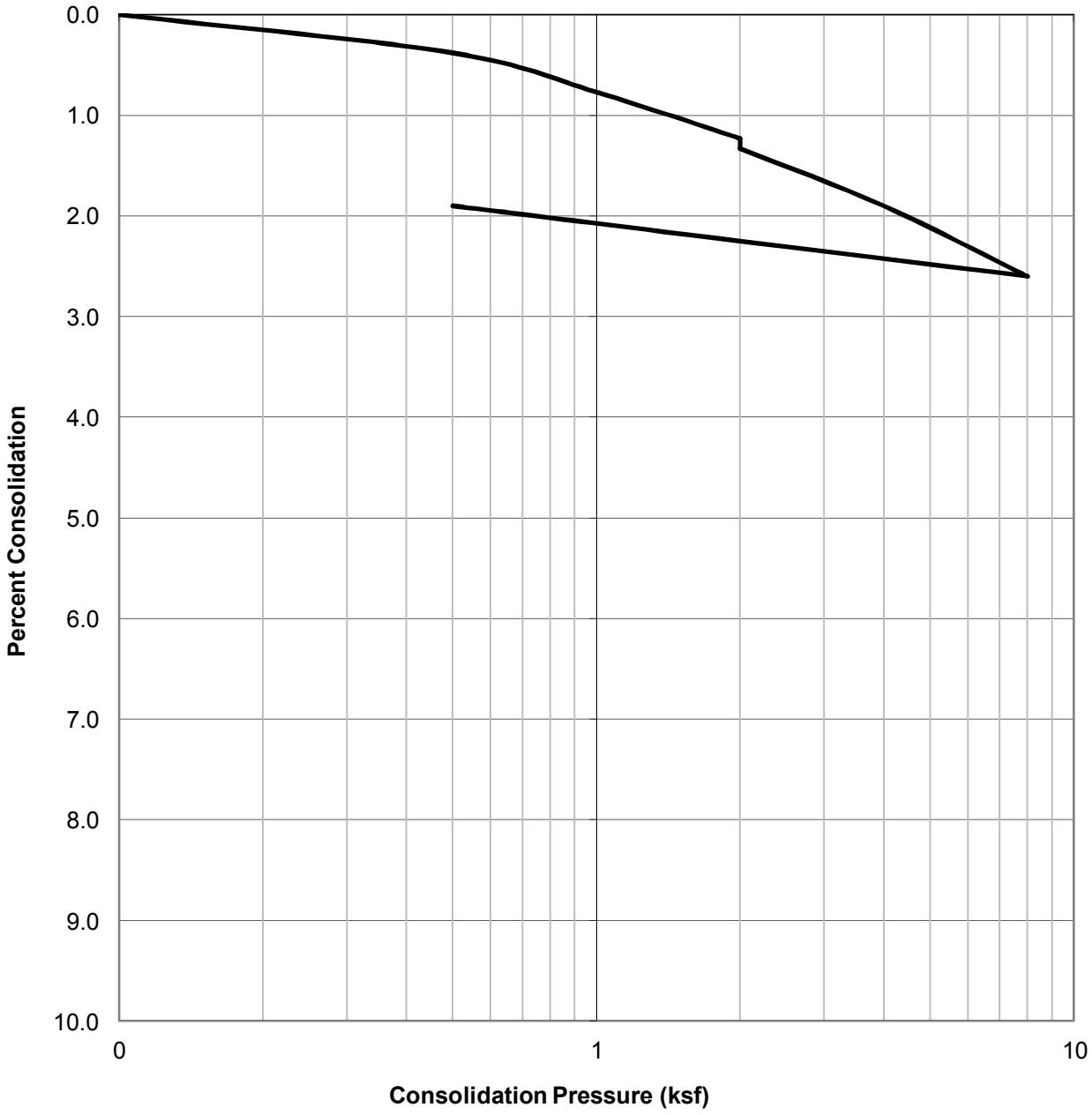
Checked by: JJK

Project No.: A8559-06-74

1744 Zonal Avenue
Los Angeles, California

May Figure B15

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B9@15	Siltstone	107.4	20.0	19.8

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

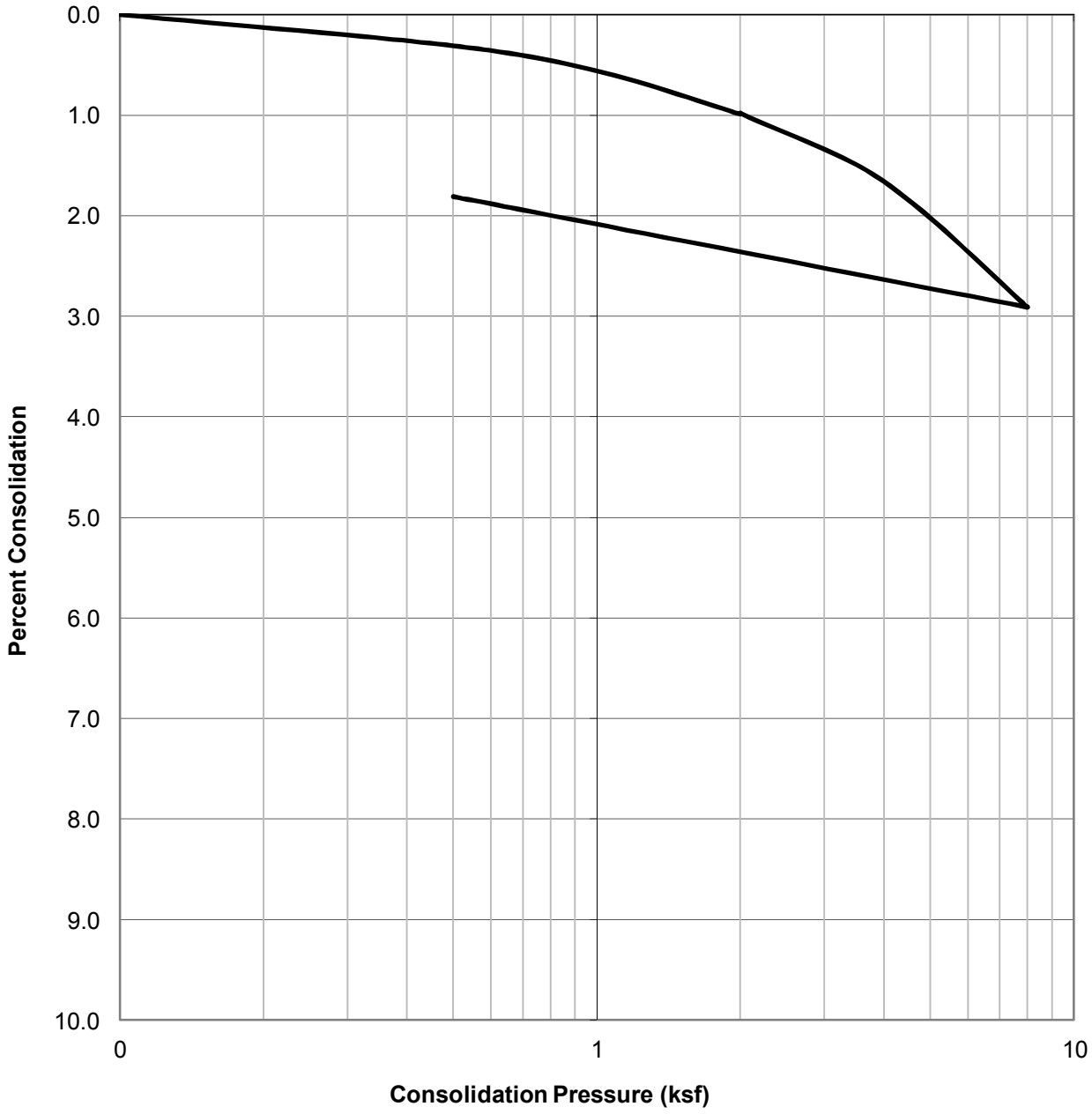
Checked by: JJK

Project No.: A8559-06-74

1744 Zonal Ave
Los Angeles, California

May Figure B16

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B10@3	Silt with Clay (CL--ML)	112.6	15.9	17.4

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: RP

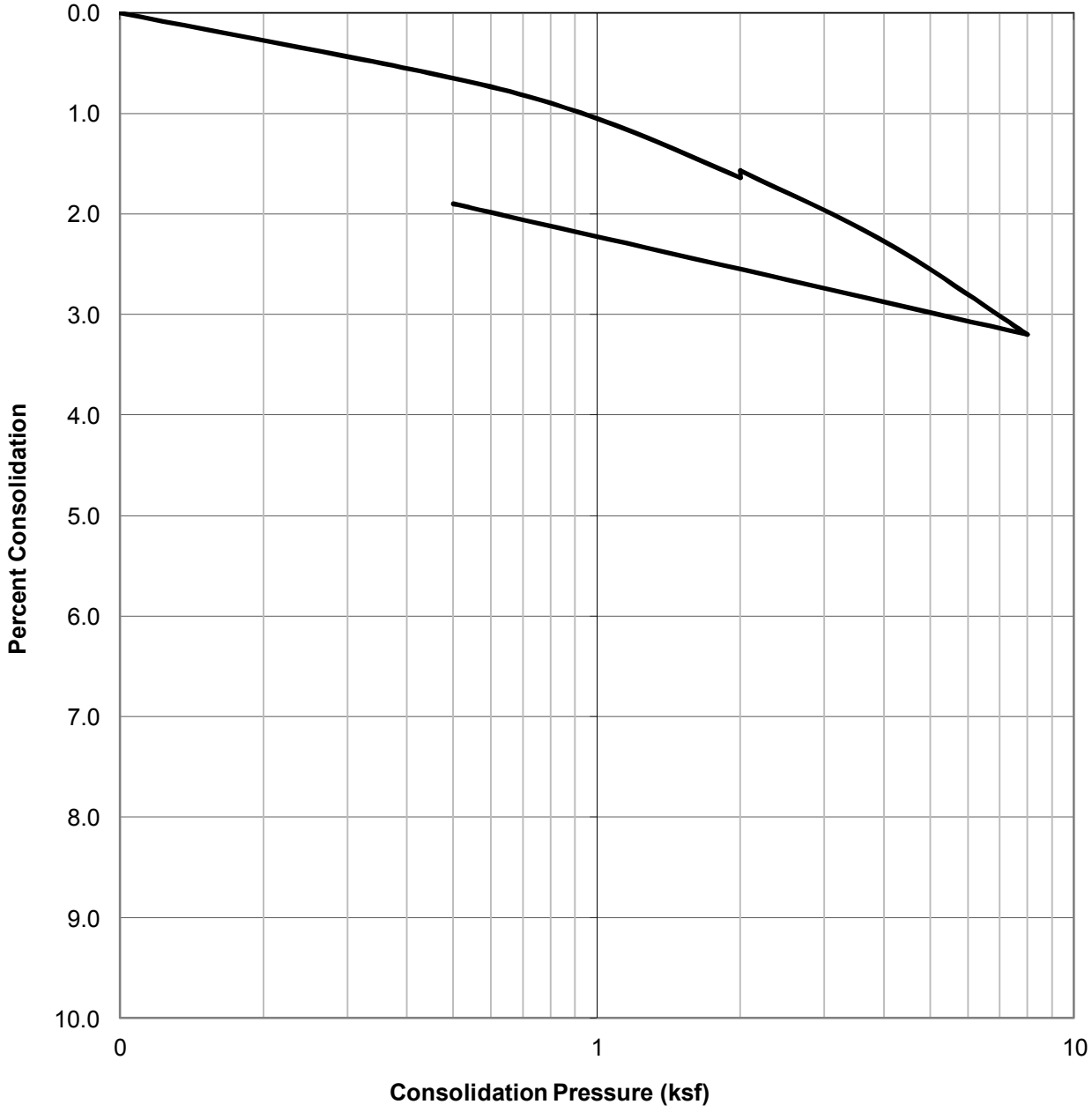
Project No.: A8559-06-74

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY

Figure B17

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B10@6	Silt with Clay (CL-ML)	103.7	22.8	23.4

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: RP

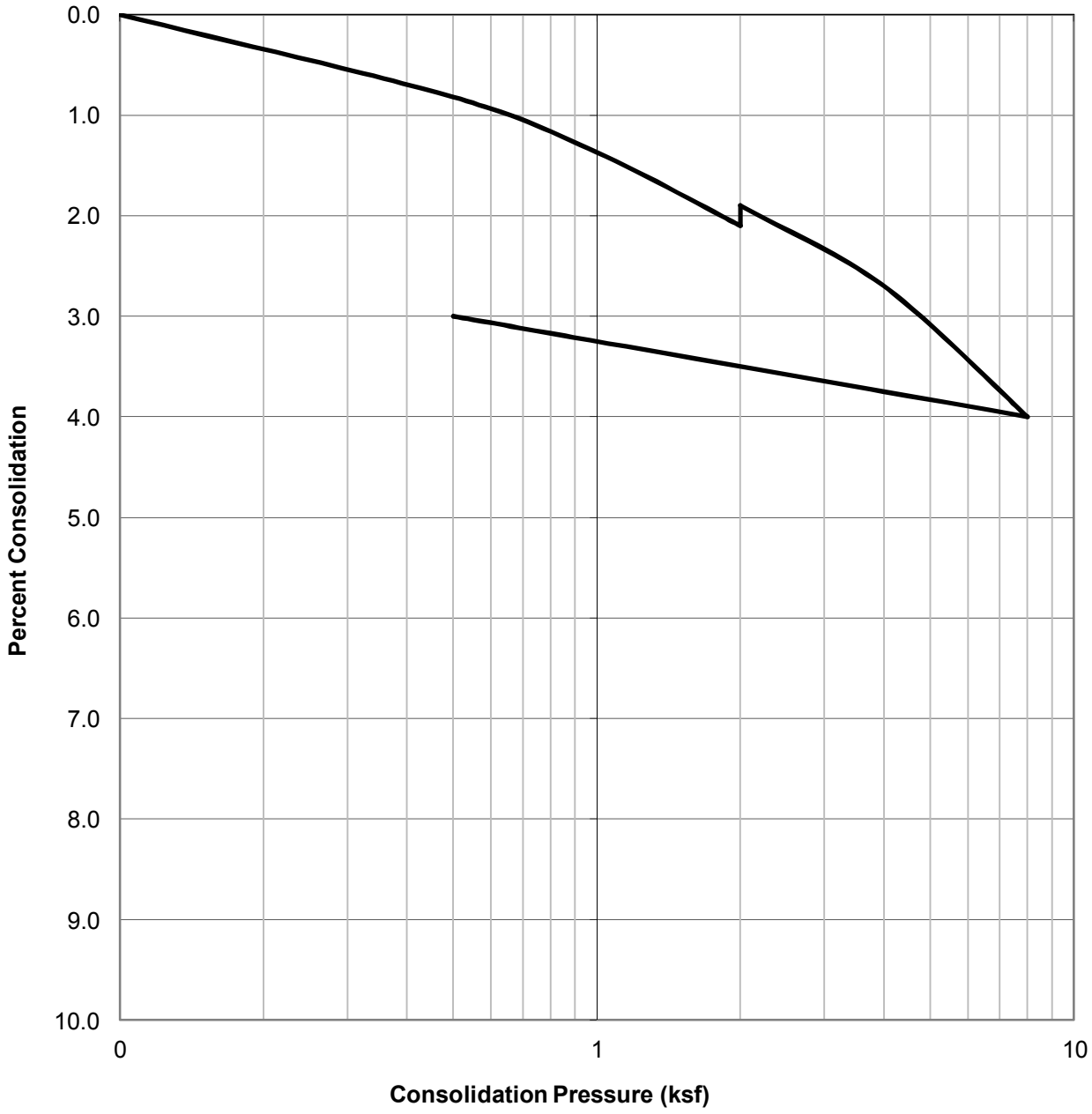
Project No.: A8559-06-74

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY

Figure B18

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B10@9	Silt with Clay (CL-ML)	101.5	24.1	23.4

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: RP

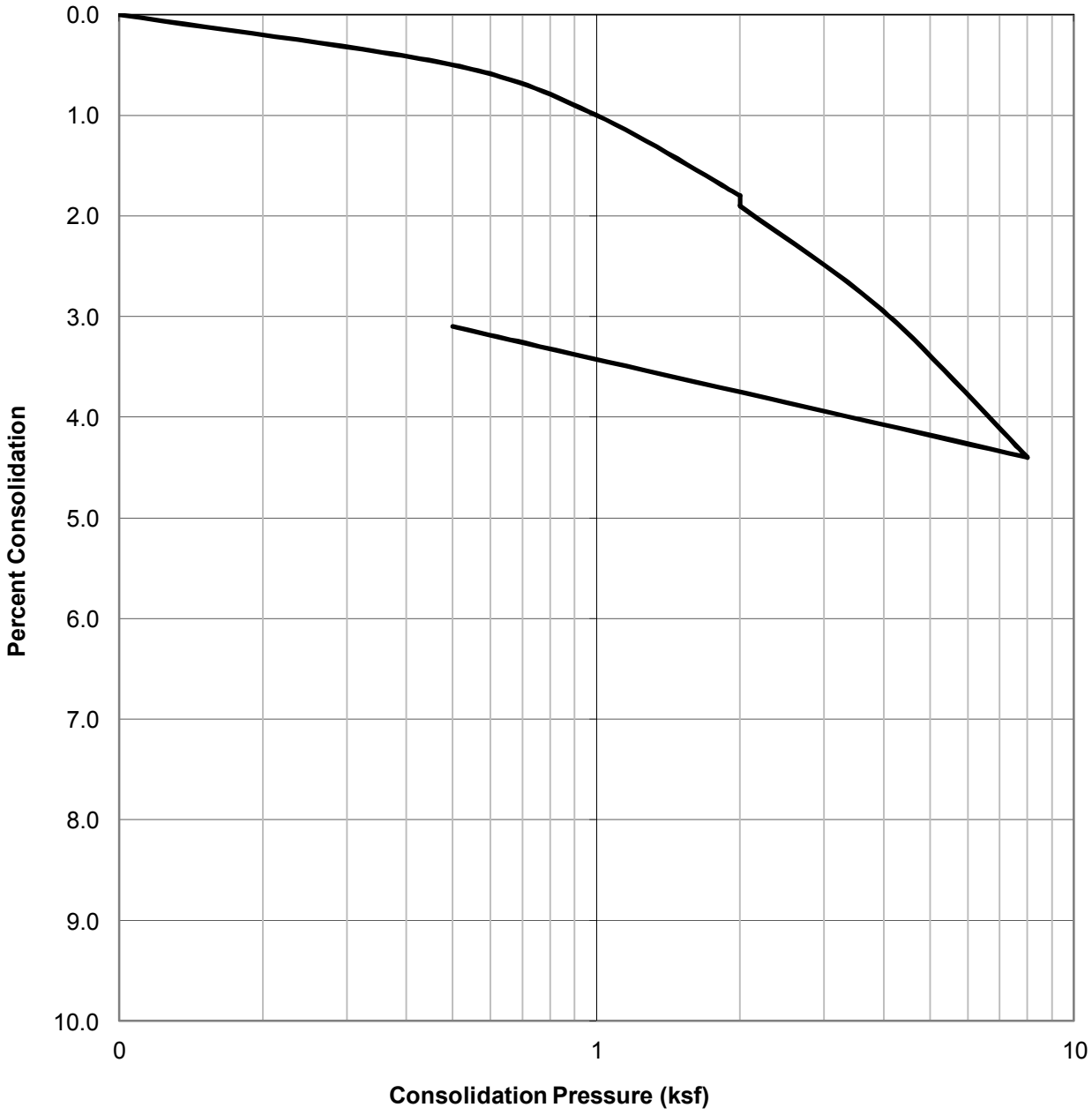
Project No.: A8559-06-74

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY

Figure B19

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
BA3@12	Silt with Clay (CL-ML)	105.2	21.8	21.2

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

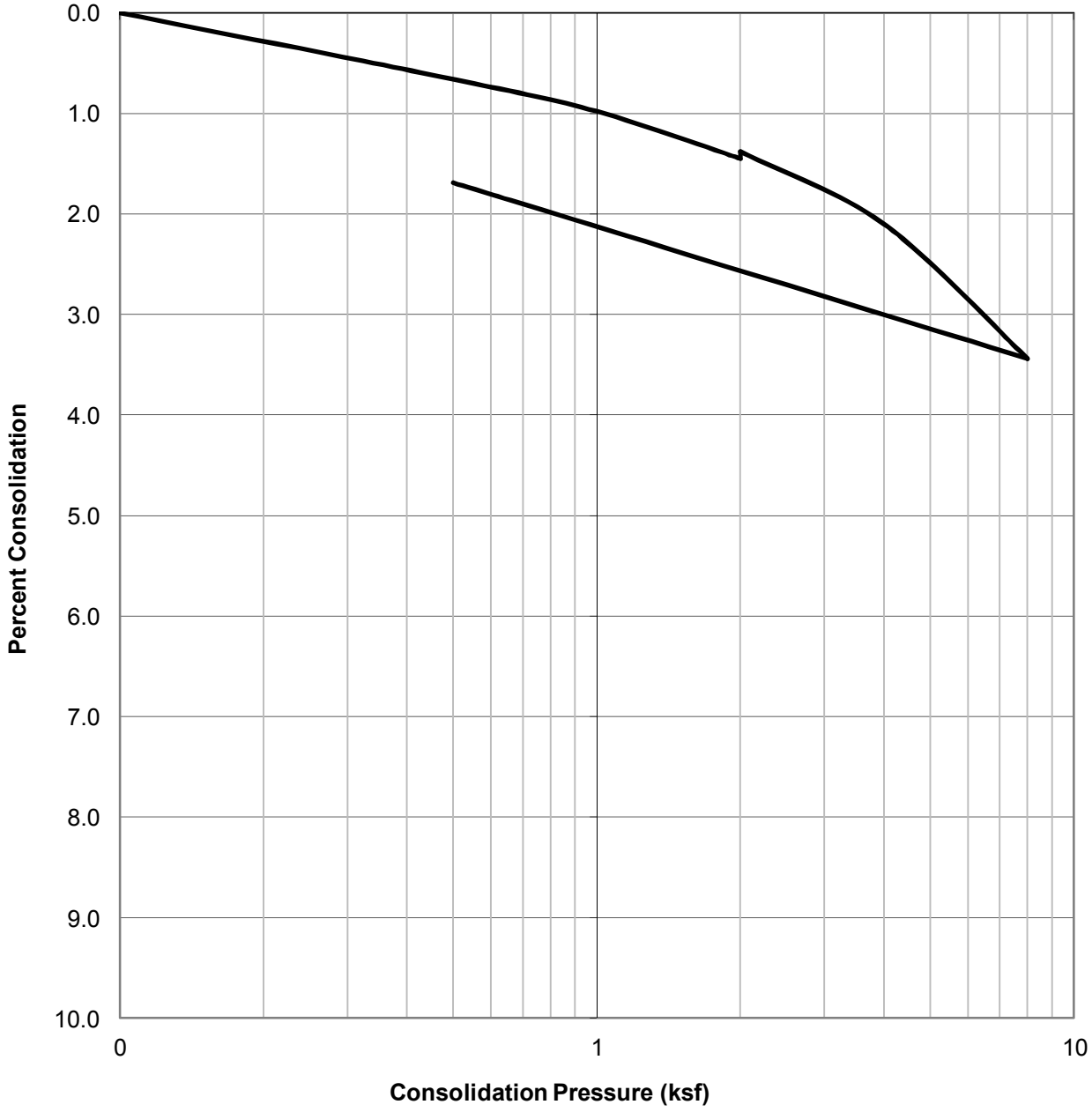
Checked by: RP

Project No.: A8559-06-74

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY Figure B20

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B10@15	Silt with Clay (CL-ML)	107.7	20.5	20.8

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: RP

Project No.: A8559-06-74

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY

Figure B21

B8@0-5'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	0.9
Wt. Comp. Soil + Mold	(gm)	756.0	809.3
Wt. of Mold	(gm)	368.4	368.4
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	650.7	809.3
Dry Wt. of Soil + Cont.	(gm)	620.0	347.9
Wt. of Container	(gm)	350.7	368.4
Moisture Content	(%)	11.4	26.8
Wet Density	(pcf)	116.9	132.8
Dry Density	(pcf)	104.9	104.8
Void Ratio		0.6	0.4
Total Porosity		0.4	0.3
Pore Volume	(cc)	78.2	51.1
Degree of Saturation	(%) [S_{meas}]	51.2	182.0

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
5/8/2019	10:00	1.0	0	0.3095
5/8/2019	10:10	1.0	10	0.3075
Add Distilled Water to the Specimen				
5/9/2019	10:00	1.0	1430	0.3695
5/9/2019	11:00	1.0	1490	0.3695

Expansion Index (EI meas) =	62
Expansion Index (Report) =	**62**

Expansion Index, EI_{50}	CBC CLASSIFICATION *	UBC CLASSIFICATION **
0-20	Non-Expansive	Very Low
21-50	Expansive	Low
51-90	Expansive	Medium
91-130	Expansive	High
>130	Expansive	Very High

* Reference: 2016 California Building Code, Section 1803.5.3

** Reference: 1997 Uniform Building Code, Table 18-I-B.

GEOCON

EXPANSION INDEX TEST RESULTS

ASTM D 4829

Checked by: RP

Project No.: A8559-06-74

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY

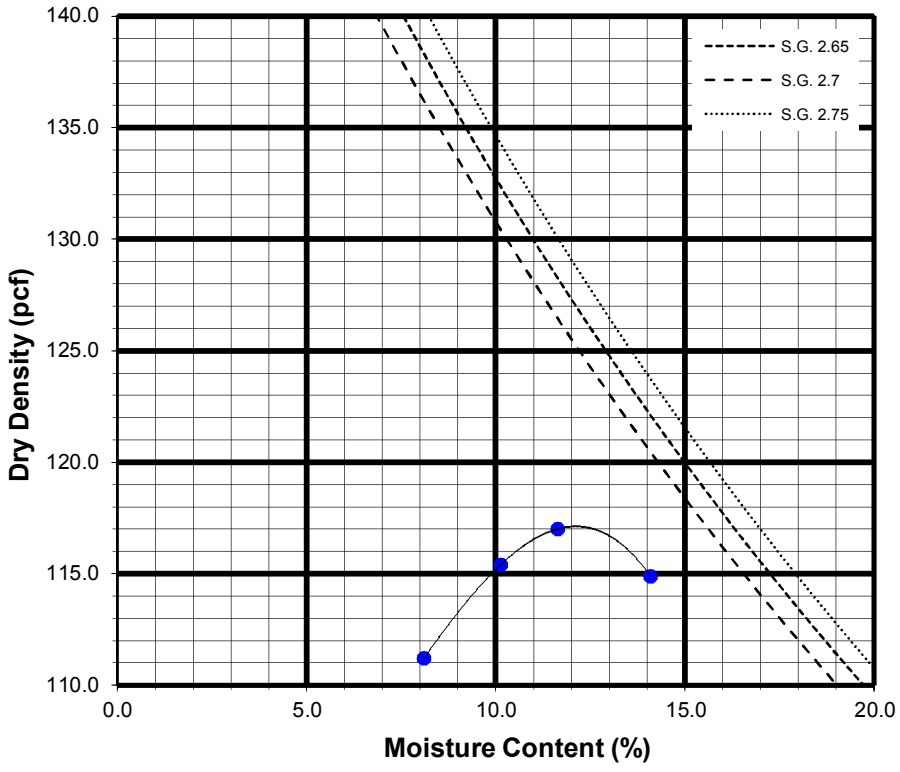
Figure B22

Sample No:

BA1@0-5'	Gray Brown Clay (CL)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	5992	6096	6149	6156		
Weight of Mold	(g)	4176	4176	4176	4176		
Net Weight of Soil	(g)	1816	1920	1973	1980		
Wet Weight of Soil + Cont.	(g)	738.7	717.4	702.3	707.3		
Dry Weight of Soil + Cont.	(g)	694.3	665.0	643.2	636.4		
Weight of Container	(g)	146.1	147.6	135.5	133.1		
Moisture Content	(%)	8.1	10.1	11.6	14.1		
Wet Density	(pcf)	120.2	127.1	130.6	131.1		
Dry Density	(pcf)	111.2	115.4	117.0	114.9		

Maximum Dry Density (pcf)	**117.0**	**Optimum Moisture Content (%)**	**11.5**



Preparation Method: A

GEOCON

MODIFIED COMPACTION TEST OF SOILS

ASTM D-1557

Checked by:

Project No.: A8559-06-74

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY

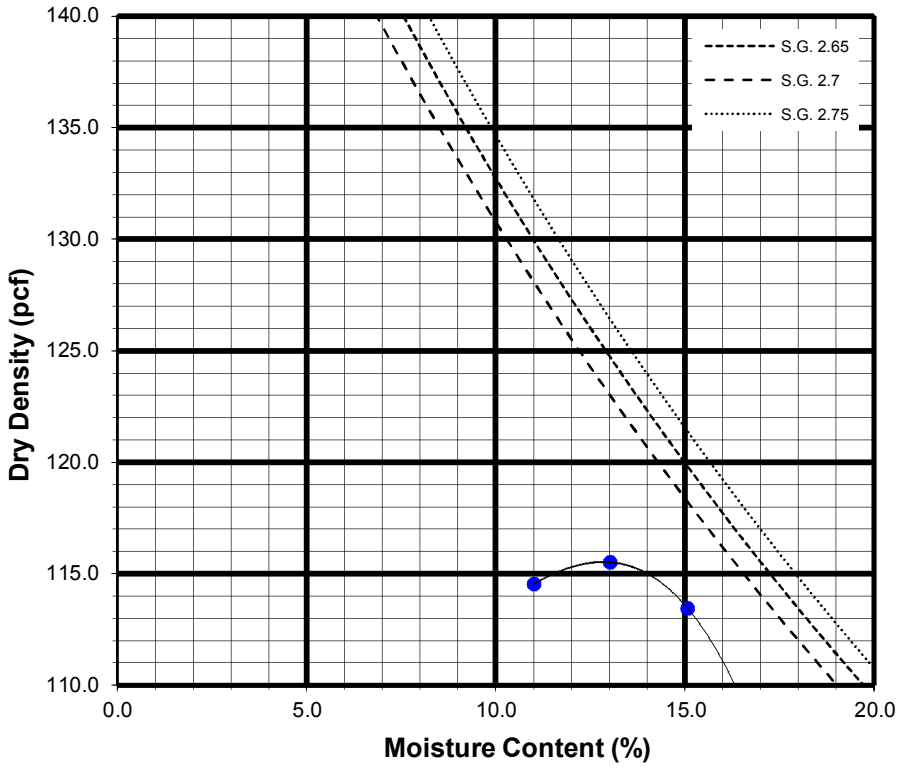
Figure B23

Sample No:

B10@5-10'	Olive Brown Silt with Clay (CL-ML)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6099	6150	6150	6074		
Weight of Mold	(g)	4178	4178	4178	4178		
Net Weight of Soil	(g)	1921	1972	1972	1896		
Wet Weight of Soil + Cont.	(g)	787.7	781.3	773.0	763.4		
Dry Weight of Soil + Cont.	(g)	724.1	708.1	691.1	672.1		
Weight of Container	(g)	146.1	145.9	147.7	135.0		
Moisture Content	(%)	11.0	13.0	15.1	17.0		
Wet Density	(pcf)	127.2	130.6	130.6	125.5		
Dry Density	(pcf)	114.5	115.5	113.5	107.3		

Maximum Dry Density (pcf)	**116.0**	**Optimum Moisture Content (%)**	**13.5**



Preparation Method: A

GEOCON

MODIFIED COMPACTION TEST OF SOILS

ASTM D-1557

Checked by:

Project No.: A8559-06-74

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY

Figure B24

SUMMARY OF LABORATORY POTENTIAL
OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
CALIFORNIA TEST NO. 643

Sample No.	pH	Resistivity (ohm centimeters)
B8 @ 0-5	7.66	710 (Severely Corrosive)
B10 @ 0-5	7.66	1200 (Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B8 @ 0-5	0.009
B10 @ 0-5	0.008

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B8 @ 0-5	0.037	S0
B10 @ 0-5	0.012259964	S0

GEOCON

CORROSIVITY TEST RESULTS

Checked by: RP

Project No.: A8559-06-74

1744 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

MAY Figure B25

APPENDIX C

APPENDIX C

GEOTECHNICAL INVESTIGATION DATED APRIL 9, 2018

GEOTECHNICAL INVESTIGATION

PROPOSED HEALTH AGENCY RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

TRACT: 2082, LOT: LT1

GEOCON
WEST, INC.
GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR
KFA, LP
SANTA MONICA, CALIFORNIA

PROJECT NO. A9733-06-01

APRIL 9, 2018

Project No. A9733-06-01
April 9, 2018

Mr. Joe Killefer
KFA, LP
1625 Olympic Boulevard
Santa Monica, California 90404

Subject: GEOTECHNICAL INVESTIGATION
PROPOSED HEALTH AGENCY RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA
TRACT: TR 2082 LOT: LT 1

Dear Mr. Killefer:

In accordance with your authorization of our proposal dated December 7, 2017, we have performed a geotechnical investigation for the proposed health agency recuperative campus located at 1774 Zonal Avenue in the City of Los Angeles, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations of this report are followed and implemented during design and construction.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned.

Very truly yours,

GEOCON WEST, INC.

REGISTERED PROFESSIONAL ENGINEER PETRINA ZEN C87489 CIVIL STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER H. HARRY DERKALOUSDIAN C 79694 CIVIL STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST SUSAN FRANZEN KIRKGARD No. 1754 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

Petrina Zen	Harry Derkalousdian	Susan F. Kirkgard
PE 87489	PE 79694	CEG 1754

(EMAIL) Addressee

TABLE OF CONTENTS

1. PURPOSE AND SCOPE 1
2. SITE AND PROJECT DESCRIPTION 1
3. GEOLOGIC SETTING 2
4. SOIL AND GEOLOGIC CONDITIONS 2
   - 4.1 Artificial Fill 2
   - 4.2 Alluvium 3
   - 4.3 Puente Formation 3
5. GROUNDWATER 3
6. GEOLOGIC HAZARDS 3
   - 6.1 Surface Fault Rupture 3
   - 6.2 Seismicity 5
   - 6.3 Seismic Design Criteria 5
   - 6.4 Liquefaction Potential 7
   - 6.5 Slope Stability 8
   - 6.6 Earthquake-Induced Flooding 9
   - 6.7 Tsunamis, Seiches, and Flooding 9
   - 6.8 Oil Fields & Methane Potential 9
   - 6.9 Subsidence 10
7. CONCLUSIONS AND RECOMMENDATIONS 11
   - 7.1 General 11
   - 7.2 Soil and Excavation Characteristics 13
   - 7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate 13
   - 7.4 Grading 14
   - 7.5 Controlled Low Strength Material (CLSM) 17
   - 7.6 Shrinkage 18
   - 7.7 Foundation Design 18
   - 7.8 Foundation Settlement 19
   - 7.9 Miscellaneous Foundations 20
   - 7.10 Lateral Design 20
   - 7.11 Concrete Slabs-on-Grade 20
   - 7.12 Preliminary Pavement Recommendations 22
   - 7.13 Retaining Walls Design 23
   - 7.14 Retaining Wall Drainage 24
   - 7.15 Temporary Excavations 25
   - 7.16 Surface Drainage 26
   - 7.17 Plan Review 26

LIMITATIONS AND UNIFORMITY OF CONDITIONS

LIST OF REFERENCES

MAPS, TABLES, AND ILLUSTRATIONS
- Figure 1, Vicinity Map
- Figure 2, Site Plan
- Figure 3, Regional Fault Map
- Figure 4, Regional Seismicity Map
- Figures 5 and 6, DE Empirical Estimation of Liquefaction Potential
- Figures 7 and 8, MCE Empirical Estimation of Liquefaction Potential
- Figures 9 and 10, Retaining Wall Drain Detail

TABLE OF CONTENTS (Continued)

APPENDIX A
FIELD INVESTIGATION
Figures A1 through A5, Boring Logs

APPENDIX B
LABORATORY TESTING
Figures B1 and B2, Direct Shear Test Results
Figures B3 and B4, Consolidation Test Results
Figure B5, Atterberg Limits Test Results
Figure B6, Grain Size Analysis Test Results
Figure B7, Laboratory Test Results
Figure B8, Corrosivity Test Results

GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed health agency recuperative campus located at 1774 Zonal Avenue in the City of Los Angeles, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on March 6, 2018, by excavating five 8-inch diameter borings to depths of approximately 15½ and 56 feet below the existing ground surface utilizing a truck-mounted hollow-stem auger drilling machine. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 1774 Zonal Avenue in the City of Los Angeles, California. The site is a rectangular-shaped parcel bounded by Zonal Avenue to the north, by a paved surface parking lot to the east, by Women's and Children's Hospital campus to the west, and by a paved surface parking lot for a commercial development to the south. An approximately 40-foot-high, 2:1 (H:V gradient) graded slope ascends from the southeast corner of the site to the entrance drive for the University of Southern California Health Sciences Campus located east-southeast of the site. The site is predominantly vacant with the exception of a temporary trailer structure located in the southwest portion of the parcel. Topography at the site is relatively level, with no pronounced highs or lows. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to the city streets. Vegetation onsite consists of native grass and weeds in localized areas.

Based on the information provided by the Client, it is our understanding that the proposed development will consist of a three-story recuperative center structure, and four two-story crisis residential treatment center structures (see Site Plan, Figure 2). All structures will utilize shipping containers as the primary structures and will be constructed at or near present grade.

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structures will be up to 300 kips, and wall loads will be up to 4 kips per linear foot.

Once the design phase and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Any changes in the design, location or elevation of any structure, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

3. GEOLOGIC SETTING

The site is located in the northeastern portion of the Los Angeles Basin, along the southwestern flank of the Repetto Hills. The Los Angeles Basins is a coastal plain bounded by the Santa Monica Mountains on the north, the Elysian Hills and Repetto Hills on the northeast, the Puente Hills and Whittier Fault on the east, the Palos Verdes Peninsula and Pacific Ocean on the west and south, and the Santa Ana Mountains and San Joaquin Hills on the southeast. The basin is underlain by a deep structural depression which has been filled by both marine and continental sedimentary deposits underlain by a basement complex of igneous and metamorphic composition. Regionally, the site is located within the northern portion of the Peninsular Ranges geomorphic province. This geomorphic province is characterized by northwest-trending physiographic and geologic features such as the Newport-Inglewood Fault Zone and the Whittier Fault Zone located approximately 9.5 miles to the southwest and the 9.8 miles to the southeast.

4. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and Quaternary age alluvial deposits consisting of sand, silt, and gravel (Dibblee, 1989; California Geological Survey, 2012). The Quaternary age sediments are in turn underlain by sedimentary bedrock of the Miocene age Puente Formation (Lamar, 1970; Dibblee, 1989). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our field explorations to a maximum depth of 1.5 feet below existing ground surface. The artificial fill generally consists of brown to dark brown, fine- to medium-grained silty sand with some fine to coarse gravel. The artificial fill is characterized as dry and loose. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Alluvium

Quaternary age alluvium was encountered beneath the fill. The alluvium generally consists of light brown to dark brown, yellowish brown or olive brown silty sand, sandy silt and silt. The alluvial soils are primarily fine- to medium-grained, slightly moist to wet and medium dense to dense or soft to hard.

4.3 Puente Formation

Miocene age sedimentary bedrock of the Puente Formation was encountered in borings B1 and B5 at depths of approximately 21½ and 14 feet beneath the existing ground surface, respectively. The bedrock consists of interbedded siltstone and fine-grained sandstone that is thinly bedded, soft and highly weathered.

5. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Los Angeles Quadrangle (California Division of Mines and Geology [CDMG], 1998) indicates the historically highest groundwater level in the area is approximately 20 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was encountered in boring B1 at a depth of approximately 17 feet below the existing ground surface. Based on the depth to groundwater encountered in our borings, the reported historic high groundwater levels (CDMG, 1998) and the depth of proposed construction, groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 7.16).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (CGS, 2018a). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years), but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 2018b; CGS, 2017) or a city-designated Preliminary Fault Rupture Study Area (City of Los Angeles, 2018) for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Raymond Fault located approximately 4.0 miles to the north (Ziony and Jones, 1989). Other nearby active faults include the Hollywood Fault, the Verdugo Fault, the Newport-Inglewood Fault Zone, and the Whittier Fault Zone located approximately 4.2 miles north-northwest, 5.6 miles north, 9.5 miles southwest, and 9.8 miles southeast of the site, respectively (Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 33 miles north of the site.

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Los Angeles Basin at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987 M_w 5.9 Whittier Narrows earthquake and the January 17, 1994 M_w 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. These thrust faults and others in the Los Angeles area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 4, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
San Jacinto-Hemet area	April 21, 1918	6.8	73	ESE
Near Redlands	July 23, 1923	6.3	55	E
Long Beach	March 10, 1933	6.4	34	SSE
Tehachapi	July 21, 1952	7.5	79	NW
San Fernando	February 9, 1971	6.6	26	NNW
Whittier Narrows	October 1, 1987	5.9	7	E
Sierra Madre	June 28, 1991	5.8	18	NE
Landers	June 28, 1992	7.3	102	E
Big Bear	June 28, 1992	6.4	80	E
Northridge	January 17, 1994	6.7	21	WNW
Hector Mine	October 16, 1999	7.1	117	ENE

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in Southern California and the effects of ground shaking can be mitigated if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

6.3 Seismic Design Criteria

The following table summarizes summarizes site-specific design criteria obtained from the 2016 California Building Code (CBC; Based on the 2015 International Building Code [IBC] and ASCE 7-10), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the computer program *U.S. Seismic Design Maps*, provided by the USGS. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2016 CBC and Table 20.3-1 of ASCE 7-10. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

2016 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2016 CBC Reference
Site Class	D	Section 1613.3.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	2.556g	Figure 1613.3.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.891g	Figure 1613.3.1(2)
Site Coefficient, F_A	1.0	Table 1613.3.3(1)
Site Coefficient, F_V	1.5	Table 1613.3.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.556g	Section 1613.3.3 (Eqn 16-37)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.336g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.704g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.891g	Section 1613.3.4 (Eqn 16-40)

The table below presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-10.

ASCE 7-10 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.972g	Figure 22-7
Site Coefficient, F_{PGA}	1.0	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.972g	Section 11.8.3 (Eqn 11.8-1)

The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2016 California Building Code and ASCE 7-10, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain "Life Safety" during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2008 Conterminous U.S. Dynamic edition. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.63 magnitude event occurring at a hypocentral distance of 5.71 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.64 magnitude occurring at a hypocentral distance of 9.48 kilometers from the site.

Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

6.4 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

The current standard of practice, as outlined in the "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California" and "Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California" requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

The State of California Seismic Hazard Zone Map for the Los Angeles Quadrangle (CDMG, 1999) indicates that the site is located in an area identified as having a potential for liquefaction. Also, according to the Los Angeles County Safety Element (1990), the site is located within an area identified as having a potential for liquefaction.

Liquefaction analysis of the soils underlying the site was performed using an updated version of the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance and field performance data.

Screening criteria developed by Bray and Sancio (2006) characterize fine-grained soils which are not susceptible to liquefaction as soils with a plasticity index (PI) that is greater than 18 or with a saturated moisture content that is less than 80 percent of the liquid limit. In order to apply the screening criteria, laboratory testing was performed to evaluate the Atterberg Limits and saturated moisture content of select soil samples. Laboratory test results used for the screening criteria are presented as Figure B5.

The liquefaction analysis was performed for a Design Earthquake level by using a historic high groundwater table of 17 feet below the ground surface, a magnitude 6.64 earthquake, and a peak horizontal acceleration of 0.648g ($\frac{2}{3}PGA_M$). The enclosed liquefaction analysis, included herein for boring B1, indicates that the alluvial soils below the historic high groundwater level would not be susceptible to liquefaction settlement during Design Earthquake ground motion (see enclosed calculation sheets, Figures 5 and 6).

It is our understanding that the intent of the Building Code is to maintain "Life Safety" during Maximum Considered Earthquake level events. Therefore, additional analysis was performed to evaluate the potential for liquefaction during a MCE event. The structural engineer should evaluate the proposed structure for the anticipated MCE liquefaction induced settlements and verify that anticipated deformations would not cause the foundation system to lose the ability to support the gravity loads and/or cause collapse of the structure.

The liquefaction analysis was also performed for the Maximum Considered Earthquake level by using a historic high groundwater table of 17 feet below the ground surface, a magnitude 6.63 earthquake, and a peak horizontal acceleration of 0.972g (PGA_M). The enclosed liquefaction analysis, included herein for boring B1, indicates that the alluvial soils below the historic high groundwater level would not be susceptible to liquefaction settlement during Maximum Considered Earthquake ground motion (see enclosed calculation sheets, Figures 7 and 8).

6.5 Slope Stability

The topography at the site is relatively level and the topography in the immediate site vicinity slopes gently to the west or southwest. The site is located within a City of Los Angeles Hillside Grading Area and a Hillside Ordinance Area (City of Los Angeles, 2018). However, the site and adjacent sites are not located within an area identified as having a potential for seismic slope instability (CDMG, 1999; City of Los Angeles, 2018). As previously indicated, an approximately 40-foot-high, 2:1 (H:V gradient) graded slope ascends from the southeast corner of the site. Based on the 2:1 gradient of the slope, it is not considered a stability hazard and is not anticipated to adversely impact the site. There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.6 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. The Los Angeles County Safety Element (Leighton, 1990) indicates that the site is located within the Ascot Reservoir inundation area. However, this reservoir, as well as others in California, are continually monitored by various governmental agencies (such as the State of California Division of Safety of Dams and the U.S. Army Corps of Engineers) to guard against the threat of dam failure. Current design, construction practices, and ongoing programs of review, modification, or total reconstruction of existing dams are intended to ensure that all dams are capable of withstanding the maximum considered earthquake (MCE) for the site. Therefore, the potential for inundation at the site as a result of an earthquake-induced dam failure is considered low.

6.7 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area. Therefore, tsunamis are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Therefore, flooding resulting from a seismically-induced seiche is considered unlikely.

The site is within an area of minimal flooding (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2018; LACDPW, 2018b). Therefore, flooding is not anticipated to adversely impact the site.

6.8 Oil Fields & Methane Potential

Based on a review of the California Division of Oil, Gas and Geothermal Resources (DOGGR) Well Finder Website, the site is not located within the limits of an oilfield and oil or gas wells are not located within 1,200 feet of the site. However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of the DOGGR.

The site is located within the boundaries of a city-designated Methane Zone (City of Los Angeles, 2018). Therefore, there is a potential for methane or other volatile gases to occur at the site. The City of Los Angeles will require a methane study be performed for the proposed development. It is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.9 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is not located within an area of known ground subsidence. No large-scale extraction of groundwater, gas, oil, or geothermal energy is occurring or planned at the site or in the general site vicinity. There appears to be little or no potential for ground subsidence due to withdrawal of fluids or gases at the site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction.

7.1.2 The enclosed seismically-induced settlement analyses indicates that the alluvial soils underlying the site would not be susceptible to liquefaction as a result of the Design Earthquake peak ground acceleration ($\frac{2}{3}PGA_M$).

7.1.3 Up to 1½ feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the Grading section of this report are followed (see Section 7.4).

7.1.4 Based on these considerations, it is recommended that the upper 5 feet of existing earth materials within the building footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as needed to remove any encountered fill or soft soils as necessary at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. Proposed building foundations should be underlain by a minimum of 3 feet of newly placed engineered fill. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 7.4).

7.1.5 Subsequent to the recommended grading, the proposed structures may be supported on a conventional shallow spread foundation system deriving support in newly placed engineered fill.

7.1.6 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the upper 12 inches of the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.1.7 It is anticipated that stable excavations for the recommended grading associated with the proposed structure can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.15).

7.1.8 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils found at or below a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

7.1.9 Where new paving is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft alluvial soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of subgrade soil should be scarified and properly compacted for paving support. Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.12).

7.1.10 Due to the shallow existing groundwater elevation at the site, and minimum City of Los Angeles setback requirements for infiltration systems from existing groundwater elevations and foundations, a stormwater infiltration system is not recommended for this development. It is suggested that stormwater be retained, filtered and discharged in accordance with the requirements of the local governing agency.

7.1.11 Once the design and foundation loading configuration for the proposed structure proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be re-evaluated by this office.

7.1.12 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular soils are encountered.

7.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of existing adjacent improvements.

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.15).

7.2.4 The upper 5 feet of existing site soils encountered during this investigation are considered to have a "moderate" to "high" expansive potential (EI = 81 & 101); and are classified as "expansive" based on the 2016 California Building Code (CBC) Section 1803.5.3. Recommendations presented herein assume that the building foundations and slabs will derive support in these materials.

7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "severely corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B8) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that ABS pipes be considered in lieu of cast-iron for subdrains and retaining wall drains.

7.3.2 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B8) and indicate that the on-site materials possess "severe" sulfate exposure to concrete structures as defined by 2016 CBC Section 1904 and ACI 318-11 Sections 4.2 and 4.3. *The table below presents a summary of concrete requirements set forth by 2016 CBC Section 1904 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.*

REQUIREMENTS FOR CONCRETE EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate Exposure	Exposure Class	Water-Soluble Sulfate Percent by Weight	Cement Type (ASTM C150)	Maximum Water to Cement Ratio by Weight	Minimum Compressive Strength (psi)
Not Applicable	S0	SO_4<0.10	--	--	2,500
Moderate	S1	0.10≤SO_4<0.20	II	0.50	4,000
Severe	**S2**	**0.20≤SO_4≤2.00**	**V**	**0.45**	**4,500**
Very Severe	S3	SO_4>2.00	V+Pozzolan or Slag	0.45	4,500

7.3.3 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

7.4 Grading

7.4.1 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and alluvial soil encountered during exploration is suitable for re-use as engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris are removed.

7.4.2 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer, geotechnical engineer, and building official in attendance. Special soil handling requirements can be discussed at that time.

7.4.3 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.) and the City of Los Angeles Inspector.

7.4.4 As a minimum, it is recommended that the upper five feet of existing earth materials within the proposed building footprint area be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as necessary to remove deeper artificial fill or soft alluvial soil at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint area, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. Proposed building foundations should be underlain by a minimum of three feet of newly placed engineered fill. The limits of existing fill and/or soft alluvial soils removal will be verified by the Geocon representative during site grading activities.

7.4.5 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Prior to placing any fill, the upper 12 inches of the excavation bottom must be scarified, moistened, and proof-rolled with heavy equipment in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

7.4.6 The City of Los Angeles Department of Building and Safety requires a minimum compactive effort of 95 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition) where the soils to be utilized in the fill have less than 15 percent finer than 0.005 millimeter. Soils with more than 15 percent finer than 0.005 millimeter may be compacted to 90 percent of the laboratory maximum dry density in accordance with ASTM D 1557 (latest edition). Fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned at least two percent over optimum moisture content and properly compacted in accordance with ASTM D 1557 (latest edition).

7.4.7. Where new paving is to be placed, it is recommended that all existing fill and soft alluvium be excavated and properly compacted for paving support. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to two percent above optimum moisture content, and compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.12).

7.4.8 It is anticipated that stable excavations for the recommended grading can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of the existing offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.15).

7.4.9 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed or is undesirable, foundations may derive support directly in the undisturbed alluvial soils found at or below a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.

7.4.10 Utility trenches should be properly backfilled in accordance with the requirements of the Green Book (latest edition). The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry as backfill is also acceptable (see Section 7.5). Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

7.4.11 All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. If necessary, import soils used as structural fill should have an expansion index less than 50 and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B8). Import soils placed in the building area should be placed uniformly across the building pad or in a manner that is approved by the Geotechnical Engineer (a representative of Geocon).

7.4.12 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding materials, fill, steel, gravel, or concrete.

7.5 Controlled Low Strength Material (CLSM)

7.5.1 Controlled Low Strength Material (CLSM) may be utilized in lieu of compacted soil as engineered fill where approved in writing by the Geotechnical Engineer. Where utilized within the City of Los Angeles use of CLSM is subject to the following requirements:

Standard Requirements

1. CLSM shall be ready-mixed by a City of Los Angeles approved batch plant;
2. CLSM shall not be placed on uncertified fill, on incompetent natural soil, nor below water;
3. CLSM shall not be placed on a sloping surface with a gradient steeper than 5:1 (horizontal to vertical);
4. Placement of the CLSM shall be under the continuous inspection of a concrete deputy inspector;
5. The excavation bottom shall be accepted by the soil engineer and the City Inspector prior to placing CLSM.

Requirements for CLSM that will be used for support of footings

1. The cement content of the CLSM shall not be less than 188 pounds per cubic yard (min. 2 sacks);
2. The excavation bottom must be level, cleaned of loose soils and approved in writing by Geocon prior to placement of the CLSM;
3. The ultimate compressive strength of the CLSM shall be no less than 100 pounds per square inch (psi) when tested on the 28th-day per ASTM D4832 (latest edition), Standard Test Method for Preparation and Testing of Controlled Low Strength Material Test Cylinders. Compression testing will be performed in accordance with ASTM C39 and City of Los Angeles requirements;
4. Samples of the CLSM will be collected during placement, a minimum of one test (two cylinders) for each 50 cubic yards or fraction thereof;
5. Overexcavation for CLSM placement shall extend laterally beyond the footprint of any proposed footings as required for placement of compacted fill, unless justified otherwise by the soil engineer that footings will have adequate vertical and horizontal bearing capacity.

7.6 Shrinkage

7.6.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 10 and 15 percent should be anticipated when excavating and compacting the upper five feet of existing earth materials on the site to an average relative compaction of 95 percent.

7.4.2 If import soils will be utilized in the building pad, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.). Soils can be borrowed from non-building pad areas and later replaced with imported soils.

7.7 Foundation Design

7.7.1 Subsequent to the recommended grading, a conventional shallow spread foundation system may be utilized for support of the proposed structure provided foundations derive support in newly placed engineered fill. Foundations should be underlain by a minimum of three feet of newly placed engineered fill.

7.7.2 Continuous footings may be designed for an allowable bearing capacity of 2,200 pounds per square foot (psf), and should be a minimum of 12 inches in width, 30 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.7.3 Isolated spread foundations may be designed for an allowable bearing capacity of 2,600 psf, and should be a minimum of 24 inches in width, 30 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

7.7.4 The allowable soil bearing pressure above may be increased by 200 psf and 400 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 4,200 psf.

7.7.5 The allowable bearing pressures may be increased by one-third for transient loads due to wind or seismic forces.

7.7.6 If depth increases are utilized for the perimeter foundations, this office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary. Additional grading should be conducted as-needed in order to maintain the required 3-foot thick blanket of engineered fill below proposed foundations.

7.7.7 Continuous footings should be reinforced with four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. Reinforcement for spread footings should be designed by the project structural engineer.

7.7.8 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only, and are not intended to be used in lieu of those required for structural purposes.

7.7.9 Due to the expansive potential of the subgrade soils, the moisture content in the slab and foundation subgrade should be maintained at 2 percent above optimum moisture content prior to and at the time of concrete placement.

7.7.10 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

7.7.11 This office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

7.8 Foundation Settlement

7.8.1 The maximum expected static settlement for a structure supported on a conventional foundation system deriving support in the recommended bearing materials and designed with a maximum bearing pressure of 4,000 psf is estimated to be less than 1 inch and occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ¾ inch over a distance of 20 feet.

7.8.2 Once the design and foundation loading configurations for the proposed structures proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

7.9 Miscellaneous Foundations

7.9.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures which will not be tied to the proposed structure may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, such as adjacent to property lines, foundations may derive support in the undisturbed alluvial soils found at or below a depth of 24 inches, and should be deepened as necessary to maintain a minimum 12 inch embedment into the recommended bearing materials.

7.9.2 If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 30 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

7.9.3 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated.

7.10 Lateral Design

7.10.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces in the competent alluvial soils or in properly compacted engineered fill.

7.10.2 Passive earth pressure for the sides of foundations and slabs poured against properly compacted engineered fill or competent alluvial soils may be computed as an equivalent fluid having a density of 220 pcf with a maximum earth pressure of 2,200 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

7.11 Concrete Slabs-on-Grade

7.11.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Pavement Recommendations* section of this report (Section 7.12).

7.11.2 Subsequent to the recommended grading, concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 4 inches thick and minimum slab reinforcement should consist of No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.

7.11.3 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the Los Angeles Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Los Angeles Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

7.11.4 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

7.11.5 Exterior slabs for walkways or flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to two percent above optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 8 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

7.11.6 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

7.12 Preliminary Pavement Recommendations

7.12.1 Where new paving is to be placed, it is recommended that all existing fill and soft or unsuitable alluvial materials be excavated and properly recompacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing unsuitable material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to 2 percent above optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

7.12.2 The following pavement sections are based on an assumed R-Value of 20. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.

7.12.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking And Driveways	4.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	12.0

7.12.4 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction"* (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation"* (Caltrans). The use of Crushed Miscellaneous Base in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction"* (Green Book).

7.12.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 95 percent relative compactions, respectively, determined by ASTM Test Method D 1557 (latest edition).

7.12.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

7.13 Retaining Walls Design

7.13.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 5 feet. In the event that walls significantly higher than 5 feet are planned, Geocon should be contacted for additional recommendations.

7.13.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Foundation Design* sections of this report (see Section 7.7).

7.13.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure) of 30 pcf.

7.13.4 Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure) of 50 pcf.

7.13.5 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvial soils or engineered fill derived from onsite soils. If import soil will be used to backfill proposed retaining walls, revised earth pressures may be required to account for the geotechnical properties of the import soil used as engineered fill. This should be evaluated once the use of import soil is established. All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site.

7.13.6 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If retaining wall drainage is not implemented, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf. The value includes hydrostatic pressures plus buoyant lateral earth pressures.

7.13.7 Additional active pressure should be added for a surcharge condition due to sloping ground, vehicular traffic or adjacent structures and should be designed for each condition as the project progresses. Once the design becomes more finalized, an addendum letter can be prepared revising recommendations and addressing specific surcharge conditions throughout the project, if necessary.

7.14 Retaining Wall Drainage

7.14.1 Retaining walls should be provided with a drainage system extended at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 9). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

7.14.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 10). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

7.14.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

7.14.4 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

7.15 Temporary Excavations

7.15.1 Excavations on the order of 5 feet in height may be required during grading operations. The excavations are expected to expose artificial fill and alluvial soils, which are suitable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present, and where not surcharged by adjacent traffic or structures.

7.15.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to maximum height of 8 feet. A uniform slope does not have a vertical portion.

7.15.3 If excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such as slot-cutting or shoring may be necessary in order to maintain lateral support of offsite improvements. Recommendations for alterative temporary excavation measures can be provided under separate cover, if needed.

7.15.4 Where sloped embankments are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. Geocon personnel should inspect the soils exposed in the cut slopes during excavation so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

7.16 Surface Drainage

7.16.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

7.16.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2016 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.16.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond.

7.16.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or an impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

7.17 Plan Review

7.17.1 Grading, shoring and foundation plans should be reviewed by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon West, Inc.

2. This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

LIST OF REFERENCES

California Department of Water Resources, 2018, *Groundwater Level Data by Township, Range, and Section,* http://www.water.ca.gov/waterdatalibrary/groundwater/ hydrographs/index_trs.cfm.

California Department of Water Resources, 1961, *Planned Utilization of Groundwater Basins of the Coastal Plain of Los Angeles County*, Bulletin 104, Appendix A.

California Division of Mines and Geology, 1999; *State of California Seismic Hazard Zones, Los Angeles Quadrangle,* Official Map, Released: March 25, 1999.

California Division of Mines and Geology, 1998, *Seismic Hazard Evaluation of the Los Angeles 7.5-Minute Quadrangle, Los Angeles County, California,* Open File Report 98-20.

California Division of Oil, Gas and Geothermal Resources, 2018, Division of Oil, Gas, and Geothermal Resources Well Finder, http://maps.conservation.ca.gov.doggr/index.html#close. Accessed March 12, 2018.

California Geological Survey, 2018a, *Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California,* Special Publication 42, Revised 2018.

California Geological Survey, 2018b, CGS Information Warehouse, Regulatory Map Portal, http://maps.conservation.ca.gov/cgs/informationwarehouse/index.html?map=regulatorymaps.

California Geological Survey, 2017, *Earthquake Zones of Required Investigation, Los Angeles Quadrangle, Los Angeles County, California*, *dated June 15, 2017*.

California Geological Survey, 2012, *Geologic Compilation of Quaternary Surficial Deposits in Southern California, Los Angeles 30' X 60' Quadrangle*, A Project for the Department of Water Resources by the California Geological Survey, Compiled from existing sources by Trinda L. Bedrossian, CEG and Peter D. Roffers, CGS Special Report 217, Plate 9, Scale 1:100,000.

Dibblee, T. W., Jr., 1989, *Geologic Map of the Los Angeles Quadrangle, Los Angeles County, California,* Dibblee Foundation Map # DF-22.

FEMA, 2018, Online Flood Hazard Maps, *Flood Insurance Rate Map, Los Angeles County, California and Unincorporated Areas, Map Number 06037C1636F, Date Accessed: March 12, 2018,* http://www.esri.com/hazards/index.html.

Jennings, C. W. and Bryant, W. A., 2010, *Fault Activity Map of California,* California Geological Survey Geologic Data Map No. 6.

Lamar, D.L., 1970, *Geology of the Elysian Park-Repetto Hills Area, Los Angeles County, California,* California Division of Mines and Geology Special Report 101.

Leighton and Associates, Inc., 1990, *Technical Appendix to the Safety Element of the Los Angeles County General Plan, Hazard Reduction in Los Angeles County*.

Los Angeles, City of, 2018, NavigateLA website, http://navigatela.lacity.org.

LIST OF REFERENCES (Continued)

Los Angeles, City of, 1996, *Safety Element of the Los Angeles City General Plan*.

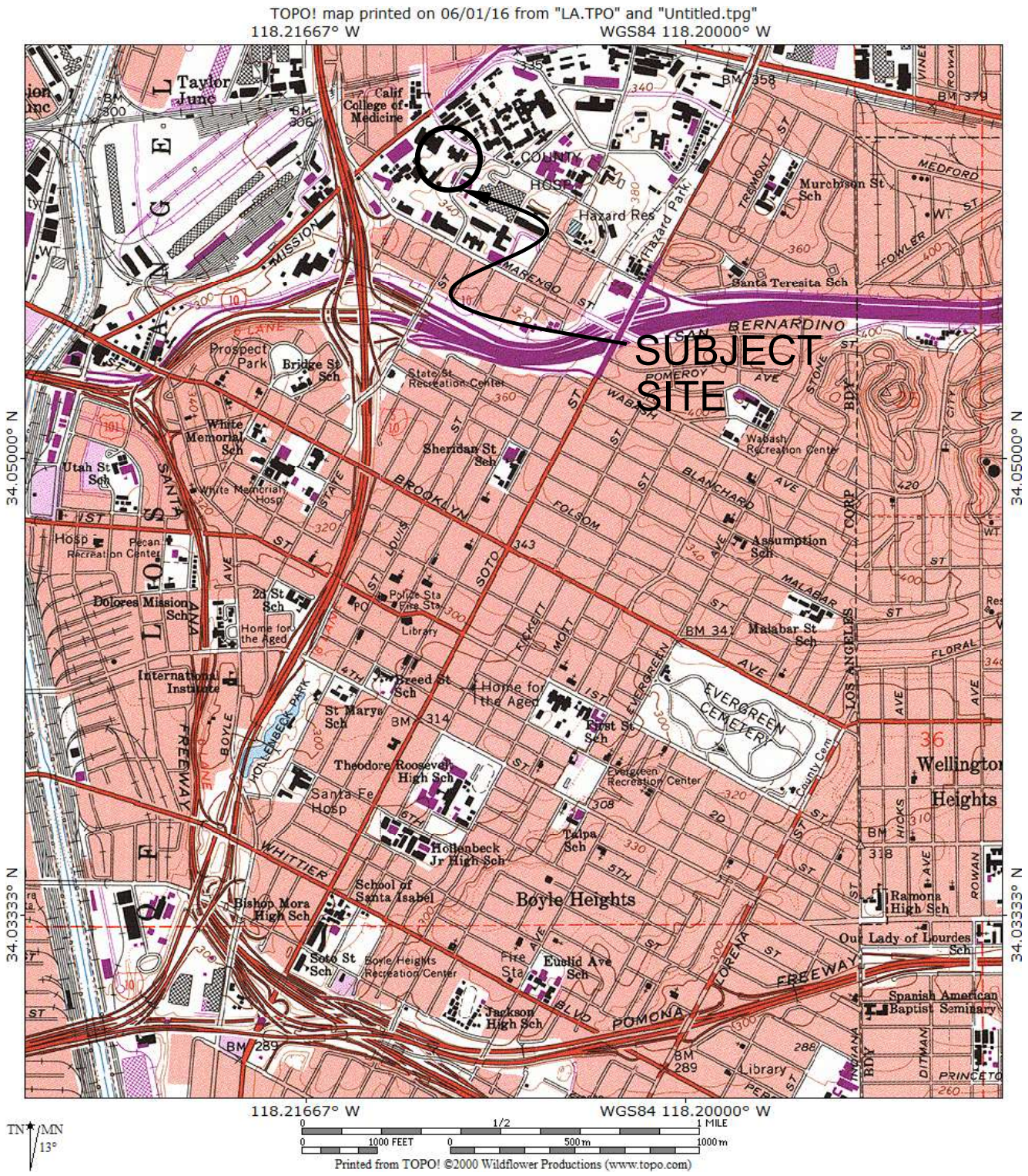
Los Angeles County Department of Public Works, 2018a, Ground Water Wells Website, http://dpw2.co.la.ca.us/website/wells/viewer.asp.

Los Angeles County Department of Public Works, 2018b, Flood Zone Determination Website, http://dpw.lacounty.gov/apps/wmd/floodzone/map.htm.

Toppozada, T., Branum, D., Petersen, M, Hallstrom, C., and Reichle, M., 2000, *Epicenters and Areas Damaged by M> 5 California Earthquakes, 1800 – 1999,* California Geological Survey, Map Sheet 49.

U.S. Geological Survey Seismic Design Maps Web Application, 2016, http://earthquake.usgs.gov/designmaps/us/application.php.

Ziony, J. I., and Jones, L. M., 1989, *Map Showing Late Quaternary Faults and 1978–1984 Seismicity of the Los Angeles Region, California*, U.S. Geological Survey Miscellaneous Field Studies Map MF-1964.



REFERENCE:U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, LOS ANGELES, CA QUADRANGLE

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

VICINITY MAP

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

DRAFTED BY: RA	CHECKED BY: SFK

APRIL 2018	PROJECT NO. A9733-06-01	FIG. 1



0 40' 80'

LEGEND

B5 Approximate Location of Boring

GEOCON WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

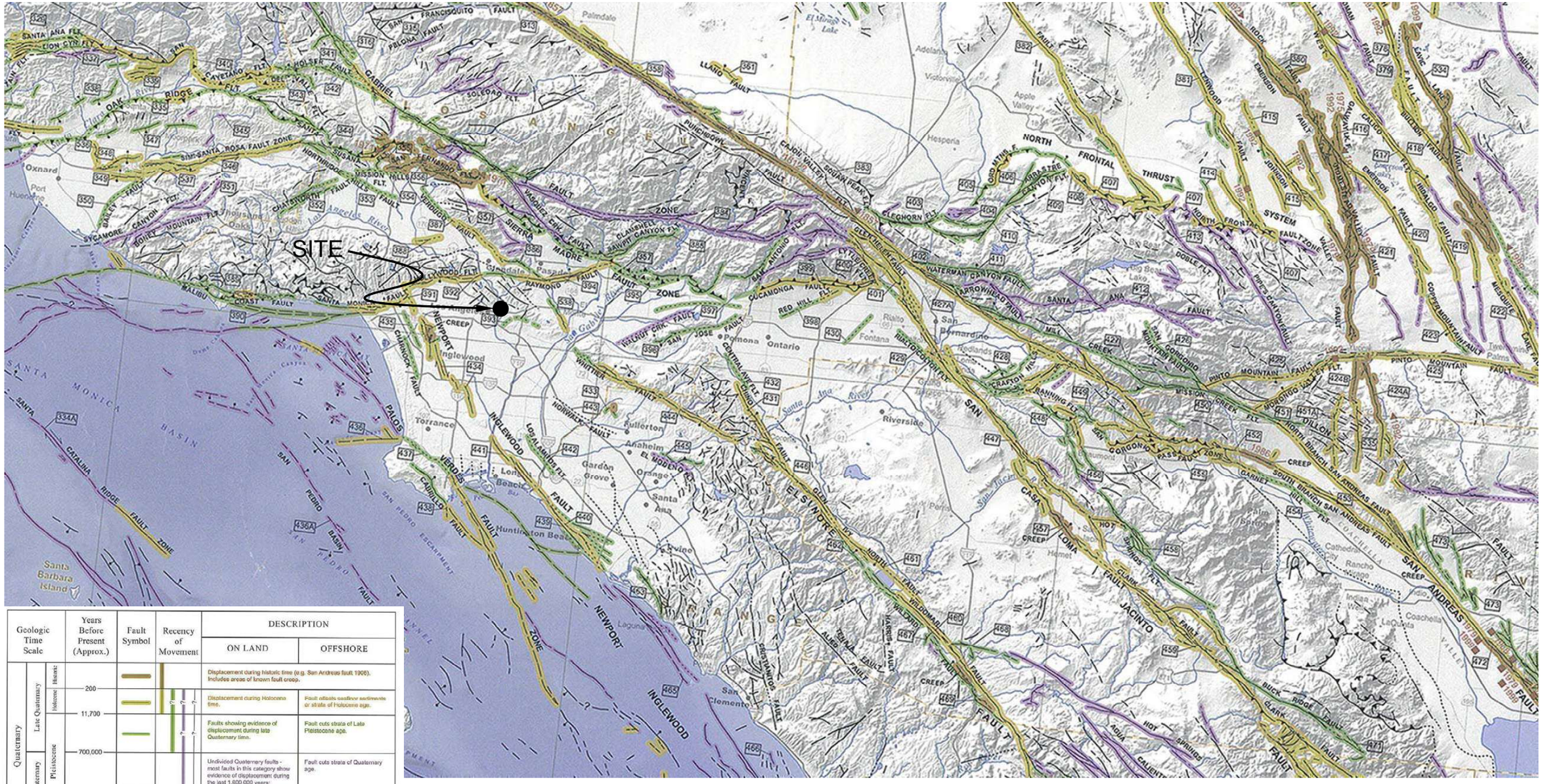
DRAFTED BY: PZ	CHECKED BY: HHD

SITE PLAN

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018	PROJECT NO. A9733-06-01	FIG. 2

Reference: Jennings, C.W. and Bryant, W. A., 2010, Fault Activity Map of California, California Geological Survey Geologic Data Map No. 6.



Geologic Time Scale		Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
					ON LAND	OFFSHORE
Quaternary	Late Quaternary – Historic	200			Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
	Late Quaternary – Holocene	11,700		? ? ?	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
	Pleistocene			? ?	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
	Early Quaternary – Pleistocene	700,000		?	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary		1,600,000*			Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.
		4.5 billion (Age of Earth)				

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.8 Ma criterion.

N

0 12 24 Miles

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RA | CHECKED BY: SFK

REGIONAL FAULT MAP

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. 3

SITE

SANTA BARBARA

VENTURA

LOS ANGELES

ORANGE

RIVERSIDE

IMPERIAL

SAN DIEGO

120°

33°

119°

EPICENTER MAP LEGEND

Period	1800 - 1868	1869 - 1931	1932 - 1999
≥ 7.0			
6.5 - 6.9			
6.0 - 6.4			
5.5 - 5.9			
5.0 - 5.4			

Magnitude (M)

Historical Faulting

Holocene Faulting

Highways (Major)

Highways (Minor)

Lakes

65 Last two digits of M ≥ 6.5 earthquake year

N

0 20 40 Miles

Reference: Toppozada, T., Branum, D., Petersen, M., Hallstrom, C., Cramer, C., and Reichle, M., 2000, Epicenters and Areas Damaged by M≥5 California Earthquakes, 1800 - 1999, California Geological Survey, Map Sheet 49.

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RA | CHECKED BY: SFK

REGIONAL SEISMICITY MAP

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. 4

GEOCON

Client : 1774 Zonal Ave
File No. : A9733-06-01
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
DESIGN EARTHQUAKE

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.64
Peak Horiz. Acceleration PGA_M (g):	0.972
2/3 PGA_M (g):	0.648
Calculated Mag.Wtg.Factor:	0.736
Historic High Groundwater:	17.0
Groundwater Depth During Exploration:	17.0

By Thomas F. Blake (1994-1996)
ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.15
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf): 62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	132.4	0	15.0	5.0	1		84	2.000	38.8	132.4	Infin.	0.998	0.309	--
2.0	132.4	0	15.0	5.0	1	38	84	2.000	45.8	132.4	Infin.	0.993	0.308	--
3.0	132.4	0	15.0	5.0	1	38	84	2.000	45.8	132.4	Infin.	0.989	0.306	--
4.0	132.4	0	15.0	5.0	1	38	84	2.000	45.8	132.4	Infin.	0.984	0.305	--
5.0	132.4	0	15.0	5.0	1	38	84	1.872	43.3	132.4	Infin.	0.979	0.304	--
6.0	132.4	0	15.0	5.0	1	38	84	1.694	39.9	132.4	Infin.	0.975	0.302	--
7.0	132.4	0	15.0	5.0	1	38	84	1.558	37.2	132.4	Infin.	0.970	0.301	--
8.0	140.0	0	35.0	7.5	1		119	1.448	65.5	140.0	Infin.	0.966	0.299	--
9.0	140.0	0	35.0	7.5	1		119	1.355	61.4	140.0	Infin.	0.961	0.298	--
10.0	140.0	0	41.0	10.0	1		123	1.279	67.8	140.0	Infin.	0.957	0.297	--
11.0	140.0	0	41.0	10.0	1		123	1.214	64.4	140.0	Infin.	0.952	0.295	--
12.0	140.0	0	41.0	10.0	1		123	1.158	61.4	140.0	Infin.	0.947	0.294	--
13.0	129.6	0	6.0	15.0	0			1.111	9.3	129.6	~	0.943	0.292	~
14.0	129.6	0	6.0	15.0	0			1.071	8.9	129.6	~	0.938	0.291	~
15.0	129.6	0	6.0	15.0	0			1.035	8.6	129.6	~	0.934	0.289	~
16.0	129.6	0	6.0	15.0	0			1.002	8.4	129.6	~	0.929	0.288	~
17.0	129.6	1	6.0	15.0	0			0.979	8.2	67.2	~	0.925	0.291	~
18.0	128.3	1	11.0	20.0	0			0.964	16.4	65.9	~	0.920	0.297	~
19.0	128.3	1	11.0	20.0	0			0.951	16.1	65.9	~	0.915	0.303	~
20.0	128.3	1	11.0	20.0	0			0.937	15.9	65.9	~	0.911	0.308	~
21.5	128.3	1	11.0	20.0	0			0.921	15.6	65.9	~	0.905	0.314	~
22.0	122.2	1	14.0	25.0	0			0.916	21.1	59.8	~	0.901	0.316	~
23.0	122.2	1	14.0	25.0	0			0.902	20.8	59.8	~	0.897	0.322	~
24.0	122.2	1	14.0	25.0	0			0.892	20.6	59.8	~	0.893	0.326	~
25.0	122.2	1	14.0	25.0	0			0.882	20.3	59.8	~	0.888	0.330	~
26.0	122.2	1	14.0	25.0	0			0.872	20.1	59.8	~	0.883	0.333	~
27.0	125.3	1	14.0	25.0	0			0.863	19.9	62.9	~	0.879	0.336	~
28.0	125.3	1	40.0	30.0	1		98	0.853	58.9	62.9	Infin.	0.874	0.339	Non-Liq.
29.0	125.3	1	40.0	30.0	1		98	0.844	58.2	62.9	Infin.	0.870	0.341	Non-Liq.
30.0	125.3	1	40.0	30.0	1		98	0.835	57.6	62.9	Infin.	0.865	0.344	Non-Liq.
31.0	125.3	1	40.0	30.0	1		98	0.826	57.0	62.9	Infin.	0.861	0.346	Non-Liq.
32.0	129.5	1	40.0	30.0	1		98	0.818	56.4	67.1	Infin.	0.856	0.348	Non-Liq.
33.0	129.5	1	40.0	30.0	1		98	0.809	55.8	67.1	Infin.	0.851	0.349	Non-Liq.
34.0	129.5	1	40.0	30.0	1		98	0.801	55.2	67.1	Infin.	0.847	0.351	Non-Liq.
35.0	129.5	1	51.0	35.0	1		108	0.793	69.7	67.1	Infin.	0.842	0.352	Non-Liq.
36.0	129.5	1	51.0	35.0	1		108	0.785	69.0	67.1	Infin.	0.838	0.353	Non-Liq.
37.0	129.5	1	51.0	35.0	1		108	0.777	68.4	67.1	Infin.	0.833	0.354	Non-Liq.
38.0	129.5	1	51.0	35.0	1		108	0.770	67.7	67.1	Infin.	0.829	0.355	Non-Liq.
39.0	129.5	1	51.0	35.0	1		108	0.762	67.1	67.1	Infin.	0.824	0.355	Non-Liq.
40.0	129.5	1	50.0	40.0	1		103	0.755	65.1	67.1	Infin.	0.819	0.356	Non-Liq.
41.0	129.5	1	50.0	40.0	1		103	0.748	64.6	67.1	Infin.	0.815	0.356	Non-Liq.
42.0	129.5	1	50.0	40.0	1		103	0.742	64.0	67.1	Infin.	0.810	0.357	Non-Liq.
43.0	129.5	1	50.0	40.0	1		103	0.735	63.4	67.1	Infin.	0.806	0.357	Non-Liq.
44.0	129.5	1	50.0	40.0	1		103	0.729	62.9	67.1	Infin.	0.801	0.357	Non-Liq.
45.0	129.5	1	54.0	45.0	1		104	0.723	67.3	67.1	Infin.	0.797	0.357	Non-Liq.
46.0	129.5	1	54.0	45.0	1		104	0.717	66.8	67.1	Infin.	0.792	0.357	Non-Liq.
47.0	129.5	1	54.0	45.0	1		104	0.711	66.2	67.1	Infin.	0.787	0.357	Non-Liq.
48.0	129.5	1	54.0	45.0	1		104	0.705	65.7	67.1	Infin.	0.783	0.356	Non-Liq.
49.0	129.5	1	54.0	45.0	1		104	0.700	65.2	67.1	Infin.	0.778	0.356	Non-Liq.
50.0	129.5	1	50.0	50.0	1		98	0.694	59.9	67.1	Infin.	0.774	0.356	Non-Liq.

Figure 5

GEOCON

Client : 1774 Zonal Ave
File No. : A9733-06-01
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS
DESIGN EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.64
PGAM (g):	0.972
2/3 PGAM (g):	0.65
Calculated Mag.Wtg.Factor:	0.736
Historic High Groundwater:	17.0
Groundwater @ Exploration:	17.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	τ_{av}/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain $[e_{15}]$ (%)	EQ. SETTLE. Pe (in.)
1.0	15	132.3608	0.033	0.033	84	39	0.421	--	0.00	0.00
2.0	15	132.3608	0.099	0.099	84	46	0.421	--	0.00	0.00
3.0	15	132.3608	0.165	0.165	84	46	0.421	--	0.00	0.00
4.0	15	132.3608	0.232	0.232	84	46	0.421	--	0.00	0.00
5.0	15	132.3608	0.298	0.298	84	43	0.421	--	0.00	0.00
6.0	15	132.3608	0.364	0.364	84	40	0.421	--	0.00	0.00
7.0	15	132.3608	0.430	0.430	84	37	0.421	--	0.00	0.00
8.0	35	139.968	0.498	0.498	119	66	0.421	--	0.00	0.00
9.0	35	139.968	0.568	0.568	119	61	0.421	--	0.00	0.00
10.0	41	139.968	0.638	0.638	123	68	0.421	--	0.00	0.00
11.0	41	139.968	0.708	0.708	123	64	0.421	--	0.00	0.00
12.0	41	139.968	0.778	0.778	123	61	0.421	--	0.00	0.00
13.0	6	129.6464	0.846	0.846		9	0.421	~	0.00	0.00
14.0	6	129.6464	0.910	0.910		9	0.421	~	0.00	0.00
15.0	6	129.6464	0.975	0.975		9	0.421	~	0.00	0.00
16.0	6	129.6464	1.040	1.040		8	0.421	~	0.00	0.00
17.0	6	129.6464	1.105	1.089		8	0.427	~	0.00	0.00
18.0	11	128.3194	1.169	1.123		16	0.439	~	0.00	0.00
19.0	11	128.3194	1.234	1.156		16	0.450	~	0.00	0.00
20.0	11	128.3194	1.298	1.188		16	0.460	~	0.00	0.00
21.5	11	128.3194	1.378	1.230		16	0.472	~	0.00	0.00
22.0	14	122.1536	1.409	1.245		21	0.477	~	0.00	0.00
23.0	14	122.1536	1.486	1.283		21	0.488	~	0.00	0.00
24.0	14	122.1536	1.547	1.313		21	0.497	~	0.00	0.00
25.0	14	122.1536	1.608	1.343		20	0.505	~	0.00	0.00
26.0	14	122.1536	1.669	1.372		20	0.512	~	0.00	0.00
27.0	14	125.2875	1.731	1.403		20	0.520	~	0.00	0.00
28.0	40	125.2875	1.793	1.434	98	59	0.527	Non-Liq.	0.00	0.00
29.0	40	125.2875	1.856	1.466	98	58	0.534	Non-Liq.	0.00	0.00
30.0	40	125.2875	1.919	1.497	98	58	0.540	Non-Liq.	0.00	0.00
31.0	40	125.2875	1.981	1.529	98	57	0.546	Non-Liq.	0.00	0.00
32.0	40	129.5382	2.045	1.561	98	56	0.552	Non-Liq.	0.00	0.00
33.0	40	129.5382	2.110	1.595	98	56	0.557	Non-Liq.	0.00	0.00
34.0	40	129.5382	2.174	1.628	98	55	0.563	Non-Liq.	0.00	0.00
35.0	51	129.5382	2.239	1.662	108	70	0.568	Non-Liq.	0.00	0.00
36.0	51	129.5382	2.304	1.696	108	69	0.573	Non-Liq.	0.00	0.00
37.0	51	129.5382	2.369	1.729	108	68	0.577	Non-Liq.	0.00	0.00
38.0	51	129.5382	2.434	1.763	108	68	0.582	Non-Liq.	0.00	0.00
39.0	51	129.5382	2.498	1.796	108	67	0.586	Non-Liq.	0.00	0.00
40.0	50	129.5382	2.563	1.830	103	65	0.590	Non-Liq.	0.00	0.00
41.0	50	129.5382	2.628	1.863	103	65	0.594	Non-Liq.	0.00	0.00
42.0	50	129.5382	2.693	1.897	103	64	0.598	Non-Liq.	0.00	0.00
43.0	50	129.5382	2.757	1.931	103	63	0.602	Non-Liq.	0.00	0.00
44.0	50	129.5382	2.822	1.964	103	63	0.605	Non-Liq.	0.00	0.00
45.0	54	129.5382	2.887	1.998	104	67	0.609	Non-Liq.	0.00	0.00
46.0	54	129.5382	2.952	2.031	104	67	0.612	Non-Liq.	0.00	0.00
47.0	54	129.5382	3.016	2.065	104	66	0.616	Non-Liq.	0.00	0.00
48.0	54	129.5382	3.081	2.098	104	66	0.619	Non-Liq.	0.00	0.00
49.0	54	129.5382	3.146	2.132	104	65	0.622	Non-Liq.	0.00	0.00
50.0	50	129.5382	3.211	2.166	98	60	0.625	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.0 INCHES

Figure 6

GEOCON

Client : 1774 Zonal Ave
File No. : A9733-06-01
Boring : 1

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL
MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.63
Peak Horiz. Acceleration PGA_M (g):	0.972
Calculated Mag.Wtg.Factor:	0.733
Historic High Groundwater:	17.0
Groundwater Depth During Exploration:	17.0

By Thomas F. Blake (1994-1996)
ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR)(0-no or 1-yes):	1.0
Bore Dia. Corr. (CB):	1.15
Sampler Corr. (CS):	1.20
Use Ksigma (0 or 1):	1.0

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf): 62.4

Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	FIELD SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)	Est. Dr (%)	CN Factor	Corrected (N1)60	Eff. Unit Wt. (psf)	Resist. CRR	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	132.4	0	15.0	5.0	1	38	84	2.000	45.8	132.4	Infin.	0.998	0.462	--
2.0	132.4	0	15.0	5.0	1	38	84	2.000	45.8	132.4	Infin.	0.993	0.460	--
3.0	132.4	0	15.0	5.0	1	38	84	2.000	45.8	132.4	Infin.	0.989	0.458	--
4.0	132.4	0	15.0	5.0	1	38	84	2.000	45.8	132.4	Infin.	0.984	0.456	--
5.0	132.4	0	15.0	5.0	1	38	84	1.872	43.3	132.4	Infin.	0.979	0.454	--
6.0	132.4	0	15.0	5.0	1	38	84	1.694	39.9	132.4	Infin.	0.975	0.451	--
7.0	132.4	0	15.0	5.0	1	38	84	1.558	37.2	132.4	Infin.	0.970	0.449	--
8.0	140.0	0	35.0	7.5	1		119	1.448	65.5	140.0	Infin.	0.966	0.447	--
9.0	140.0	0	35.0	7.5	1		119	1.355	61.4	140.0	Infin.	0.961	0.445	--
10.0	140.0	0	41.0	10.0	1		123	1.279	67.8	140.0	Infin.	0.957	0.443	--
11.0	140.0	0	41.0	10.0	1		123	1.214	64.4	140.0	Infin.	0.952	0.441	--
12.0	140.0	0	41.0	10.0	1		123	1.158	61.4	140.0	Infin.	0.947	0.439	--
13.0	129.6	0	6.0	15.0	0			1.111	9.3	129.6	~	0.943	0.437	~
14.0	129.6	0	6.0	15.0	0			1.071	8.9	129.6	~	0.938	0.434	~
15.0	129.6	0	6.0	15.0	0			1.035	8.6	129.6	~	0.934	0.432	~
16.0	129.6	0	6.0	15.0	0			1.002	8.4	129.6	~	0.929	0.430	~
17.0	129.6	1	6.0	15.0	0			0.979	8.2	67.2	~	0.925	0.434	~
18.0	128.3	1	11.0	20.0	0			0.964	16.4	65.9	~	0.920	0.444	~
19.0	128.3	1	11.0	20.0	0			0.951	16.1	65.9	~	0.915	0.452	~
20.0	128.3	1	11.0	20.0	0			0.937	15.9	65.9	~	0.911	0.461	~
21.5	128.3	1	11.0	20.0	0			0.921	15.6	65.9	~	0.905	0.470	~
22.0	122.2	1	14.0	25.0	0			0.916	21.1	59.8	~	0.901	0.472	~
23.0	122.2	1	14.0	25.0	0			0.902	20.8	59.8	~	0.897	0.481	~
24.0	122.2	1	14.0	25.0	0			0.892	20.6	59.8	~	0.893	0.487	~
25.0	122.2	1	14.0	25.0	0			0.882	20.3	59.8	~	0.888	0.492	~
26.0	122.2	1	14.0	25.0	0			0.872	20.1	59.8	~	0.883	0.497	~
27.0	125.3	1	14.0	25.0	0			0.863	19.9	62.9	~	0.879	0.502	~
28.0	125.3	1	40.0	30.0	1		98	0.853	58.9	62.9	Infin.	0.874	0.506	Non-Liq.
29.0	125.3	1	40.0	30.0	1		98	0.844	58.2	62.9	Infin.	0.870	0.510	Non-Liq.
30.0	125.3	1	40.0	30.0	1		98	0.835	57.6	62.9	Infin.	0.865	0.513	Non-Liq.
31.0	125.3	1	40.0	30.0	1		98	0.826	57.0	62.9	Infin.	0.861	0.516	Non-Liq.
32.0	129.5	1	40.0	30.0	1		98	0.818	56.4	67.1	Infin.	0.856	0.519	Non-Liq.
33.0	129.5	1	40.0	30.0	1		98	0.809	55.8	67.1	Infin.	0.851	0.521	Non-Liq.
34.0	129.5	1	40.0	30.0	1		98	0.801	55.2	67.1	Infin.	0.847	0.524	Non-Liq.
35.0	129.5	1	51.0	35.0	1		108	0.793	69.7	67.1	Infin.	0.842	0.525	Non-Liq.
36.0	129.5	1	51.0	35.0	1		108	0.785	69.0	67.1	Infin.	0.838	0.527	Non-Liq.
37.0	129.5	1	51.0	35.0	1		108	0.777	68.4	67.1	Infin.	0.833	0.528	Non-Liq.
38.0	129.5	1	51.0	35.0	1		108	0.770	67.7	67.1	Infin.	0.829	0.530	Non-Liq.
39.0	129.5	1	51.0	35.0	1		108	0.762	67.1	67.1	Infin.	0.824	0.531	Non-Liq.
40.0	129.5	1	50.0	40.0	1		103	0.755	65.1	67.1	Infin.	0.819	0.531	Non-Liq.
41.0	129.5	1	50.0	40.0	1		103	0.748	64.6	67.1	Infin.	0.815	0.532	Non-Liq.
42.0	129.5	1	50.0	40.0	1		103	0.742	64.0	67.1	Infin.	0.810	0.533	Non-Liq.
43.0	129.5	1	50.0	40.0	1		103	0.735	63.4	67.1	Infin.	0.806	0.533	Non-Liq.
44.0	129.5	1	50.0	40.0	1		103	0.729	62.9	67.1	Infin.	0.801	0.533	Non-Liq.
45.0	129.5	1	54.0	45.0	1		104	0.723	67.3	67.1	Infin.	0.797	0.533	Non-Liq.
46.0	129.5	1	54.0	45.0	1		104	0.717	66.8	67.1	Infin.	0.792	0.533	Non-Liq.
47.0	129.5	1	54.0	45.0	1		104	0.711	66.2	67.1	Infin.	0.787	0.533	Non-Liq.
48.0	129.5	1	54.0	45.0	1		104	0.705	65.7	67.1	Infin.	0.783	0.532	Non-Liq.
49.0	129.5	1	54.0	45.0	1		104	0.700	65.2	67.1	Infin.	0.778	0.532	Non-Liq.
50.0	129.5	1	50.0	50.0	1		98	0.694	59.9	67.1	Infin.	0.774	0.531	Non-Liq.

Figure 7

GEOCON

Client : 1774 Zonal Ave
File No. : A9733-06-01
Boring : 1

LIQUEFACTION SETTLEMENT ANALYSIS
MAXIMUM CONSIDERED EARTHQUAKE

(SATURATED SAND AT INITIAL LIQUEFACTION CONDITION)

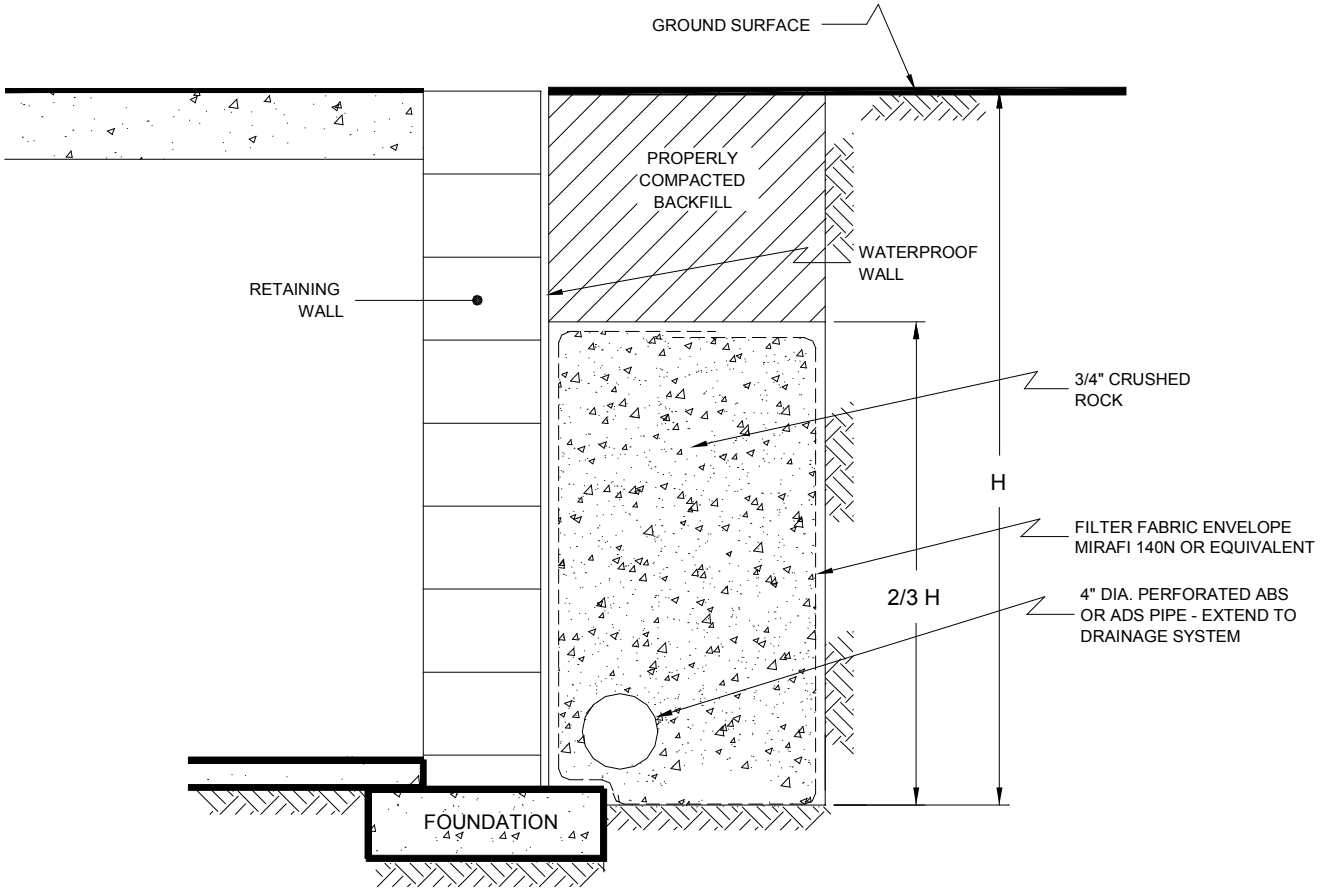
NCEER (1996) METHOD
EARTHQUAKE INFORMATION:

Earthquake Magnitude:	6.63
PGA_M (g):	0.972
Calculated Mag.Wtg.Factor:	0.733
Historic High Groundwater:	17.0
Groundwater @ Exploration:	17.0

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	τ_{av}/σ'_o	LIQUEFACTION SAFETY FACTOR	Volumetric Strain $[e_{15}]$ (%)	EQ. SETTLE. Pe (in.)
1.0	15	132.3608	0.033	0.033	84	46	0.632	--	0.00	0.00
2.0	15	132.3608	0.099	0.099	84	46	0.632	--	0.00	0.00
3.0	15	132.3608	0.165	0.165	84	46	0.632	--	0.00	0.00
4.0	15	132.3608	0.232	0.232	84	46	0.632	--	0.00	0.00
5.0	15	132.3608	0.298	0.298	84	43	0.632	--	0.00	0.00
6.0	15	132.3608	0.364	0.364	84	40	0.632	--	0.00	0.00
7.0	15	132.3608	0.430	0.430	84	37	0.632	--	0.00	0.00
8.0	35	139.968	0.498	0.498	119	66	0.632	--	0.00	0.00
9.0	35	139.968	0.568	0.568	119	61	0.632	--	0.00	0.00
10.0	41	139.968	0.638	0.638	123	68	0.632	--	0.00	0.00
11.0	41	139.968	0.708	0.708	123	64	0.632	--	0.00	0.00
12.0	41	139.968	0.778	0.778	123	61	0.632	--	0.00	0.00
13.0	6	129.6464	0.846	0.846		9	0.632	~	0.00	0.00
14.0	6	129.6464	0.910	0.910		9	0.632	~	0.00	0.00
15.0	6	129.6464	0.975	0.975		9	0.632	~	0.00	0.00
16.0	6	129.6464	1.040	1.040		8	0.632	~	0.00	0.00
17.0	6	129.6464	1.105	1.089		8	0.641	~	0.00	0.00
18.0	11	128.3194	1.169	1.123		16	0.658	~	0.00	0.00
19.0	11	128.3194	1.234	1.156		16	0.674	~	0.00	0.00
20.0	11	128.3194	1.298	1.188		16	0.690	~	0.00	0.00
21.5	11	128.3194	1.378	1.230		16	0.708	~	0.00	0.00
22.0	14	122.1536	1.409	1.245		21	0.715	~	0.00	0.00
23.0	14	122.1536	1.486	1.283		21	0.732	~	0.00	0.00
24.0	14	122.1536	1.547	1.313		21	0.744	~	0.00	0.00
25.0	14	122.1536	1.608	1.343		20	0.757	~	0.00	0.00
26.0	14	122.1536	1.669	1.372		20	0.768	~	0.00	0.00
27.0	14	125.2875	1.731	1.403		20	0.779	~	0.00	0.00
28.0	40	125.2875	1.793	1.434	98	59	0.790	Non-Liq.	0.00	0.00
29.0	40	125.2875	1.856	1.466	98	58	0.800	Non-Liq.	0.00	0.00
30.0	40	125.2875	1.919	1.497	98	58	0.810	Non-Liq.	0.00	0.00
31.0	40	125.2875	1.981	1.529	98	57	0.819	Non-Liq.	0.00	0.00
32.0	40	129.5382	2.045	1.561	98	56	0.827	Non-Liq.	0.00	0.00
33.0	40	129.5382	2.110	1.595	98	56	0.836	Non-Liq.	0.00	0.00
34.0	40	129.5382	2.174	1.628	98	55	0.844	Non-Liq.	0.00	0.00
35.0	51	129.5382	2.239	1.662	108	70	0.851	Non-Liq.	0.00	0.00
36.0	51	129.5382	2.304	1.696	108	69	0.858	Non-Liq.	0.00	0.00
37.0	51	129.5382	2.369	1.729	108	68	0.865	Non-Liq.	0.00	0.00
38.0	51	129.5382	2.434	1.763	108	68	0.872	Non-Liq.	0.00	0.00
39.0	51	129.5382	2.498	1.796	108	67	0.879	Non-Liq.	0.00	0.00
40.0	50	129.5382	2.563	1.830	103	65	0.885	Non-Liq.	0.00	0.00
41.0	50	129.5382	2.628	1.863	103	65	0.891	Non-Liq.	0.00	0.00
42.0	50	129.5382	2.693	1.897	103	64	0.897	Non-Liq.	0.00	0.00
43.0	50	129.5382	2.757	1.931	103	63	0.902	Non-Liq.	0.00	0.00
44.0	50	129.5382	2.822	1.964	103	63	0.908	Non-Liq.	0.00	0.00
45.0	54	129.5382	2.887	1.998	104	67	0.913	Non-Liq.	0.00	0.00
46.0	54	129.5382	2.952	2.031	104	67	0.918	Non-Liq.	0.00	0.00
47.0	54	129.5382	3.016	2.065	104	66	0.923	Non-Liq.	0.00	0.00
48.0	54	129.5382	3.081	2.098	104	66	0.928	Non-Liq.	0.00	0.00
49.0	54	129.5382	3.146	2.132	104	65	0.932	Non-Liq.	0.00	0.00
50.0	50	129.5382	3.211	2.166	98	60	0.937	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 0.0 INCHES

Figure 8



NO SCALE

GEOCON WEST, INC.

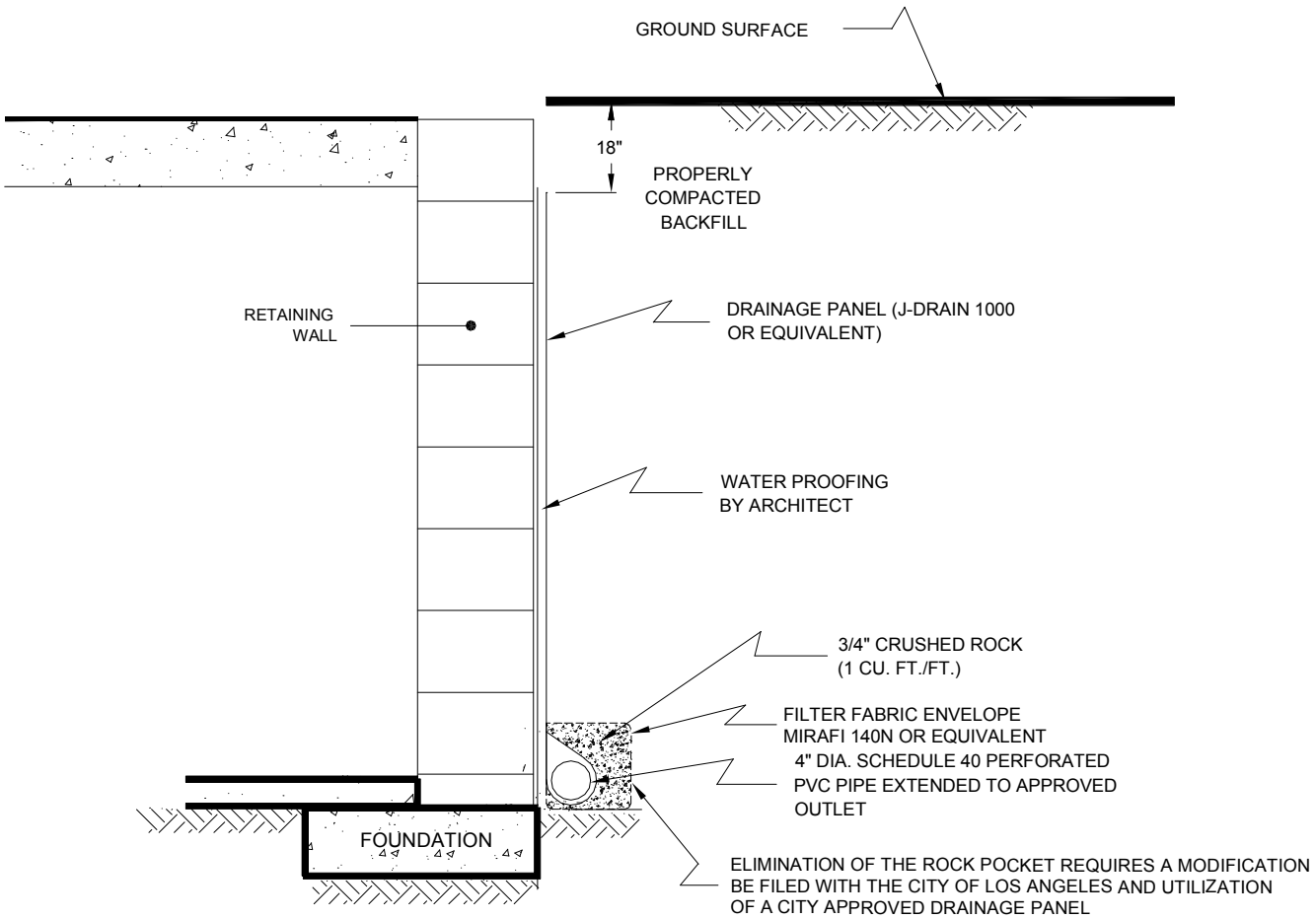
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: PZ	CHECKED BY: HHD

RETAINING WALL DRAIN DETAIL

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018	PROJECT NO. A9733-06-01	FIG. 9



NO SCALE

GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: PZ	CHECKED BY: HHD

RETAINING WALL DRAIN DETAIL

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018	PROJECT NO. A9733-06-01	FIG. 10

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on March 6, 2018, by excavating five 8-inch-diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths of approximately 15½ feet and 56 feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2 3/8-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained. Standard Penetration Tests (SPTs) were performed in boring B1.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A5. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the boring logs were revised based on subsequent laboratory testing. The location of the borings are shown on Figure 2.

PROJECT NO. A9733-06-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**GRAVEL** **ARTIFICIAL FILL** Silty Sand, loose, dry, brown, fine- to medium-grained, some coarse-grained, some gravel (to 1"), some brick and asphalt fragments.			
2	B1@2.5'			SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained, some coarse-grained.	51	115.7	14.4
4	B1@5'				- trace coarse-grained	15	--	--
6								
8	B1@7.5'			ML	Sandy Silt, hard, slightly moist, dark brown, fine- to medium-grained, some coarse-grained.	63	121.5	15.2
10	B1@10'			SM	Silty Sand, dense, slightly moist, dark brown, fine- to medium-grained, some coarse-grained, some gravel (to 0.5").	41	--	--
12	B1@12.5'				Silt, firm, slightly moist, brown, trace fine- to coarse-grained sand.	20	108.4	19.6
14								
16	B1@15'		▼	ML	- soft, wet, fine- to medium-grained, some coarse-grained	6	--	--
18	B1@17.5'				- firm, light brown to yellowish brown	15	103.4	24.1
20	B1@20'				- saturated, yellowish brown	11	--	--
22	B1@22.5'				**PUENTE FORMATION** Interbedded Siltstone and Sandstone, yellowish brown, fine- to medium-grained, thinly bedded, soft, highly weathered, heavy oxidation.	24	94.4	29.4
24								
26	B1@25'					14	--	--
28	B1@27.5'				- moderately hard	43	97.5	28.5

A9733-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 1 of 2

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 1** ELEV. (MSL.) -- DATE COMPLETED **3/6/18** EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH** MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30'					40	--	--
32	B1@32.5'				- hard	50 (5")	103.3	25.4
34								
36	B1@35'					51	--	--
38								
40	B1@40'					50 (6")	--	--
42								
44								
46	B1@45'					54	--	--
48								
50	B1@50'					50 (4")	--	--
52								
54								
56					Total depth of boring: 56 feet Fill to 1.5 feet. Groundwater encountered at 17 feet. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

A9733-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 2 of 2

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

BORING 2

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**GRAVEL** **ARTIFICIAL FILL** Silty Sand, loose, dry, brown, fine- to medium-grained, some coarse-grained, some gravel (to 1"), fragments of brick and asphalt.			
2	B2@2'					65	101.2	15.3
4					**ALLUVIUM** Silty Sand, dense, slightly moist, brown, fine- to medium-grained, some coarse-grained.			
6	B2@5'			SM	- decrease in medium- to coarse-grained	65	103.9	12.9
8	B2@7'				- medium dense, increase in silt content	49	112.7	20.8
10	B2@10'			ML	Sandy Silt, stiff, slightly moist, brown, fine- to medium-grained.	33	109.7	18.8
12	B2@12'				- some coarse-grained, increase in sand content	34	100.9	21.1
14	B2@15'			SM	Silty Sand, dense, slightly moist, light brown, fine- to medium-grained, some coarse-grained.	57	110.1	15.7

Total depth of boring: 15.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9733-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

BORING 3

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			**GRAVEL**			
			ARTIFICIAL FILL			
2	B3@2'		Silty Sand, loose, dry, brown, fine- to medium-grained, some coarse-grained, some gravel (to 1").	20	104.6	19.2
		ML	**ALLUVIUM**			
4			Sandy Silt, firm, slightly moist, dark brown, fine- to coarse-grained.			
	B3@5'		- stiff, increase in sand content	28	101.4	15.8
6			Silty Sand, medium dense, slightly moist, light brown, fine-grained, some medium-grained, trace gravel (to 0.5").			
	B3@7'			28	111.9	16.4
8		SM				
10	B3@10'		- some oxidation staining	39	109.5	20.1
12	B3@12'		Sandy Silt, stiff, slightly moist, olive brown, fine-grained, trace medium- to coarse-grained.	32	94.7	27.6
		ML				
14						
	B3@15'		- decrease in sand content	40	114.9	18.0

Total depth of boring: 15.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9733-06-01 BORING LOGS.GPJ

Figure A3,
Log of Boring 3, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

BORING 4

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT**			
					ARTIFICIAL FILL			
2	B4@2'			ML	Silty Sand, loose, dry, dark brown, fine- to medium-grained, some coarse-grained, some gravel (to 1").	18	104.0	16.2
					ALLUVIUM			
4					Sandy Silt, firm, slightly moist, brown, fine- to coarse-grained.			
					Silty Sand, loose, slightly moist, light yellowish brown, fine-grained.			
6	B4@5'			SM		14	112.4	19.8
	B4@7'				- some bedrock framents	14	100.8	23.7
8								
10					Sandy Silt, firm, slightly moist, dark brown, fine- to medium-grained.			
	B4@10'					17	103.7	24.5
12	B4@12'			ML	- soft, moist, some coarse-grained	8	98.7	26.7
14								
	B4@15'				- firm, brown to yellowish brown, fine- to coarse-grained	13	97.1	26.6

Total depth of boring: 15.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9733-06-01 BORING LOGS.GPJ

Figure A4,
Log of Boring 4, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

BORING 5

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ASPHALT**			
					ARTIFICIAL FILL Silty Sand, loose, dry, brown, fine- to medium-grained, some coarse-grained, some gravel (to 1").			
2	B5@2'					12	99.9	21.0
					ALLUVIUM Sandy Silt, firm, slightly moist, light brown, fine- to coarse-grained.			
4								
	B5@5'				- yellowish brown, fine- to medium-grained	14	100.9	22.1
6				ML				
	B5@7'				- dark brown	18	110.5	16.5
8								
10	B5@10'				- soft, dark yellowish brown, trace coarse-grained	9	86.7	16.8
					Sand with Silt, well-graded, medium dense, slightly moist, brown to yellowish brown, fine- to coarse-grained, some gravel (to 0.5").			
12	B5@12'			SW-SM		27	119.0	10.3
14					**PUENTE FORMATION** Interbedded Siltstone and Sandstone, yellowish brown, fine- to medium-grained, thinly bedded, soft to moderately hard, highly weathered, oxidation.			
	B5@15'					53	100.1	23.7

Total depth of boring: 15.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9733-06-01 BORING LOGS.GPJ

Figure A5,
Log of Boring 5, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

APPENDIX B

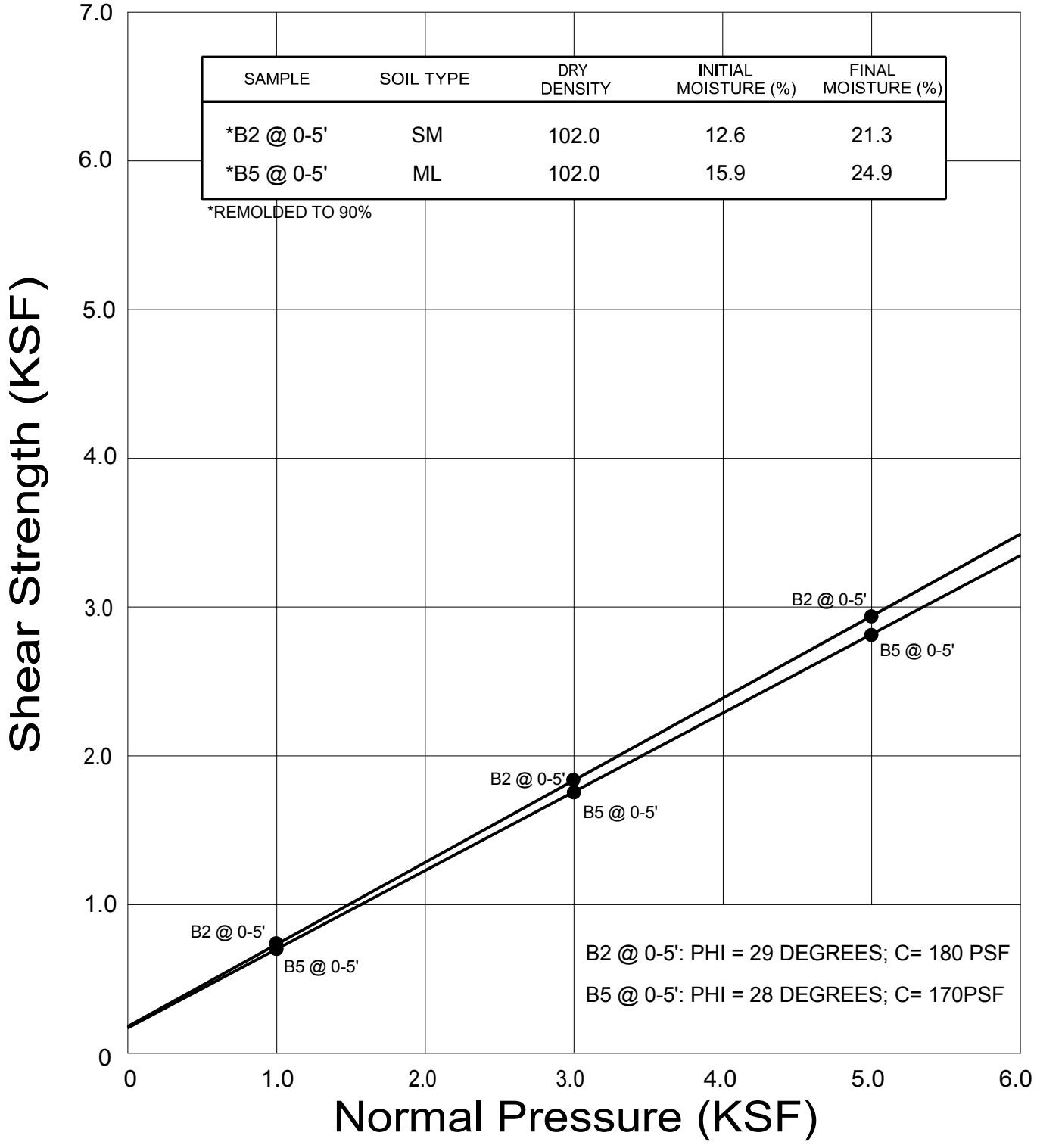
APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, plasticity indices, grain size, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B8. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.

SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
*B2 @ 0-5'	SM	102.0	12.6	21.3
*B5 @ 0-5'	ML	102.0	15.9	24.9

*REMOLDED TO 90%



● Direct Shear, Saturated

GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

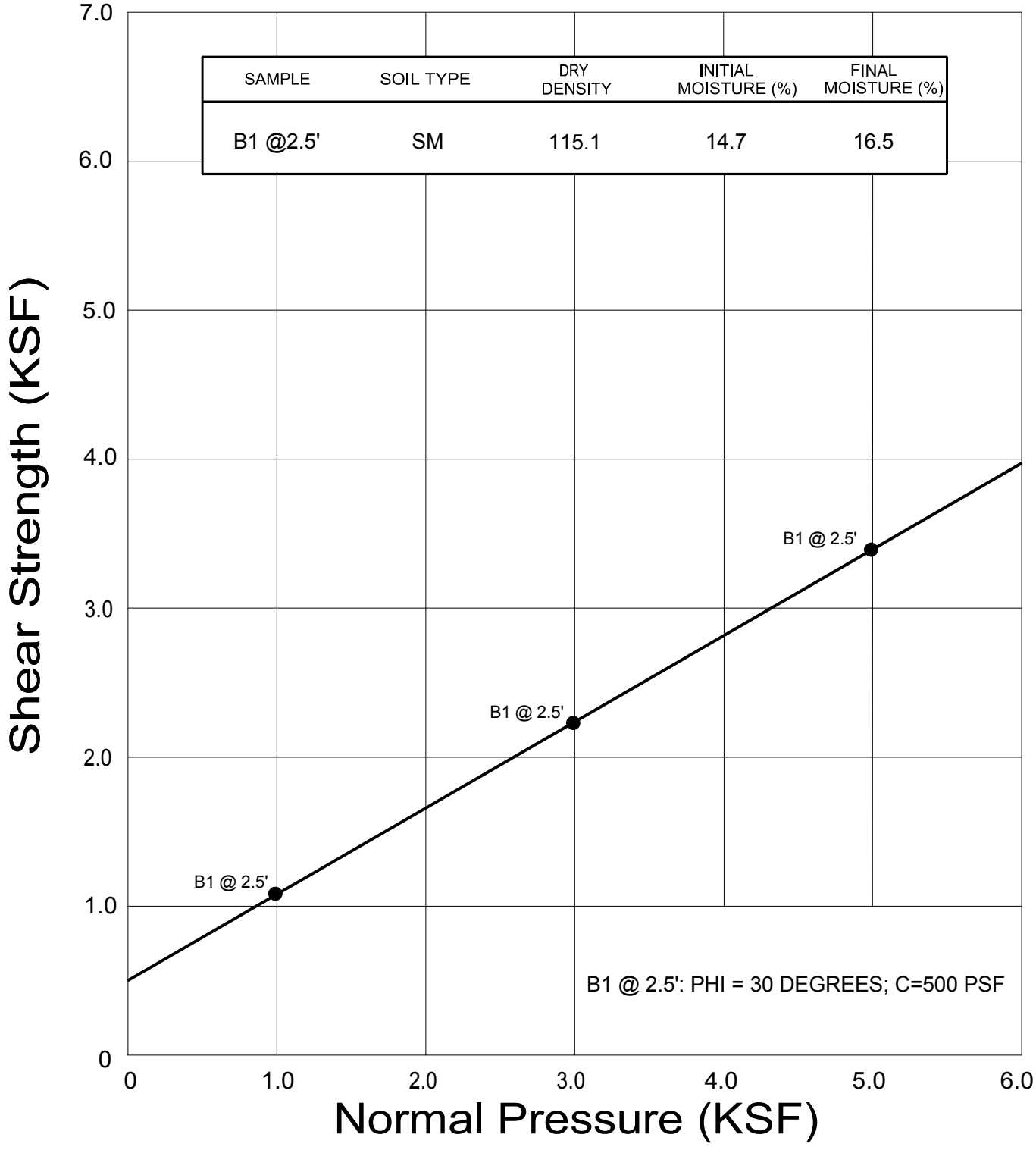
DRAFTED BY: RP | CHECKED BY: HHD

DIRECT SHEAR TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. B1

SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @2.5'	SM	115.1	14.7	16.5



● Direct Shear, Saturated

GEOCON
WEST, INC.

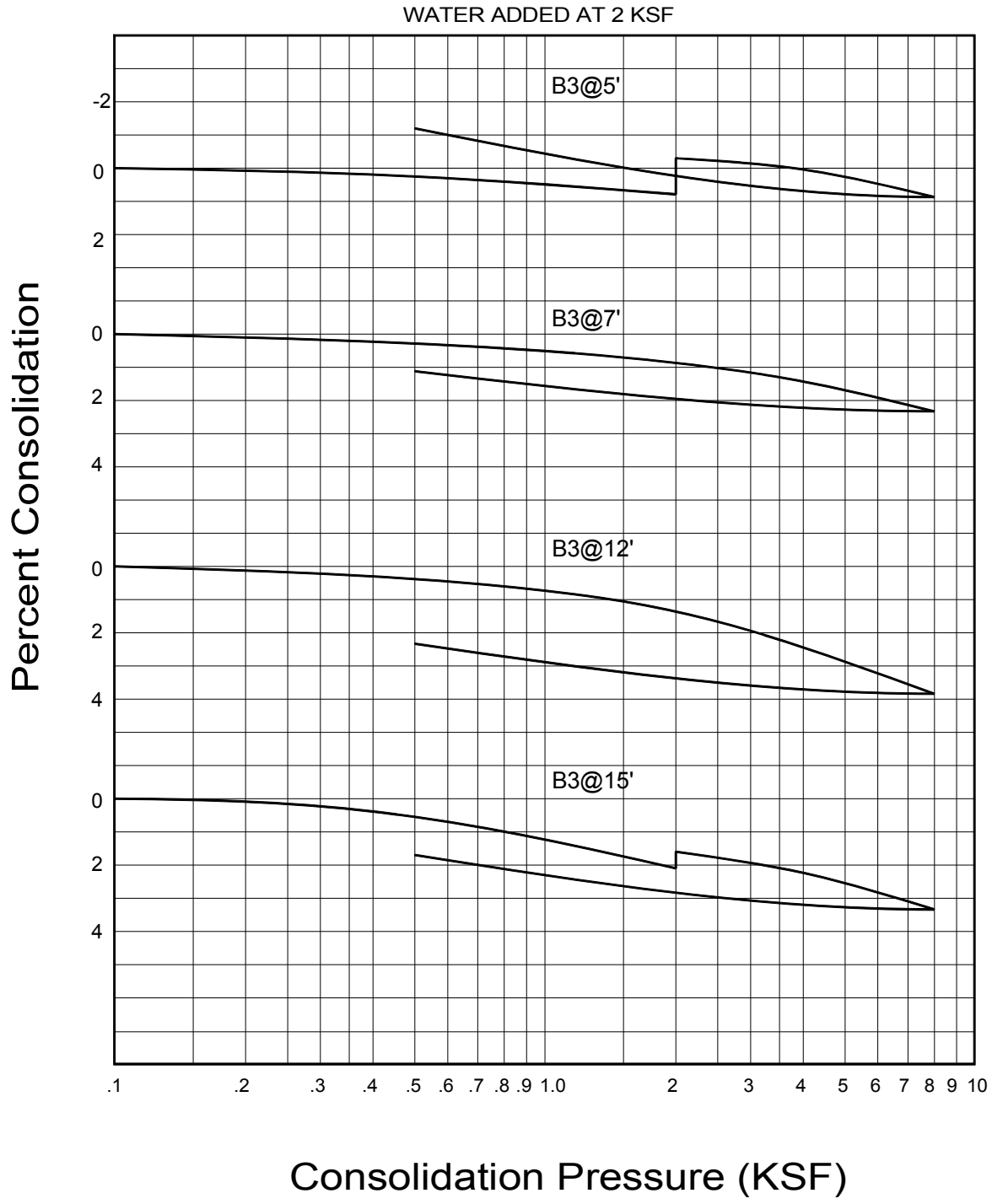
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMH | CHECKED BY: HHD

DIRECT SHEAR TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. B2



GEOCON
WEST, INC.

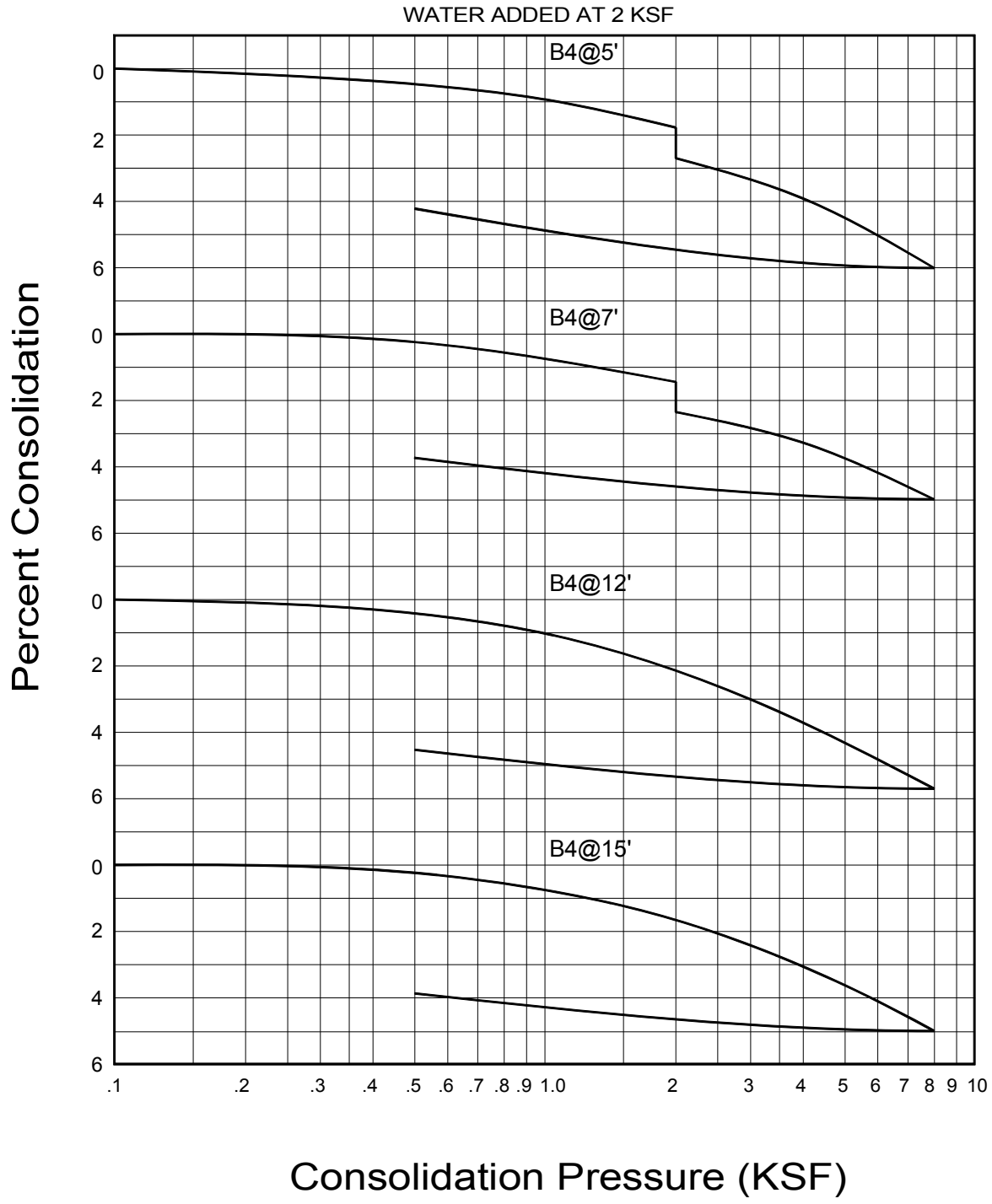
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RP	CHECKED BY: HHD

CONSOLIDATION TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018	PROJECT NO. A9733-06-01	FIG. B3



GEOCON
WEST, INC.

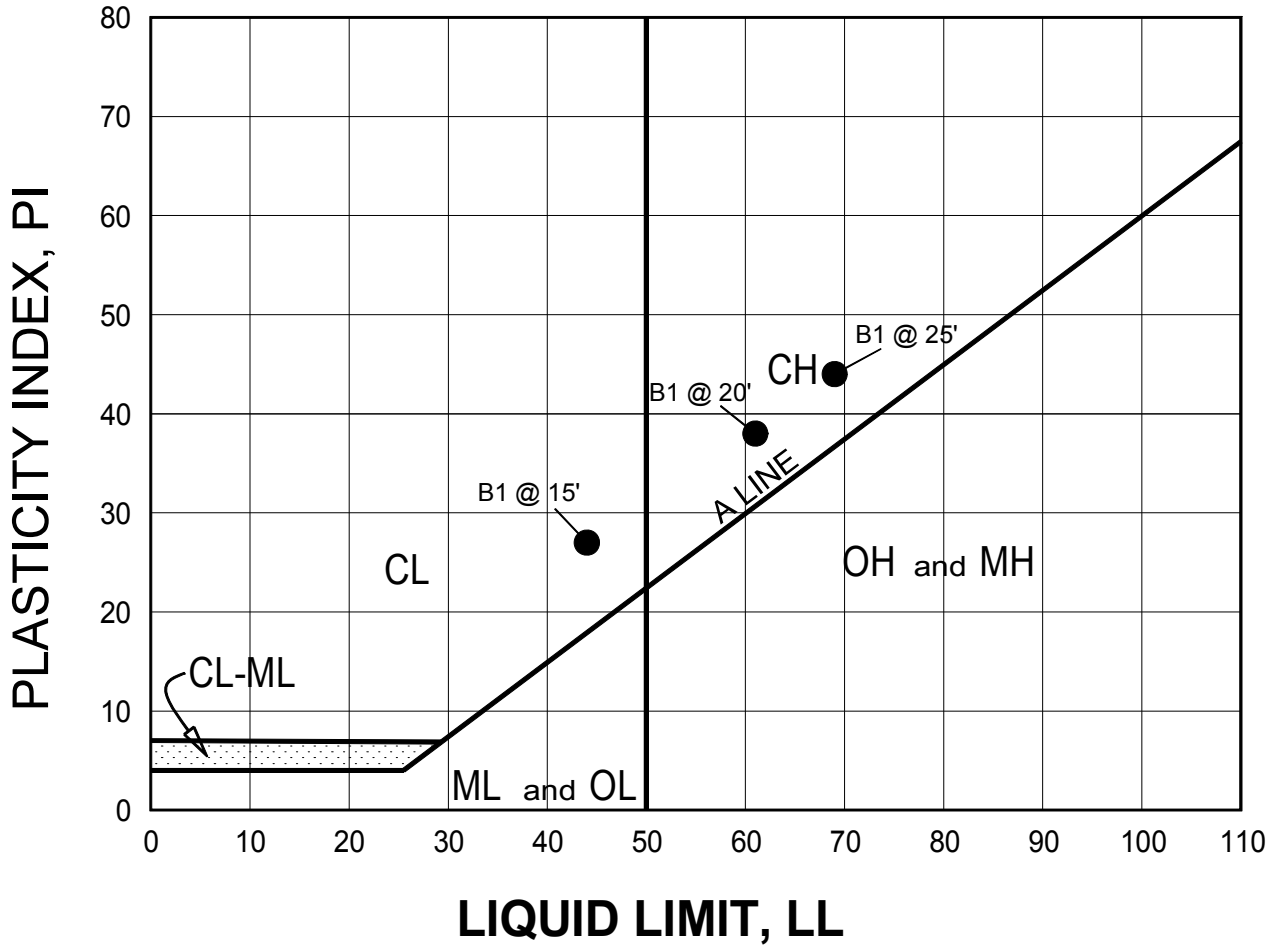
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RP	CHECKED BY: HHD

CONSOLIDATION TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018	PROJECT NO. A9733-06-01	FIG. B4



BORING NUMBER	DEPTH (FEET)	LL	PL	PI	MOISTURE CONTENT AT SATURATION (%)	SOIL BEHAVIOR
B1	5.0	24.0	18.0	6.0	---	*N/P
B1	15.0	44.0	17.0	27.0	15.1	CL
B1	20.0	61.0	23.0	38.0	16.1	CH
B1	25.0	69.0	25.0	44.0	19.1	CH

*N/P indicates Non-Plastic

GEOCON WEST, INC.

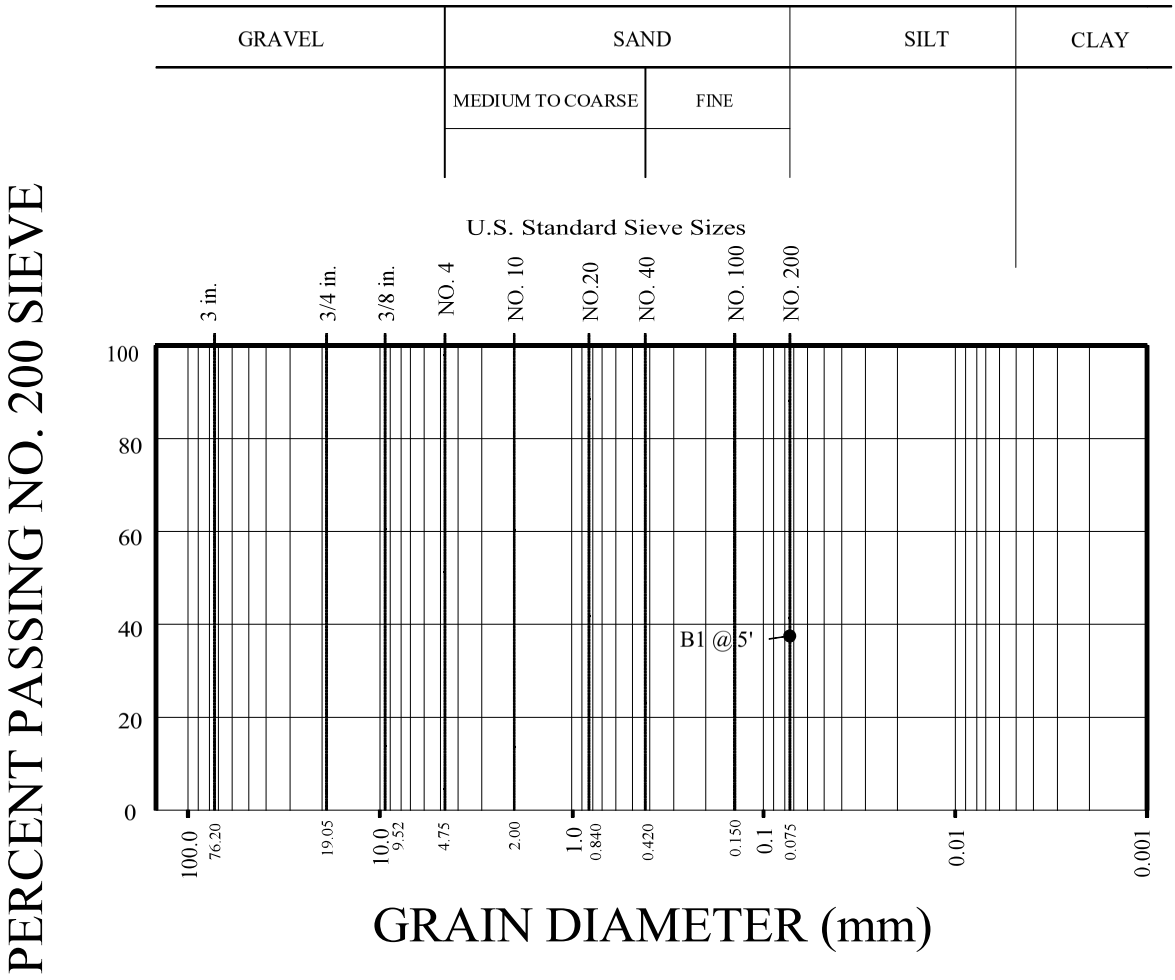
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMH | CHECKED BY: HHD

ATTERBERG LIMIT

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. B5



SAMPLE	PERCENT PASSING NO. 200 SIEVE
B1 @ 5'	37.6

GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RP	CHECKED BY: HHD

GRAIN SIZE ANALYSIS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018	PROJECT NO. A9733-06-01	FIG. B6

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829-11

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification	**CBC Classification
	Before	After				
B2 @ 0-5'	10.4	24.1	106.1	81	Moderate	Expansive
B5 @ 0-5'	13.1	29.1	100.0	101	High	Expansive

* Reference: 1997 Uniform Building Code, Table 18-I-B.

** Reference: 2016 California Building Code, Section 1803.5.3

SUMMARY OF LABORATORY MAXIMUM DENSITY AND AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-12

Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B2 @ 0-5'	Brown Silty Sand	120.0	12.0
B5 @ 0-5'	Yellowish Brown Sandy SIlt	113.0	16.0

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMH | CHECKED BY: HHD

LABORATORY TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. B7

SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS CALIFORNIA TEST NO. 643

Sample No.	pH	Resistivity (Ohm Centimeters)
B2 @ 0-5'	7.55	740 (Severely Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B2 @ 0-5'	0.013

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS CALIFORNIA TEST NO. 417

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B2 @ 0-5'	0.498	Severe

* Reference: 2016 California Building Code, Section 1904.3 and ACI 318-11 Section 4.3.

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMH | CHECKED BY: HHD

CORROSIVITY TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. B8

APPENDIX F.4

FINAL GEOTECHNICAL INVESTIGATION AND BULK FACTOR REPORT, WOMEN'S AND CHILDREN'S HOSPITAL DEMOLITION, LAC+USC MEDICAL CENTER, 1240 NORTH MISSION ROAD, LOS ANGELES, CALIFORNIA

AECOM Imagine it. Delivered.

Report

Geotechnical Investigation and Bulk Factor

Women's and Children's Hospital Demolition
LAC+USC Medical Center
1240 North Mission Road
Los Angeles, California

Prepared for:
County of Los Angeles Department of Public Works

Project Number: 60597019

October 21, 2019

AECOM
One California Plaza
300 South Grand Avenue
Los Angeles, CA 90071

T: +1 (213) 593 8100
F: +1 (213) 593 8178
aecom.com

October 21, 2019

County of Los Angeles
Department of Public Works
Project Management Division I
900 South Fremont Avenue, 5th Floor
Alhambra, CA 91803-1331

Attention: Mr. Juan M Sarda, P.E.
Project Manager

Subject: Final Report
Geotechnical Investigation and Bulk Factor
Women's and Children's Hospital Demolition
LAC+USC Medical Center
1240 North Mission Road
Los Angeles, California
Project No. 60597019

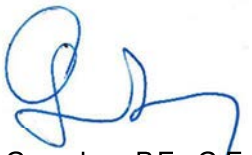
Dear Mr. Sarda:

AECOM Technical Services, Inc. (AECOM) is pleased to present the results of our geotechnical investigation support services performed for the proposed Women's and Children's (WC) Hospital Demolition (Project) at LAC+USC Medical Center (LAC+USC) located at 1240 North Mission Road in Los Angeles, California. This report summarizes the results of our investigation and contains our geotechnical engineering recommendations for the proposed demolition and subsequent make ready construction of the Project.

AECOM prepared this report exclusively for the Project Management Division (PMD) of the Los Angeles County Department of Public Works (LACDPW) for its use in project planning, design, and construction. If you have any questions regarding this report, or if we can be of further service, please contact us at 213-593-8100.

Sincerely,

AECOM Technical Services, Inc.



Garry Lay, P.E., G.E.
Principal Engineer, Vice President

AECOM

Table of Contents

Section 1 Introduction 1-1
1.1 General 1-1
1.2 Project Description 1-1
1.3 Purpose and Scope of Work 1-1
Section 2 Field Investigation and Laboratory Testing 2-1
2.1 Review of Existing Data 2-1
2.2 Geotechnical Field Exploration 2-1
2.2.1 Pre-Feld Activities 2-1
2.2.2 Geotechnical Soil Boring 2-1
2.3 Field Infiltration Tests 2-2
2.4 Geotechnical Laboratory Testing 2-3
Section 3 Site Conditions 3-1
3.1 Regional Geology 3-1
3.2 Local Geology 3-1
3.3 Faulting and Seismicity 3-2
3.4 Previous Site Development 3-4
3.5 Surface Conditions 3-4
3.6 Subsurface Conditions 3-4
3.7 Groundwater 3-4
3.8 Generalized Soil Profile 3-5
Section 4 Geological and Seismic Hazards Evaluation 4-1
4.1 General 4-1
4.2 Geological Hazards 4-1
4.2.1 Subsidence 4-1
4.2.2 Oil Wells 4-1
4.2.3 Flooding 4-1
4.2.4 Methane 4-2
4.2.5 Hydrocollapse 4-2
4.2.6 Expansive Soil 4-2
4.3 Seismic Hazards 4-2
4.3.1 Ground Rupture 4-2
4.3.2 Ground Motion 4-3
4.3.3 Liquefaction and Seismically-Induced Settlement 4-4
4.3.4 Lateral Spread Displacement 4-5
4.3.5 Earthquake-Induced Flooding 4-5
4.3.6 Other Seismic Hazards 4-5
Section 5 Discussion and Recommendations 5-1
5.1 General 5-1
5.2 Earthwork 5-1
5.2.1 Site Preparation 5-1
5.2.2 Structure Foundation Demolition 5-1
5.2.3 Over-excavations 5-2
5.2.4 Fill and Backfills 5-2
5.2.5 Grading Factor 5-4
5.2.6 Temporary Excavation 5-4
5.2.7 Temporary Shoring 5-5
5.3 Interim Slopes 5-5

5.4 Permanent Slopes 5-6
5.5 Retaining Structure 5-6
5.5.1 Bearing Capacity 5-6
5.5.2 Settlement 5-6
5.5.3 Lateral Resistance 5-6
5.6 Concrete Flatwork 5-6
5.7 Site Drainage 5-6
5.8 Low Impact Development Mitigation Measure 5-7
5.9 Pavement Section Design 5-7
5.10 Utilities 5-8
5.11 Corrosivity 5-8
5.12 County Building Code Section 111 Statement 5-9
Section 6 Construction Considerations 6-1
6.1 Excavation 6-1
6.2 Groundwater 6-1
Section 7 Design Review 7-1
Section 8 Construction Monitoring 8-1
Section 9 References 9-1
Section 10 Uncertainties and Limitations 10-1

AECOM

Figures

Figure 1	Vicinity Map
Figure 2	Plot Plan
Figure 3	Regional Geologic Map
Figure 4	Site Geologic Map
Figure 5	Regional Fault Map
Figure 6	Geologic Cross-Section A-A’
Figure 7	Geologic Cross-Section B-B’
Figure 8	Oil, Gas & Geothermal - District 1 Map 119
Figure 9	Fault Zone Map
Figure 10	Seismic Hazard Zone Map
Figure 11	Lateral Earth Pressures for Temporary Shoring
Figure 12	Lateral Earth Pressures
Figure 13	Pavement Section Design
Figure 14	Vertical Earth Pressures Buried Objects

Tables

Table 1 - SUMMARY OF SOIL BORINGS 2-2
Table 2 - FIELD INFILTRATION TEST SUMMARY 2-2
Table 3 - MAJOR FAULT CHARACTERIZATION 3-3
Table 4 - GENERALIZED SUBSURFACE PROFILE 3-5
Table 5 - SEISMIC DESIGN PARAMETERS 4-3
Table 6 - LIQUEFACTION POTENTIAL AND SEISMIC SETTLEMENT 4-4
Table 7 - SELECT BACKFILL CRITERIA 5-3
Table 8 - GREENBOOK AGGREGATE BASE GRADATION CRITERIA 5-3
Table 9 - CALTRANS CLASS 2 AGGREGATE BASE GRADATION CRITERIA 5-3
Table 10 - AGGREGATE BASE QUALITY CHARACTERISTICS 5-4
Table 11 - SLOPE STABILITY ANALYSES 5-4
Table 12 - DESIGN INFILTRATION RATE 5-7
Table 13 - CORROSION POTENTIAL 5-8

Appendices

Appendix A	Relevant Previous Geotechnical Data
Appendix B	Field Exploration and Logs of Boring
Appendix C	Field Infiltration Test
Appendix D	Geotechnical Laboratory Testing
Appendix E	Liquefaction and Seismic Settlement Analyses
Appendix F	Slope Stability Analyses

Section 1 Introduction

1.1 General

This report presents the results of AECOM Technical Services, Inc. (AECOM) geotechnical support services for the proposed Women's and Children's (WC) Hospital Demolition (Project) at LAC+USC Medical Center (LAC+USC) located at 1240 North Mission Road in Los Angeles, California. The location of the Project site relative to the surrounding area is shown on the Vicinity Map, Figure 1. Generally, the Project site is located at the northwest corner of the campus, bounded by Zonal Avenue to the north, North Mission Road to the west, Parking Lot 10 to the south, and the proposed LAC+USC Restorative Village to the east. The geographic coordinates of the subject improvements are 34.0617° North and 1f17.2113° West.

This geotechnical investigation was performed for the Project Management Division (PMD) of Los Angeles County Department of Public Works (LACDPW) under the As-needed Geotechnical Engineering, Materials Testing and Inspection Services, Consultant Services Agreement, PW15013, between LACDPW and AECOM.

The report provides the findings from our subsurface explorations and laboratory testing, our interpretation of the geologic and geotechnical conditions encountered, and recommendations for preliminary pad design and planned demolition of the Project. Conclusions and recommendations presented in this report are based on subsurface conditions encountered at our exploration locations. As subsurface conditions may vary at different locations, these conclusions and recommendations should not be extrapolated to other areas, or used for other facilities, without our prior review. This report excludes any discussion of the environmental issues at the Project site.

1.2 Project Description

The Project area encompasses the LAC+USC WC Hospital with appurtenances structures (i.e. Cooling Tower/Air Compressor Unit and WC Hospital Storage) and surrounding buildings/trailers for a total plan area of approximately 170,000 square feet. We understand that the Project will consist of demolishing the existing WC Hospital Building (Building 401), Compressor Unit (LBL 443), Hospital Storage area (LBL 444), various buildings/trailers including Childcare Trailer (421), Guard House (445), Trailer 7A - Peds Trailer (425), Trailer 7B - Record Storage (426), Trailer 7C - Record Storage (427) and Trailer 11 (432), as shown on Figure 2. Since there is a basement underneath the existing Building 401, a portion of the concrete debris generated from demolition may be crushed onsite and used for backfilling the void left behind by the existing basement, while the excess debris will be export after demolition. Otherwise the backfill of the demolished basement may require import soil and export all concrete debris due to air quality consideration. After demolition, the Project site will be graded and developed into a surface parking lot for interim. Eventually, the site will be redeveloped for the future psychiatric tower.

The subject report will only address the planned demolition of the existing structures as well as the interim grading for the proposed surface parking lot.

1.3 Purpose and Scope of Work

The purpose of our proposed geotechnical support services will be to explore the subsurface conditions, to conduct laboratory testing, and to provide estimated bulking factors for the materials generated (primarily soil and concrete debris) to support quantity estimates to be performed by the Project Civil Engineer, and the geotechnical recommendations related to the proposed interim grading.

Our services were performed in accordance with the requirements of the 2016 California Building Code (CBC)/2017 County of Los Angeles Building Code (CLABC) and American Society of Civil Engineers

(ASCE)/Structural Engineering Institute (SEI) 7-10 Standards with the field and laboratory testing procedures following the pertinent American Society for Testing and Materials (ASTM) and California Test Method (CTM) Standards. Seismic hazard evaluation will be based on guidelines established by the California Geological Survey (CGS) in the Special Publication (SP) 117A.

The scope of services as outlined in the work authorization No. A15013CP-00016 was followed closely and generally included the following tasks:

1) Reviewed available geologic maps/reports and previous geotechnical information relative to the Project;
2) Obtained a permit from Los Angeles County Department of Environmental Health (EH) for explorations penetrating the groundwater, or deeper than 10 feet;
3) Notified Underground Service Alert (USA) prior to beginning of our field investigation program;
4) Explored subsurface conditions by drilling and sampling 10 geotechnical exploratory borings;
5) Conducted 2 field infiltration tests to collect design information for low impact development (LID) mitigation measures;
6) Performed laboratory testing on selected soil samples obtained from the borings to evaluate soil index, shear strength, consolidation potential, expansion index, sand equivalent, maximum density, and corrosion potential;
7) Performed geologic/seismic hazard evaluation including liquefaction and seismic-induced settlement analyses based on the updated seismic design criteria, and provide applicable mitigation measures, where required;
8) Performed engineering analyses to develop geotechnical recommendations related to the planned demolition and the subsequent interim site development; and
9) Prepared this report containing our findings and recommendations for the proposed improvements in accordance with the current standards and requirements, including:
    - Brief description of the Project;
    - Plan drawn to scale depicting the locations of relevant explorations;
    - Summary of the relevant field and laboratory testing programs;
    - Subsurface soil conditions, soil types, groundwater information;
    - Discussion of the site geologic and subsurface conditions;
    - Two geologic cross sections;
    - Geological and seismic hazards at the site, including subsidence, ground shaking, ground rupture, seismic-induced settlements, liquefaction, and lateral spreading potential in accordance with 2008 CGS SP 117A, CGS published hazard maps and reports, and 2016 CBC, including seismic design criteria based on the ASCE 7-10;
    - Generalized geotechnical design profile and shear strength parameters for foundation design and excavation support, if applicable, based on the field and laboratory data;
    - Active, passive and at-rest earth pressure under static and seismic loading conditions for retaining wall;
    - Recommended shallow foundation design capacities;
    - Magnitude of total and differential footing settlements;
    - Temporary excavation and shoring alternative, and associated design input, as well as the potential impact to the existing adjacent structure;
    - Fill material and compaction requirements, including bedding and pipe zone material, and suitability of on-site soils for utility backfill;
    - Earthwork recommendations;
    - Geotechnical consideration for low impact development (LID) mitigation measures;

- Corrosion potential to steel and concrete will be evaluated based on ACI 318 and Caltrans Corrosion Guideline;
- Bulking factors;
- Construction monitoring recommendations; and
- Los Angeles County Building Code Section 111 compliance statement.

Section 2 Field Investigation and Laboratory Testing

The geotechnical investigation included reviewing available geotechnical and geologic data, as-built information and performing subsurface explorations. Geotechnical laboratory testing was conducted on selected soil samples collected during the field exploration program.

2.1 Review of Existing Data

Available geologic and geotechnical documents reviewed during the current investigation are listed in Section 9. Pertinent information from these documents was incorporated into the preparation of this report. Relevant field data and previous studies within the project vicinity are attached in Appendix A.

2.2 Geotechnical Field Exploration

The field investigation consisted of exploratory nine hollow-stem auger (HSA) borings and one mud-rotary wash boring.

2.2.1 Pre-Feld Activities

AECOM conducted pre-field activities including USA notification prior to commencing soil sampling and testing activities to identify underground utilities in relation to the Project site. We also prepared a site-specific Health and Safety Plan (HASP), which was reviewed and approved by an AECOM Certified Industrial Hygienist (CIH). Our subconsultant, Southwest Geophysics Inc. (SGI), conducted a geophysical survey to further verify the locations of suspected underground utilities utilizing a variety of non-destructive techniques such as ground penetrating radar (GPR), electromagnetic, electrical resistivity, and magnetometer surveys to check for underground utilities.

In addition, AECOM conducted a site kickoff meeting with facility personnel to discuss logistical matters (i.e. site access and other site requirements/restrictions) for the field exploration program on January 24, 2019.

2.2.2 Geotechnical Soil Boring

Nine soil borings were completed at the site using hollow-stem auger drilling technique between January 28, 2019 and February 8, 2019 and one soil boring was completed at the site using mud-rotary drilling technique on February 7, 2019 at the locations shown on Figure 2. The borings were drilled to a maximum depth of 81 feet below the existing ground surface (bgs). The boring depths were selected to extend below the anticipated building basement to evaluate the slope stability during demolition and to support shoring design if necessary. A summary of the soil boring operations is provided in Table 1. Details of the field investigation, including sampling procedures, and boring logs are presented in Appendix B.

TABLE 1 - SUMMARY OF SOIL BORINGS

APPROXIMATE LOCATION	BORING NUMBER	APPROXIMATE COORDINATES			APPROXIMATE BORING DEPTH (feet)	APPROXIMATE BORING BOTTOM ELEVATION (feet - MSL)
		NORTHING[1] (feet)	EASTING[1] (feet)	SURFACE ELEVATION[2] (feet MSL)		
Trailer 432	B-19-01	6497703.343	1845147.383	331.0	21.5	309.5
Trailer 426	B-19-02	6497695.304	1845069.521	330.5	61.5	269.0
Between 401 and 427	B-19-03	6497762.583	1844944.191	322.5	41.5	281.0
Behind Storage Area	B-19-04	6497906.238	1844843.084	318.5	21.5	297.0
LBL #444	B-19-05	6497809.683	1844729.015	320.0	81.0	239.0
Front of WC	B-19-06	6497532.463	1844832.223	332.5	21.5	311.0
Adjacent to Trailer 425	B-19-07	6497457.282	1844944.105	333.0	41.5	291.5
Adjacent to Trailer 421	B-19-08	6497591.091	1845058.458	332.5	21.5	311.0
Front of WC	B-19-09	6497524.459	1845058.599	330.0	11.5	318.5
Adjacent to Trailer 421	B-19-10	6497681.011	1844753.304	333.0	17.5	315.5

Notes:
1. Approximate locations. Northing and easting based on California Zone 05, NAD83.
2. Based on topo map; MSL=mean sea level.

2.3 Field Infiltration Tests

Two field infiltration tests were conducted by AECOM to evaluate the in-situ soil permeability characteristics of the project site. The tests were performed following the "Guideline for geotechnical Investigation, and Reporting, Low Impact Development Stormwater Infiltration (GS200.2)" by LACDPW Geotechnical and Materials Engineering Division (GMED). The adjusted field infiltration test results without applicable factors of safety were summarized in Table 2. Details of the test procedures and results are summarized in Appendix C.

TABLE 2 - FIELD INFILTRATION TEST SUMMARY

APPROXIMATE LOCATION	WELL NUMBER	TEST LOCATION			INFILTRATION LAYER		MEASURED PERCOLATION RATE (inches per hour)
		NORTHING[1] (feet)	EASTING[1] (feet)	SURFACE ELEVATION[2] (feet MSL)	DEPTH (feet)	ELEVATION (feet)	
Front of WC	B-19-09	6497524.459	1845058.599	330	6 to 10	324 to 320	2.72
Adjacent to Trailer 421	B-19-10	6497681.011	1844753.304	333	11 to 16	322 to 317	2.64

Notes:
1. Approximate locations. Northing and easting based on California Zone 05, NAD83.
2. Based on topographic map; MSL=mean sea level.

2.4 Geotechnical Laboratory Testing

Geotechnical soil samples obtained from the borings were carefully sealed and packaged in the field to reduce moisture loss and disturbance. The samples were delivered to the appropriate testing laboratories, where they were further examined and classified. Laboratory testing was performed on selected samples to confirm (and to modify if necessary) the visual classification of the soils based on the field identification. Geotechnical tests performed included:

- Soil classification (ASTM D2488);
- Moisture content test (ASTM D2216);
- In-situ density test (ASTM D2937);
- Particle size analysis (ASTM D422, D1140);
- Atterberg limits (ASTM D4318);
- Direct shear test (ASTM D3080);
- Maximum dry density and optimum moisture content (ASTM D1557);
- Consolidation test (ASTM D2435);
- Expansion index (ASTM D4829);
- Chemical (Corrosion) Test (Resistivity, Sulfate, pH and Chloride) (California Test Methods 417, 422, 532, & 643).

A description of the laboratory testing and the test results are presented in Appendix D of this report. For ease of reference, test results of moisture, in-situ density, and fines content of soils tested are also shown on the Log of Boring in Appendix B.

Section 3 Site Conditions

AECOM' knowledge of the site geology and subsurface conditions has been developed from a review of the area's geology, historical information, and relevant field and laboratory programs performed at the Campus.

3.1 Regional Geology

The site is located within the northern portion of the Peninsular Ranges Geomorphic Province, which is characterized by a series of generally northwest-trending mountain ranges and intervening valleys. The northern portion of the peninsular Ranges Province, generally referred to as the Los Angeles Basin is a broad sediment filled trough.

The project site is located on the southern edge of the northeast block of the Los Angeles Basin. The Los Angeles Basin is situated at the juncture of the Transverse Ranges and the Peninsular Ranges geomorphic provinces as shown on Figure 3.

The Peninsular Range geomorphic province is characterized by northwest-southeast trending alignments of mountain ranges and hills with intervening basins reflecting the influence of northwest trending major fault zones controlling the geologic structure of the region. The Peninsular Ranges are composed of metamorphosed sedimentary and volcanic rocks of Jurassic age intruded by mid-Cretaceous plutonic rock of the southern California batholith. The geologically complex and seismically active Transverse Ranges geomorphic province is characterized by east-west trending mountains, intervening valleys, and geologic structures. This topographic and geologic pattern is formed by north-south crustal compression that is the consequence of a left (restraining) bend in the San Andreas fault, known as the "Big Bend". Many of the active faults in the Transverse Ranges are thrust or high-angle reverse, left lateral, oblique slip faults. These two geomorphic provinces have contrasting tectonic characteristics that overlap within the Los Angeles Basin resulting in a complex tectonic environment marked by active faulting and historic seismicity. The Los Angeles Basin is cross by numerous northwest-trending active faults and is underlain by several blind thrust faults.

The site lies within the east-west trending Repetto Hills. The Repetto Hills are the topographic expression of an uplifted fold structure known as the Elysian Park Anticline. The Elysian Park Anticline is the result of movement along the Elysian Park thrust. The Elysian Park Thrust is a north-dipping "blind" thrust that underlies this portion of the Los Angeles Basin. The project site is located on the southernmost margin of the south limb of the Elysian Park Anticline along a narrow zone of sharp folding which occurs as secondary structural features within the south limb of the Elysian Park Anticline (Soper, 1943).

3.2 Local Geology

The Project Site as shown on Figure 4 is located along the southern edge of the Repetto Hills and is underlain by a thick sequence of Miocene age marine sedimentary rocks known as the Puente Formation. Since their deposition, these rocks have been folded and faulted, and eventually uplifted to form the present-day hills.

The subsurface site conditions consist of a mantle of artificial fill overlying sedimentary bedrock of the Miocene age Puente Formation (Yorba Member). The distribution of artificial fill deposits was the result of the development of hospital structures. The southernmost portion of the LAC+USC site consists of broad southwest-dipping gently sloping topography consisting of upper Quaternary-age unconsolidated alluvial deposits. These alluvial deposits were deposited predominantly by the ancestral Los Angeles River and canyon streams draining the local uplands during a period of continued uplift.

3.3 Faulting and Seismicity

The Los Angeles Basin, as well as most of Southern California, is located within a complex zone of faults and folds resulting from north-south directed compressional forces resulting from the east-west deflection of the San Andreas fault that forms the northern boundary of the Transverse Ranges. Numerous generally east-west to northwest trending faults have formed as a result of these north-south compressional forces acting within this area. The major faults within the vicinity of the Los Angeles Basin are characterized by a combination of blind thrust, left- and right-lateral strike-slip, and reverse faulting.

Numerous active and potentially active faults occur throughout southern the California area. As defined by the California Geologic Survey (CGS), an active fault is a fault that has evidence of displacement within Holocene time (back to approximately 11,000 years before present) and a potentially active fault is a fault that has evidence of displacement within Pleistocene time (between 11,000 and 1.6 million years before present; CGS 2018).

The Regional Fault Map, as shown on Figure 5, includes the location of active and potentially active faults, within the general vicinity of the LAC+USC Campus (Campus). Most of the larger earthquakes have been associated with larger faults that have been mapped at the ground surface. Several moderate to large earthquakes in the region have also occurred on deep-seated buried thrust faults in this geological complex region of Southern California. The most recent significant earthquake was the magnitude 6.7 Northridge earthquake, which occurred on a shallowly south-dipping thrust fault that underlies much of the San Fernando Valley.

The closest known active fault to the site with evidence of surface rupture is the Raymond fault which is located approximately 3.8 miles (6.0 kilometers) to the north of the Campus. The Raymond fault is primarily a left-lateral strike-slip fault with a minor component of high angle reverse offset, placing basement rocks north of the fault over alluvial sediments south of the fault. This fault extends southwesterly from the Sierra Madre fault zone at the base of the San Gabriel Mountains to the Raymond Hill area of South Pasadena where it trends in a westerly direction into Los Angeles. The Raymond fault has long been recognized as a groundwater barrier in the vicinity of the City of Pasadena and numerous geomorphic features along its entire length. This is best expressed by an escarpment in the San Marino and Pasadena areas, where it forms a prominent escarpment up to 150 feet high. The most recent fault movement, based on radiocarbon ages from materials collected in an excavation shows that it occurred sometime between 1,600 to 2,100 years before present (Weaver and Dolan, 2000). An average slip rate of 1.5 millimeter per year and a maximum magnitude of 6.5 are estimated by CGS (2018). The largest historical earthquake was the 1988 Pasadena Earthquake with a magnitude of 5.0.

Other active faults in the area include the Hollywood fault located approximately 5.4 miles to the northwest, and the Verdugo fault Zone located approximately 5.5 miles to the north. The historically active San Andreas fault is located approximately 32.8 miles to the north. The closest active blind thrust fault is the Elysian Park thrust.

The Elysian Park thrust is a blind thrust fault, which underlies the area and produced the uplift of the Elysian Park Anticline. A series of south convergent folds and flexures are interpreted to result from compression above this north dipping blind thrust fault (Oskin and others, 2000). The accommodation of movement produced by the Elysian Park thrust appears to be primarily as folding of the near surface deposits as opposed to active faulting. Oskin and others (2000) reported that the Elysian Park Anticline lacks a surficial exposed frontal fault system but rather contraction at the surface is evident as fold growth.

Several faults have been mapped throughout the Miocene age Puente Formation bedrock along the flanks of the Elysian Park Anticline (Lamar, 1970) but none of these have been shown to be active within Holocene time. Soper (1943) reported that there is surface evidence of both normal and reverse faulting at the east end of the Los Angeles City Oil Field but provided no further details regarding the nature of this evidence. The characteristics of the nearby faults are summarized in Table 3.

The closest potentially active faults within the area include the MacArthur Park Escarpment. The MacArthur Park Escarpment, which is located approximately 4.1 miles to the southwest of the Campus has previously been interpreted as a potentially active fault (Dolan et al., 2000). However, Oskin et al. (2000) concluded that the MacArthur Park Escarpment is part of a 20-kilometer fold that traverses the south limb of the Elysian Park Anticline.

TABLE 3 - MAJOR FAULT CHARACTERIZATION

FAULT[1]	APPROXIMATE DISTANCE[2] (kilometers)	APPROXIMATE DISTANCE[2] (miles)	TYPE OF FAULT	MAXIMUM EARTQUAKE MAGNITUDE[3] (Mw)
Upper Elysian Park	0.8	0.5	Blind Thrust	6.40
Puente Hills	5.3	3.3	Blind Thrust	6.92
Raymond	6.0	3.8	Reverse Oblique	6.55
Hollywood	8.7	5.4	Reverse/ Strike Slip Component	6.54
Verdugo	8.9	5.5	Reverse	6.75
Newport-Inglewood	15.4	9.6	Strike Slip	6.99
Whittier	16.4	10.2	Reverse-Oblique	6.83
Sierra Madre	16.8	10.4	Reverse	7.01
Santa Monica	18.8	11.7	Reverse	6.63
Clamshell-Sawpit Canyon	23.4	14.5	Reverse	6.50
San Gabriel	23.8	14.8	Strike Slip	7.08
Malibu Coast	29.4	18.3	Reverse	6.48
Northridge Hill	29.5	18.3	Blind Thrust	6.74
Palos Verdes	30.3	18.8	Strike Slip	7.15
San Jose	30.5	19.0	Strike Slip	6.40
Santa Susana	31.0	19.3	Thrust	6.73
Anacapa-Dume	32.0	19.9	Reverse	7.05
Chino-Central Ave. (Elsinore)	42.9	26.7	Strike Slip	6.70
Cucamonga	43.7	27.2	Thrust	6.90
San Joaquin Hills	47.7	29.7	Blind Thrust	6.60
San Andreas	52.8	32.8	Strike Slip	7.10

Notes:
1. Fault data based on Uniform California Earthquake Rupture Forecast, Version 3 (UCEFF3; Field et al., 2013).
2. Distance to fault, R_x, which is defined as the perpendicular distance from the site to the surface projection of the top of fault.
3. The maximum earthquake magnitude is estimated using the Leonard 2010 magnitude-area scaling relations.

3.4 Previous Site Development

Prior to the development of the area, a drainage channel passed near the eastern edge of the current hospital. The drainage channel was channelized prior to the construction of the previous psychopathic ward.

Prior to the construction of the current WC, the ground surface varied approximately 20 feet in elevation and was essentially at three levels separated by retaining walls. The area occupied by the current WC was occupied by an old chapel at the top of the retaining wall. The area adjacent to the previous psychopathic ward, and within the surrounding retaining walls was at a higher elevation. The previous psychopathic ward was at a low point in elevation. The surface of the asphalt roadway on the west of the psychopathic ward and the current WC hospital is approximately 10 feet higher in elevation.

Construction of the WC required grading work to level out of the project site. The retaining walls were removed for the construction of the basement and foundation of the WC hospital.

3.5 Surface Conditions

Project area encompasses the WC with appurtenances structures and surrounding minor buildings/ trailers. The project site is surrounded by asphalt-concrete surface parking lot and paved access road leading around the structures. The unoccupied trailers adjacent to the WC are in fair to poor condition. There is a large occupied 3-story parking structure (Lot 10) to the west adjacent to North Mission Road immediately outside the Project limits. The previous demolished psychopathic ward is to the southeast and is currently an empty unpaved lot. The general topography of the Project site is relatively level with the existing ground elevations of the Project site ranged from approximately 310 to 330 feet above mean sea level (MSL). The project site has the highest elevation at the northwest side and slopes towards the southeast.

3.6 Subsurface Conditions

Based on the information from the investigations, the proposed Project is underlain by a layer of fill approximately up to 25 feet deep in certain areas due to excavation required during the construction of the existing basement. However, backfill may also be anticipated along underground utilities (i.e., storm drains) depending on the depths of respective lines. The fill is primarily of silty sand, silty, and sandy clay mixture. The fill is generally medium dense for silty sand, soft to medium stiff for silty and sandy clay mixture. Below the fill, alluvium was encountered to the maximum explored depths with an exception of the Boring B-19-05. The alluvial soils generally consist of medium dense to very dense silty sand, clayey sand, and poorly graded sand, and hard sandy silt and sandy lean clay. Overall, the uncorrected standard penetration test (SPT) blow counts ranged from 19 to 31 for coarse-grained soils. Bedrock was encountered in boring B-19-05 at approximately Elevation 250 feet MSL. The subsurface conditions are presented on Figure 6 and Figure 7.

3.7 Groundwater

According to the Seismic Hazard Zone Report 029 of the Los Angeles 7.5-Minute Quadrangle, the historically highest groundwater is on the order of 20 feet or shallower below existing grade (CGS 1998). However, according to the State Water Resources Control Board (SWRCB) Geotracker published data, the 10-year historic high groundwater level was 13.49 feet bgs for Well No. TSG-MW-75 located approximately 785 feet to the southwest of the Campus. Groundwater data collected from two previous geotechnical investigations (AECOM, 2015 and Geokon, 2018) east of the Project Site. The two reports indicate groundwater level is on the order of 17 to 20 feet below adjacent grade, which corresponds to

approximately Elevation 300 feet MSL. All historic groundwater data collected were from monitoring wells located in the surrounding areas underlain by unconsolidated alluvial deposits.

Groundwater was encountered in 3 geotechnical borings ranging between 32 and 38 bgs, with highest elevation at 298 feet MSL. The variations in groundwater depths could be the result of seasonal rainfall fluctuations. Perched water may also be encountered at shallower zones during construction especially during and immediately after periods of heavy rainfall.

However, considering the site terrain and complex geology at the site, a design groundwater table at Elevation 300 feet MSL may be considered for the subject demolition for conservatism.

3.8 Generalized Soil Profile

For design purposes, the site subsurface conditions were summarized into a generalized profile and the geotechnical parameters of the idealized profile are presented in Table 4. However, the building basement does step down in elevation at various locations.

TABLE 4 - GENERALIZED SUBSURFACE PROFILE

SOIL TYPE	ELEVATION (feet MSL)	DEPTH[1] (feet bgs)	TOTAL UNIT WEIGHT (pcf)	SPT-N[2] (bpf)	SHEAR STRENGTH		
					EFFECTIVE		TOTAL
					COHESION (psf)	FRICTION ANGLE (degrees)	UNDRAINED SHEAR STRENGTH (psf)
FILL – stiff **Sandy Clay/** medium dense **Clayey Sand**	330 to 325	0 to 5	125	11 to 36	--	--	1,500
FILL – Medium dense to dense **Poorly-graded Sand/Clayey Sand/Silty Sand**	325 to 310	5 to 15	130	19 to 31	200	29	--
ALLUVIUM – stiff to hard **Lean Clay**	310 to 285	15 to 40	125	3 to 22	300	27	2,500
ALLUVIUM – Medium dense to dense **Silty Sand**	285 to 240	40 to 80	110	31 to 63	--	35	--

Notes:
1. Assumed finished grade at 330 feet MSL.
2. Range indicating maximum and minimum uncorrected values recorded.

3.9 Field Infiltration Characteristics

Data from the field infiltration tests were evaluated in accordance with the aforementioned LID guideline. Based on the investigation the overall Project site consist of fill is primarily medium dense for silty sand, soft to medium stiff for silty and sandy clay mixture. The alluvial soils beneath the fill is generally consist of medium dense to very dense silty sand, clayey sand, and poorly graded sand, and hard sandy silt and sandy lean clay. Two field infiltration tests were performed within the target infiltration layers at different depths throughout the campus. The test results indicated the measured percolation rates around a majority of the campus ranging from 2.64 to 2.72 inches per hour. The shallow tests have slower percolation rates measured due to less permeable clayey soils.

Recommended reduction factors related to test procedure (RFt) and site variability (RFv) 2 and 3, respectively. In addition, the reduction factor related to long-term siltation, plugging, and maintenance (RFs) should be determined by the Project Civil Engineer based on tributary area size, level of pretreatment and expected sediment loads, redundancy, etc. Total reduction factor, RF, should be the product of reduction factors of RFt, RFv and RFs. The design infiltration rate can be determined by dividing the measured percolation rates by the total reduction factor RF.

Section 4 Geological and Seismic Hazards Evaluation

4.1 General

Geological and seismic hazards are those hazards that could impact a site due to the surrounding geologic and seismic conditions. Geological hazards include subsidence, oil wells, flooding, methane, and poor soil conditions. Seismic hazards include phenomena that occur during an earthquake such as ground shaking, ground rupture, differential soil settlement, and lateral spread displacement. The potential impact of these hazards to the site has been assessed and is summarized in the following sections. Our assessment of these hazards was based on guidelines established by California Geological Survey (CGS; formerly California Division of Mines and Geology [CDMG]) as outlined in its Special Publication 117A (2008).

4.2 Geological Hazards

4.2.1 Subsidence

The extraction of water or petroleum from sedimentary source rocks can cause the permanent collapse of pore space previously occupied by the removed fluid. The compaction of subsurface sediment caused by fluid withdrawal will cause subsidence of the ground surface overlying a pumped reservoir. If the volume of water or petroleum is sufficiently great, the amount of resulting subsidence may be sufficient to damage nearby engineered structures. Although a detailed study has not been performed for this report, it is anticipated that the current minor extraction from oil and/or water wells in the vicinity of the site would not result in measurable subsidence at the Project Site, barring a significant increase in water extraction in the future. Therefore, the potential for subsidence is not considered a significant geologic hazard to the project.

According to the Department of Conservation, Division of Oil, Gas & Geothermal Resources (DOGGR), District 1, Map 119, the Project Site is not located within an Oil Field Zone as shown in Figure 8. The closest oil fields to the site are the Boyle Heights and Union Station Oil Fields which are located at an approximate distance of 1.0 and 1.5 miles south of the Campus, respectively. Therefore, the potential for subsidence is not considered a significant geologic hazard to the Project.

4.2.2 Oil Wells

The California Division of Oil, Gas and Geothermal Resources (DOGGR) Map 1 District 119 shows there is one (1) oil well located in the existing Visitor Parking Lot No. 5 as shown on Figure 8. This includes the Santa Fe Springs Mutual Oil Syndicate No. 1 well.

According to the Division of Oil, Gas and Geothermal Resources (DOGGR) map records, the well status is considered idle. No surficial evidence of a well was observed in Parking Lot No. 5 during our geotechnical investigation. There are no active or abandoned oil/gas wells shown within the footprint of the Project. The well locations shown are generally considered approximate. The likelihood of encountering an abandoned oil/gas well during construction is very low.

4.2.3 Flooding

The site is within an area designated by the Federal Emergency Management Agency (FEMA) as outside the 500-year floodplain (Zone X) as shown on the Flood Insurance Rate Map. According to the Los Angeles County Safety Element (1990), the site is not located within a 100-year or 500-year flood area. The potential for significant flooding is low.

4.2.4 Methane

The Project lies outside areas delineated by the City of Los Angeles as a "Methane Zone" or "Methane Buffer Zone", which are zones with known seepage and potential for seepage of methane gas to occur in buildings. Therefore, seepage of methane gas is considered a low potential hazard to the proposed project. However, a small southerly portion of the LAC+USC Medical Campus is located within the Methane and Methane Buffer Zone. The identified portion is consistent with the location of an idle oil and gas well.

4.2.5 Hydrocollapse

At their dry, natural state, soils with collapse potential possess stiffness and high apparent shear strength, but upon wetting, they could exhibit a significant decrease in volume. Such soils which exhibit this phenomenon at fairly low stress are considered collapsible soils. Collapsible soils are generally characterized by their loose structure of bulky shaped grains, often in the silt to fine sand size with small amount of clay. Since the site soils are either mostly clayey or consist dense sands, the potential for hydrocollapse is considered remote.

4.2.6 Expansive Soil

Expansive soils are fine-grained soils (clay) that can undergo a significant increase in volume with increase in water content and a significant decrease in volume with a decrease in water content. Changes in water content of an expansive soil can result in severe distress to structures constructed upon the soil. An expansion index (EI) tests were performed on a representative fill soil sample collected from our borings and the test indicated the fill soil material has a high expansion potential. Therefore, expansive soils are anticipated to pose a hazard to the proposed project.

4.3 Seismic Hazards

Seismic hazards are those hazards associated with earthquakes such as ground shaking, ground rupture, differential compaction or seismic settlement, and other phenomena. The subject Project site, like most sites in southern California, is most susceptible to ground shaking generated during earthquakes on nearby faults. The intensity of ground shaking, or strong ground motion, is highly dependent upon on the distance of the fault to the site, the magnitude of the earthquake, and the underlying soil conditions. These hazards and how they may affect the site are discussed in the following sections.

4.3.1 Ground Rupture

Surface fault rupture is ground deformation that occurs along the surface trace of a fault during an earthquake. The Alquist-Priolo Fault Zoning Act (CGS, 2018) regulates development near active faults so as to mitigate that hazard of surface fault rupture. The California Geological Survey (CGS) defines an active fault as one that has had surface displacement within Holocene time (about the last 11,000 years), and a sufficiently active fault as one that has evidenced of surface displacement along one or more of its segments or branches (CGS, 2018). The CGS considers a fault to be well defined if its trace is clearly detectable as a physical feature at or just below the ground surface.

There are no known active or potentially active faults with well-defined fault traces delineated by the CGS that have been recognized as crossing or projecting toward the project site (Bryant and Hart, 2007) as shown in Figure 9. The closest mapped earthquake fault (Raymond Hill) with surface rupture to the proposed project is located approximately 3.8 miles (6.0 kilometers) to the north. Because there are no known active faults on or projecting towards the project site, the potential for surface fault rupture at the site is considered low.

4.3.2 Ground Motion

Strong ground motion occurs as energy is released during an earthquake. The intensity of ground motion is dependent upon the distance between the site and the earthquake, the magnitude of the earthquake, and the geologic conditions underlying and surrounding the site. Earthquakes occurring on faults closest to the site would most likely generate the largest ground motions.

Majority of the Project site can be classified as Site Class D and seismic design can be performed in accordance with the criteria listed in Table 5 based on the mapped values of the current 2016 CBC and 2017 Los Angeles County Building Codes (CLABC).

However, minor portion of the Project is located within potential liquefiable zone by CGS and may be classified as Site Class F profile due to presence of potential liquefiable soils at the site. However, if the fundamental period for the future proposed structure is less than 0.5 seconds, the seismic design can be performed in accordance with the criteria listed in Table 5 based on Site Class D profile following the mapped values of the current CBC and CLABC.

TABLE 5 - SEISMIC DESIGN PARAMETERS

CHARACTERISTIC	CRITERIA
Site Class	D[1]
S_s - mapped spectral acceleration at short periods (g)	2.558[2]
S_1 - mapped spectral acceleration at 1-second period (g)	0.892[2]
F_a - site coefficient	1.0
F_v - site coefficient	1.5
S_{MS} - risk-targeted maximum considered earthquake (MCE_R) spectral acceleration at short periods (g)	2.558
S_{M1} - MCE_R spectral acceleration at 1-second period (g)	1.337
S_{DS} - design spectral acceleration at short periods (g)	1.705
S_{D1} - design spectral acceleration at 1-second period (g)	0.892
PGA – mapped maximum considered earthquake geometric mean (MCE_G) peak ground acceleration (g)	0.972[2]
Notes: 1. Site Class F within CGS liquefaction potential zone. 2. ASCE 7-10 mapped values for 2 percent probability exceedance in 50 years.	

4.3.3 Liquefaction and Seismically-Induced Settlement

Portion of the Project site is within the liquefaction zone assigned by the CGS. A site-specific liquefaction analysis was performed in general accordance with the guidelines presented in the California Geological Survey 117A (CGS, 2008), the recommendations provided in Seed et al (2003) and the Manual for Preparation of Geotechnical Reports (LACDPW, 2013). As discussed in Section 3.7, a design groundwater elevation of 300 feet MSL was used in the liquefaction analysis. The peak ground acceleration was based on the mapped maximum considered earthquake geometric mean (MCE_G) peak ground acceleration. The corresponding modal moment magnitude of 6.8 and modal distance of 10.0 kilometers using the USGS seismic deaggregation tool were considered. The input parameters were as follows:

Moment Magnitude, M_w:	6.8
Peak Ground Acceleration (PGA):	1.095 g
Design Groundwater Elevation:	300 feet MSL

The site-specific liquefaction analyses were performed assuming the design groundwater at historical high (i.e., 300 feet MSL). The evaluation indicates that potential for liquefaction at the Project site is moderate and the results are summarized in Table 6. However, the fine-grained silty soils encountered at the Project are judged unlikely to undergo severe strength loss during ground shaking.

For soils above the groundwater table, dry sand settlement could occur from the sandy soils due to ground shaking. Liquefaction and dry sand settlement evaluation were performed using the relevant boring data and laboratory testing.

Based on the results of the dry sand and liquefaction-induced settlement analyses, the total seismic-induced settlement is estimated to be up to approximately 0.3 inches and therefore, the impact to the foundations due to design earthquake is considered low. It is also judged that surface expression at the foundation is unlikely, due to the interbedded and isolated nature of potential liquefiable materials encountered at the site.

TABLE 6 - LIQUEFACTION POTENTIAL AND SEISMIC SETTLEMENT

BORING	DEPTH OF POTENTIALLY LIQUEFIABLE LAYER (feet)	POTENTIALLY LIQUEFIABLE LAYER ELEVATION[1] (feet MSL)	TOTAL THICKNESS OF POTENTIALLY LIQUEFIABLE LAYER[1,2] (feet)	TOTAL SEISMICALLY-INDUCED SETTLEMENT[3] (inches)
B-19-05	30 to 40	270 to 280	10	0.3

Notes:
1. Design groundwater at Elevation +300 feet MSL.
2. Settlement at level existing ground surface.
3. Seismically-induced settlement based on material with factor of safety less than 1.0.

The detailed calculations for dry sand settlement and liquefaction-induced settlement are presented in Appendix E.

4.3.4 Lateral Spread Displacement

According to publications by Youd et al., 2002, conditions such as free-face, sloping ground surface and liquefiable layers are factors contributing to lateral spread displacement of the ground during strong motion events. As the liquefaction potential is low at the project site; therefore, risk of lateral spread displacement is also low.

4.3.5 Earthquake-Induced Flooding

Earthquake induced flooding occurs when nearby water retaining structures, such as dams or storage tanks are breached or damaged during an earthquake. The Ascot Reservoir has been identified by the Los Angeles County Safety Element (1990) as being located approximately 1.5 miles (2.4 kilometers) to the northeast of the Campus. The Los Angeles County Safety Element identified the Project Site to be within a "dam or debris basin inundation area". The Los Angeles County Safety Element has identified the site as being isolated in a "non-inundation area". The risk of seismic-induced flooding is not considered a significant hazard.

4.3.6 Other Seismic Hazards

Other seismic hazards include tsunamis, seiches and earthquake-induced landslides. The potential for tsunamis, seiches and earthquake-induced landslides at the proposed site is low due to the distance from the Pacific Ocean to the campus; absence of oceans, lakes, or large bodies of water in the immediate area; and relatively flat topography of the site.

Section 5 Discussion and Recommendations

5.1 General

The subsurface information and geotechnical recommendations presented in this report are intended to provide general geotechnical information for the proposed demolition of the structures within the Project limit. Based on the results of our geotechnical investigation and the anticipated loading conditions, we believe that the proposed Project is feasible from a geotechnical point of view, provided the recommendations in this report are incorporated in the design and implemented during construction of the Project.

In general, the existing structures will be demolished, and the concrete debris may be crushed into aggregate materials for reuse. Due the presence of buried basement and foundation elements, major sloped excavation or shoring support are anticipated. From geotechnical standpoint, the existing piles and pile caps may remain in place but are recommended to be removed and the piles to be cut off at the top once the foundations/pile caps are removed. Significant grading and drainage modification are anticipated in order to create the interim pad for the proposed surface parking lot. However, since the proposed grading plan is not available during the subject report preparation, generic geotechnical recommendations are provided to capture the anticipated construction activities.

5.2 Earthwork

Earthwork should be performed in accordance with the applicable portion of the State and local building code, as well as the recommendations of this report, and should be performed under the observation and testing of a representative of the Geotechnical Engineer of Record (GEOR).

It is the responsibility of the contractor to select the proper equipment necessary for site excavation and backfilling. All excavations should be observed by the GEOR. Some variations in subsurface conditions are anticipated. However, if conditions encountered during construction appear to differ from those encountered in the exploratory borings, AECOM should be notified so as to consider the need for modifications.

5.2.1 Site Preparation

It is the responsibility of the contractor to notify and coordinate with USA before any proposed earthwork. All active or inactive utilities and tunnels within the construction limits should be identified for relocation, abandonment, or protection prior to grading. Any pipes greater than 2 inches in diameter to be abandoned in-place and should be filled with lean-mix concrete after review of their location and approval by the GEOR.

Where excavations are proposed, all existing debris, organic materials, deleterious materials, undocumented fill and loose soils should be removed and disposed of outside the construction limits under observation of the GEOR.

5.2.2 Structure Foundation Demolition

Due to the difficulty of removing the foundation and pile caps, the project may choose to leave the pile cap in place. Any structural element that is not removed during the demolition should be surveyed prior to subsequent grading operation.

In general, it is recommended all the foundation elements supporting the existing structures to be demolished should be removed, with an exception of the piles, which should be left in place. Depending on the proposed topography for the interim grading, the existing piles are recommended to be cut off to

ensure the remaining pile head to be at least 5 feet below the proposed interim grade. Once the piles are cut off, each pile needs to be surveyed prior to subsequent grading operation.

5.2.3 Over-excavations

By nature of the subject Project, over-excavation is not anticipated for the general grading, but may be required for the proposed interim site civil improvements.

For any proposed retaining walls to retain soil less than 5 feet high, we recommend that the upper 3 feet of foundation soil below the base of the retaining wall footings be overexcavated and replaced with compacted/engineered fill, if the walls are located at the current grade or deeper. For the retaining wall greater than 5 feet but less than 8 feet high, overexcavation should be 5 feet below the base of the retaining wall footings. For the proposed wall which to be completely within the backfill/fill for interim grading, no over-excavation is required.

The area of overexcavation should extend at least 5 feet beyond the edge of foundations, or equal to the depth of removal, whichever is greater. For taller retaining walls, locations should be identified, and more specific recommendations can be provided where applicable.

For concrete flatwork/hardscape, overexcavation can be further reduced to 2 feet with similar procedure except the shallower over-excavation depth, if the proposed improvements are located below the existing grade.

Following demolition and/or over-excavations, the upper 8 inches of the exposed subgrade to receive compacted/ engineered fill should be scarified and proof-rolled in place to at least 90 percent of the maximum dry density to secure uniform support. Probing and field density tests should be performed by a field representative of the GEOR to evaluate the extent of any loose or soft zones.

5.2.4 Fill and Backfills

Engineered fill and backfill should be compacted by:

- Placing in loose lifts/layers less than 8 inches thick.
- Moisture conditioning to above-optimum moisture content.
- Compacting to at least 95 percent relative compaction in accordance with ASTM D1557 standard.

No compaction by ponding or jetting should be allowed. If specified moisture content (above optimum) and relative compaction are not achieved, additional compaction effort, moisture conditioning of the fill soils, and/or removal and re-compaction of the below-minimum-compaction soils will be required at the expense of the contractor.

No fill should be placed, spread or rolled during unfavorable weather. When the work is interrupted by rain, operations should not be resumed until field tests by the GEOR have indicated that conditions will permit satisfactory results.

Import material for engineered filled and select backfill, if required, should meet the criteria in Table 7. Select Backfill is material placed within a horizontal distance of 5 feet or one-half of the wall height, whichever is greater, behind retaining walls. In addition, aggregate base and trench bedding materials should conform to the Greenbook or similar standards. All new fills should be free of hazardous, organic and inorganic debris. All fill and backfill materials should be observed and tested by the GEOR in order to determine their suitability.

The onsite clayey soils encountered during field investigations are not expected to meet the criteria for reuse as engineered fill. It may not be feasible to mix onsite soils generated from excavations due to space restriction for laydown/working area.

Site grading may be accomplished with conventional heavy-duty construction equipment. The fill can be compacted using rubber-tired or vibratory soil compactors as defined by the Caterpillar Performance Handbook (2017) or equivalent. However, to avoid overstressing any existing retaining/basement walls, either the backfill should be compacted using lightweight compaction equipment or the walls should be properly braced.

The concrete debris generated from the building demolition can be crushed and furnished into aggregate base material following the specifications in Table 7 per Greenbook and Table 8 per Caltrans Class 2. The aggregate base quality character should follow Table 10.

TABLE 7 - SELECT BACKFILL CRITERIA

CRITERIA	IMPORT FILL	SELECT BACKFILL
Maximum particle size (inches)	3	1
Maximum liquid limit (%)	10	5
Maximum plasticity index (%)	5	0
Minimum percentage passing the No. 4 sieve (%)	80	80
Maximum percentage passing the No. 200 sieve (%)	20	10
Minimum sand equivalent	20	20
Maximum Expansion Index	30	30
R-value	50	50
Remolded Internal Friction Angle (degrees)	34	34

TABLE 8 - GREENBOOK AGGREGATE BASE GRADATION CRITERIA

SIEVE SIZES	PERCENTAGE PASSING	
	¾ INCH MAXIMUM	
	OPERATING RANGE	CONTRACT COMPLIANCE
2"	100	
1-1/2"	85-100	100
1"	50-85	85-100
¾"		55-75
No. 4	25-45	35-60
No. 30	10-25	10-30
No. 200	2-9	2-9
ASTM C 131 Test Grading	A	B

TABLE 9 - CALTRANS CLASS 2 AGGREGATE BASE GRADATION CRITERIA

SIEVE SIZES	PERCENTAGE PASSING			
	1-1/2 INCH MAXIMUM		¾ INCH MAXIMUM	
	OPERATING RANGE	CONTRACT COMPLIANCE	OPERATING RANGE	CONTRACT COMPLIANCE
.2"	100	100	--	--
1-1/2"	90–100	87–100	--	--
1"	--	--	100	100
3/4"	50–85	45–90	90–100	87–100
No. 4	25–45	20–50	35–60	30–65
No. 30	10–25	6–29	10–30	5–35
No. 200	2–9	0–12	2–9	0–12

TABLE 10 - AGGREGATE BASE QUALITY CHARACTERISTICS

QUALITY CHARACTERISTIC	GREENBOOK REQUIREMENT		CALTRANS REQUIREMENT	
	TEST METHOD NO.	REQUIREMENT	OPERATING RANGE	CONTRACT COMPLIANCE
Resistance (R-value, min)	California 301	78 minimum	--	78
Sand equivalent (min)	California 217	35	25	22
Durability index (min) /Percentage Wear 100 revolutions 500 revolutions	ASTM C 131	 15 maximum 52 maximum	--	35

5.2.5 Grading Factor

If the onsite soil is reused for site civil improvements, a grading factor of 0.90 to 1.00 (i.e., the factor is less than unity, indicating shrinkage) may be considered to estimate the net quantity of the soils to be reused during the earthwork, assuming minimum relative compaction of 95 percent per ASTM D1557.

For furnished aggregate materials using the concrete debris from the proposed demolition, grading factors of 1.3 to 1.4 may be considered for reused and grading factors of 1.5 to 1.6 may be considered for export, respectively.

5.2.6 Temporary Excavation

Excavation approximately up to 20 feet deep may be required for portions of the existing building to reach the depth of the exiting pile caps. Excavations during construction should comply with the current California and Federal OSHA requirements. For design purposes, a Cal/OSHA soil type B can be assumed for the onsite soils. It should be noted that this assessment of Cal/OSHA soil type for temporary excavations is based on engineering classifications of the subsurface materials encountered in widely spaced exploratory borings. The Contractor should have a geotechnical professional evaluate the soil/rock conditions encountered during excavation to determine permissible temporary slope inclinations.

Based on Cal/OSHA requirements, unsupported slopes for temporary excavation in the existing undocumented fill may be vertical for cuts of less than 4 feet in depth. For deeper cuts, up to a maximum height of 20 feet, temporary excavation slopes should be made no steeper than 1H:1V (horizontal to vertical).

The slope stability analysis indicated the proposed temporary excavation scheme can be maintained to satisfy a minimum FS of 1.25 under static condition. The result is presented in Table 11 and details of the analysis are provided in Appendix F.

TABLE 11 - SLOPE STABILITY ANALYSES

ANALYSIS	VERTICAL HEIGHT (feet)	BACK SLOPE	REQUIRED FACTOR OF SAFETY (FS)	FACTOR OF SAFETY (FS)
10-foot Excavation	4	1:1 (H:V)	1.25	1.94
20-foot Excavation	4	1:1 (H:V)	1.25	1.29
Interim Static	0	1.5:1 (H:V)	1.5	2.03
Interim Pseudostatic	0	1.5:1 (H:V)	1.1	1.17
Notes: • Unsupported Condition with a surcharge of 240 psf behind the sloped excavation				

In areas where soils with little or no cohesion are encountered, shoring or flatter excavation slopes may be necessary. Overexcavation should not be performed below an imaginary plane projected at 1H:1V below horizontal from any existing building foundations. If temporary excavation is deeper than 20 feet, or there is space limitation to prohibit sloped excavation, braced or cantilevered shoring should be considered in accordance with Section 5.2.7. It should be noted that the contractor and its engineering professional are responsible for detailed shoring design, which requires final review and approval by the GEOR.

Surcharge loads from vehicles and stockpiled material should be kept away from the top of temporary excavations at a distance equal to at least one half of the excavation depth. During wet weather, runoff water should be prevented from entering the excavation, and collected and disposed of outside the construction limits. To prevent runoff from adjacent areas from entering the excavation, a perimeter berm should be constructed at the top of the slope.

5.2.7 Temporary Shoring

Where stability or lack of available set-back will not allow adequate sloping of temporary excavations, shoring may be required. If soldier beam/legging system is used, maximum horizontal spacing between soldier piles should be limited to about 8 feet. Treated timber lagging may be required in sandy zones. Any void space between the back of the lagging and excavation face should be filled with lean concrete with provisions for weep holes to reduce the potential for buildup of hydrostatic pressure.

The temporary shoring system should be designed to resist lateral earth pressures plus additional horizontal pressures imposed by foundations of adjacent structures and/or vehicle traffic load. Temporary tied-back or braced excavations should be designed to resist an earth pressure in accordance with Figure 11.

If tie-back anchor is proposed by the shoring design professional, the following criteria should be considered:

- Anchors should be at least 5 times pile diameter from the adjacent pile, if any.
- Top lateral deflection of the shoring system should be no greater than ¼-inch. Vertical movement should be controlled within ¼-inch. Rotation of the shoring wall should not be greater than 0.1 percent.
- Instrumentation program utilizing baseline survey and periodical monitoring at developed monuments should be included in the shoring design.
- Construction sequence should clearly be identified in the shoring design.
- If pressure grouted anchors are used, real-time monitoring program should be included as part of pressure grout operation for grouting pressure adjustment in order to prevent hydro fracturing or any adverse impact to the exiting adjacent foundation elements.
- Shoring system abandonment after the construction is completed should be included in the shoring design package. The anchors need to be de-tensioned after abandonment.

5.3 Interim Slopes

During the site development, interim fill and cut slopes can be constructed at no steeper than 1.5:1 (H:V) slope gradient after completion of proposed demolition. The estimated factors of safety as indicated in Table 11 meet the minimum code requirement of 1.25. Details of the analysis are provided in Appendix F.

5.4 Permanent Slopes

Permanent compacted fill slopes, if any, should be planned no steeper than 2:1 (H:V). The slopes should be planted and/or protected to reduce surface erosion.

5.5 Retaining Structure

If retaining walls are considered for the interim grading to provide necessary grade separation, the following recommendations should be followed.

5.5.1 Bearing Capacity

Spread footings should have a minimum dimension of 2 feet wide and a minimum embedment depth of 2 feet below the lowest adjacent grade founded over a minimum of 3 feet of the aforementioned engineered fill. The engineered fill should also extend at least 5 feet and no less than the thickness of the engineered fill beyond the edge of the footing. Footings resting on this fill and with the above minimum dimensions may be designed using an allowable bearing pressure of 2,000 psf. The recommended bearing pressure can be increased by 25 percent with each additional foot of width or depth, to a maximum value of 2,500 psf. The bearing pressure applies to dead plus live loads and includes a calculated factor of safety of at least 3. The allowable bearing pressure values may be increased by one-third for short-term loading due to wind or seismic forces. The edge pressure of any eccentrically loaded footing should not exceed the allowable bearing value.

5.5.2 Settlement

Based on the recommended bearing pressure, static settlement of the proposed minor spread footing is anticipated to be approximately 1-inch. Differential settlements between similarly loaded adjacent footings may be assumed to be 1-inch over a distance of 50 feet.

5.5.3 Lateral Resistance

Resistance to lateral loads may be provided by frictional resistance between the bottom of concrete footings and the underlying soils or by passive soil pressure against the sides of the footings. The coefficient of friction between poured-in-place concrete footings and the underlying engineered fill, and passive pressure against the footing front surface are provided in Figure 12.

5.6 Concrete Flatwork

Concrete flatwork/hardscape should be supported on compacted/engineered fill placed in accordance with Section 5.2.4. Due to presence of expansive soil, concrete flatwork should be underlain by 12-inches of compacted granular materials containing less than 5 percent fines (passing the No. 200 sieve). Construction joint should be provided at no more than 10 feet spacing at either direction. The structural engineer's requirements should be followed if they are more stringent. For design of concrete flatwork, and for estimating their deflections, k of 150 pci may be used.

5.7 Site Drainage

Site drainage of surface storm water should be provided for during and after construction. The grading plans should account for any alterations in existing surface drainage. During construction, provisions should be made to divert surface water away from open excavations. Any water that collects in open excavations should be pumped out to avoid softening of foundation soils.

It is recommended that the final ground surface around the proposed structures be graded to a slope of at least 2 percent for minimum 10 feet to provide positive drainage away from the structures and existing retaining wall. In cases where this is not feasible, adequate measures should be provided to collect and drain storm water around the structures.

5.8 Low Impact Development Mitigation Measure

The design infiltration rates and the associated reduction factors are as shown in Table 12. Based on the field infiltration tests and geotechnical investigation performed, the site is judged not feasible for infiltration for the storm water improvement/best management practices (BMP) design due to a low infiltration rate. In addition, due to presence of the existing underground fuel tanks located immediately north of the proposed improvement, as well as the undocumented fills toward the north, infiltration based measure should be avoided. Instead, other measure such as capture and reuse, should be considered for LID design.

TABLE 12 - DESIGN INFILTRATION RATE

APPROXIMATE LOCATION	DEPTH/ ELEVATION (feet)/ (feet MSL)	MEASURED PERCOLATION RATE[1] (inches per hour)	TOTAL REDCUTION FACTOR[1]	DESIGN INFILTRATION RATE (inches per hour)
Front of WC	6 - 10	2.72	2 x 3 x 3	Not feasible
Adjacent to Trailer 421	11-16	2.64	2 x 3 x 3	Not feasible

Notes:
1. Total reduction factor (RF) based on RFs of 3. The factor should be confirmed by Project Civil Engineer. If other RFs is considered, RF should be adjusted, and design infiltration rate should be revised accordingly.
2. Target layer is not considered feasible for infiltration.

5.9 Pavement Section Design

Recommended minimum asphalt concrete (AC) and Portland cement concrete (PCC) pavement sections for temporary and permanent conditions are presented on Figure 13. The recommended minimum pavement sections are based on the following:

- Minimum R-value of 10 for native material and 50 for import soil or crushed AB.
- Caltrans design method.
- Design life span of 20 years.
- Traffic Index (TI) of 5 for parking area, 7 for minor entry/exit lanes, and 9 for major entry/exit lanes, and fire truck/emergency vehicles/delivery truck, respectively.

Generally, rigid PCC pavement costs more for initial construction but requires less maintenance than that for flexible AC pavements. For heavy-wheel loads along limited alignments, turning areas, dolly pads, bus stops, and pickup areas, PCC pavement should be used. For PCC pavements, the following should be considered:

- Construct pavements in a 15-foot square grid or smaller.
- Expansion joints should extend the full depth of the PCC pavement.
- Potential joints depth of ¼ of the PCC pavement thickness.

- Cure for a minimum of 7 days.
- No traffic until the compressive strength exceeds 2,000 pounds per square inch (psi).
- Minimum compressive strength of 3,000 (psi).
- Dowels to strengthen joints.
- Minimum slope of 1 percent.

The minimum thickness of compacted subgrade is outlined on Figure 13. The subgrade soils should be firm, unyielding and not "pumping." Aggregate base requirements and specifications are outlined on Figure 13. The basement soil (subgrade) and aggregate base should be compacted to at least 95 percent relative compaction as shown on Figure 13. If the basement soil cannot be compacted to at least 95 percent relative compaction, the subgrade should be overexcavated similar to the recommendations in Section 5.2.

5.10 Utilities

Utility trenches (either open or backfilled) that parallel structures, pavement, or flatwork should be planned so that they do not extend below a plane with a downward slope of 1H:1V from the bottom edge of footings, pavement, or flatwork. Otherwise, temporary shoring to provide footing, pavement, flatwork, or utility support is recommended unless localized settlements on the order of 1 percent of the trench depth can be tolerated. All excavations should comply with appropriate safety standards outlined in Section 5.2.6.

Utility pipes should be placed on the bottom of a neatly cut trench on a layer of bedding according to the manufacturer's recommendations. Jetting should not be allowed for compaction purposes. We anticipate that the near-surface soils will not be suitable for use as bedding materials.

For pipeline design purposes, the vertical pressure on a pipe can be estimated based on Figure 14. A modulus of soil reaction value of 500 pounds per square inch (psi) may be used for design. The allowable design soil bearing value for associated lightly loaded structures such as manholes is 500 psf for footing with a minimum embedment and width of 1 foot.

5.11 Corrosivity

Selected tests were conducted on representative soil samples to evaluate if special precautions or considerations should be made with respect to materials in contact with soils. Specifically, the soil sample was tested to assess corrosivity parameters, which include pH, resistivity, sulfate and chloride content. The test results are summarized in Table 13.

TABLE 13 - CORROSION POTENTIAL

Boring Location and Depth	Material	pH	Minimum Resistivity (Ohm-cm)	Sulfate Content (ppm)	Chloride Content (mg/L)
B-19-04 @ 0-5'	Lean Clay	8.8	2025	56	37
B-19-09 @ 15'	Lean Clay	7.7	981	248	44

The test result for pH, which indicates the degree of alkalinity or acidity of an on-site soil, of the bulk sample is between 7.7 and 8.8.

A commonly accepted correlation between electrical resistivity and corrosivity potential for ferrous metals is as the following:

Below 1,000 ohm-cm	-	Severely corrosive
1,000 to 2,000 ohm-cm	-	Corrosive
2,000 to 10,000 ohm-cm	-	Moderately corrosive
Over 10,000 ohm-cm	-	Mildly corrosive

The minimum resistivity test result indicates that the surface soil may be corrosive to severely corrosive to metals.

The current 2016 CBC stipulates that corrosion potential to the foundation element is to be evaluated in accordance with the design procedure following American Concrete Institute (ACI) 318 (ACI, 2014). Based on the test results of corrosion potential, the site is classified as exposure class F0, S1, W0, and C1 using ACI 318 Table 19.3.1.1. Accordingly, the concrete mixture may be designed per ACI 318 Table 19.3.1.1. In addition, a minimum concrete cover of 3 inches should be maintained to provide reinforcement protection in accordance with ACI 318, Section 7.7.1.

5.12 County Building Code Section 111 Statement

Based on the findings of this geotechnical investigation, and provided that the recommendations of this report are followed, and the designs and construction are properly and adequately executed, it is our opinion that the proposed construction work within the project site will be safe from landslides, slippage, or settlement. Further, it is our opinion that the proposed construction will not adversely affect the stability of the adjacent properties, with the same provisions listed above.

Section 6 Construction Considerations

6.1 Excavation

The project site is primarily underlain by medium dense to very dense sand and stiff to hard clay or silt. Caving should be anticipated in sandy soil layers when performing other temporary excavations. Proper equipment should be selected for such subsurface materials.

6.2 Groundwater

Based on the findings of our field exploration and the available information on groundwater conditions, groundwater could be encountered during construction. Caving should be anticipated with the onsite sandy soil in combination with groundwater. Groundwater level can be as high as 20 feet bgs during demolition. Groundwater conditions may change due to various factors such as rainfalls in the area.

Section 7 Design Review

The geotechnical aspects of the Project should be reviewed by the GEOR during the design process. The scope of services may include:

- Assisting the design team in providing specific recommendations for special cases;
- Reviewing the design grading plan to ensure that all geotechnical recommendations contained in this report have been included in the design;
- Reviewing foundation design drawings;
- Reviewing the geotechnical portions of the Project for possible cost savings through alternative approaches;
- Reviewing the proposed construction techniques to evaluate if they satisfy the intent of the recommendations presented in this report.

Section 8 Construction Monitoring

As a necessary requisite to the proper use of this report, the following should be monitored by a qualified engineer/engineering geologist/technician under the supervision of the GEOR:

- Site preparation including site stripping, removal of subsurface structures, and bottom observation;
- Temporary excavation and overexcavation;
- All foundation excavations and subgrade preparation, if applicable;
- Placement of all engineered fills and backfills; and
- Installation of the shoring system, if applicable.

To capture any impact on adjacent properties due to the proposed construction activities, some means of monitoring the performance of the shoring system is recommended. If any sign of excessive deformation is observed, immediate action should be taken to correct potential issues occurred, before proceeding further excavation operation.

The GEOR should be notified at least two days in advance of the start of construction. A joint meeting between the contractor and the GEOR is recommended prior to the start of construction to discuss specific procedures and scheduling.

Section 9 References

1) American Concrete Institute, 2014, ACI 318-14, Building Code Requirements for Structure Concrete and Commentary.

2) American Society for Testing and Materials (ASTM), 1999. Annual Book of ASTM Standards, Section 4 - Construction, Volume 4.08, Soil and Rock (I): D 420 - D 4914, West Conshohocken, PA.

3) Youd T.L., et al., 2002, " Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement" (December, 2002). Journal of Geotechnical and Environmental Engineering. Volume 128 Issue 12.

4) Building News, 2009, "Greenbook," Standard Specifications for Public Works Construction.

5) California Building Code (CBC), 2016. Title 24, Volume 2, California Building Standards Commission.

6) California Division of Oil, Gas and Geothermal Resources (DOGGR), 2019, District 1, Map 119, http://www.conservation.ca.gov/dog/pages/index.aspx.

7) California Geological Survey (CGS), 1998. Seismic Hazard Zone Report 029. Seismic Hazard Zone Report for the Los Angeles 7.5-Minue Quadrangle, Los Angeles County, California.

8) CGS, 2001, Alquist-Priolo Earthquake Fault Zone (APEFZ) maps, Geographic Information System data files.

9) CGS, 2008, Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

10) CGS, 2018, Special Publication 42, Earthquake Fault Zones: A Guide for Government Agencies, Property Owners / Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California. Revised January 13, 2016.

11) Caterpillar Performance Handbook, 2017, Caterpillar, Inc., Edition 47.

12) County of Los Angeles, 2017, Title 26, County of Los Angeles Building Code.

13) Dames & Moore, 1954. Proposed Osteopathic Hospital. Soil Mechanics Engineer. February 1, 1954.

14) Dolan, J.F., et al., 2000, Paleoseismologic Evidence for an Early to Mid-Holocene Age of the Most Recent Surface Rupture on the Hollywood Fault, Los Angeles California, Bulletin of the Seismological Society of America, Vol. 90, No. 2, P. 334-344.

15) Field, E.H. et al., 2013. Uniform California earthquake rupture forecast, version 3 (UCERF3) The time-independent model: U.S. Geological Survey Open-File Report 2013–1165, 97 p., California Geological Survey Special Report 228, and Southern California Earthquake Center Publication 1792, http://pubs.usgs.gov/of/2013/1165/.

16) Geokon, 2018. Geotechnical Investigation Proposed Health Agency Recuperative Campus. Tract: 2082, Lot: LT1. Project No. A9733-06-01. April 9, 2018.

17) Lamar, D.L., 1970, Geology of the Elysian Park-Repetto Hills area, Los Angeles County, California: California Division of Mines and Geology, Special Report 101, scale 1:24,000.

18) Los Angeles County Department of Building and Safety, 2004, "Methane and Methane Buffer Zones", Prepared by GIS Mapping, Bureau of Engineering.

19) Los Angeles County Safety Element, 1990, Department of Regional Planning, County of Los Angeles, California, Flood and Inundation Hazards, Plate No. 6.

20) Mendenhall, W.C., 1905, Development of underground waters in the central coastal plain region of southern California: U.S. Geological Survey Water-Supply Paper 138

21) Miller, C.D., 1989, Potential hazards from future volcanic eruptions in California: U.S. Geological Survey Bulletin 1847, 17 p., 2 tables, plate 1, scale 1:500,000.

22) Ohta, Y. and N. Goto, 1976, Estimation of S-wave Velocity in Terms of Characteristic Indices of Soil, Butsuri-Tanko, Vol. 29, No. 4.

23) Oskin, M. et al., 2000, Active Parasitic Folds on the Elysian Park Anticline; Implications for Seismic Hazard in Central Los Angeles, California, Geological Society of America Bulletin, v. 112, p. 693-707.

24) Soper, E.K., 1943, Los Angeles City Oil Field in Geologic Formations and Economic Development of the Oil and Gas Fields of California; California State Division of Mines Bulletin, p. 282-283.

25) State of California Water Resources Control Board (SWRCB), 2019, GeoTracker.

26) United States Geological Survey (USGS), 1928, 7.5-Minute Topographic Map of the Los Angeles Quadrangle, Los Angeles County, California, Scale: 1:24,000, Meriam Library, California State University, Chico.

27) Yerkes, R.F. and Campbell, R.H., 2005, Preliminary Geologic Map of the Los Angeles 30' x 60' Quadrangle, Southern California, U.S. Geological Survey Open-File Report 2005-1019, Scale 1:100,000CGS, 2008, Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

28) Weaver, K. D., and Dolan, J. F., 2000, Paleoseismology and seismic hazards of the Raymond fault, Los Angeles County, California: Bulletin of the Seismological Society of America, v. 90, p. 1409-1428.

29) Wesnousky, S.G., 1986, Earthquakes, Quaternary Faults, and Seismic Hazards in California,

Section 10 Uncertainties and Limitations

AECOM has observed only a very small portion of the pertinent subsurface conditions. The recommendations presented herein are based on the assumption that the soil and geologic conditions do not deviate appreciably from those found during our investigation.

We recommend that AECOM review the design plans to verify that the intent of the recommendations presented herein has been properly interpreted and incorporated into the contract documents. We further recommend that geotechnically-relevant construction be observed by a qualified Geotechnical Engineer or Engineering Geologist to evaluate if the site conditions encountered are as anticipated, or to provide revised recommendations, if necessary. If variations or undesirable geotechnical conditions are encountered during construction, a Geotechnical Engineer should be consulted for further recommendations.

Geotechnical engineering and the geologic sciences are characterized by uncertainty. Professional judgments presented herein are based partly on our understanding of the proposed construction, and partly on our general experience. Our engineering work and judgments rendered meet current professional standards; we do not guarantee the performance of the Project in any respect.

- o0o -

It has been a pleasure to assist you with this Project. We look forward to being of further assistance as construction begins. Should you have any questions regarding this report, please contact us.

Very truly yours,



Dennis H. Nguyen, P.E.
Staff Geotechnical Engineer



Joseph P. Gratzer, P.G., C.E.G.
Senior Geologist

Reviewed by:

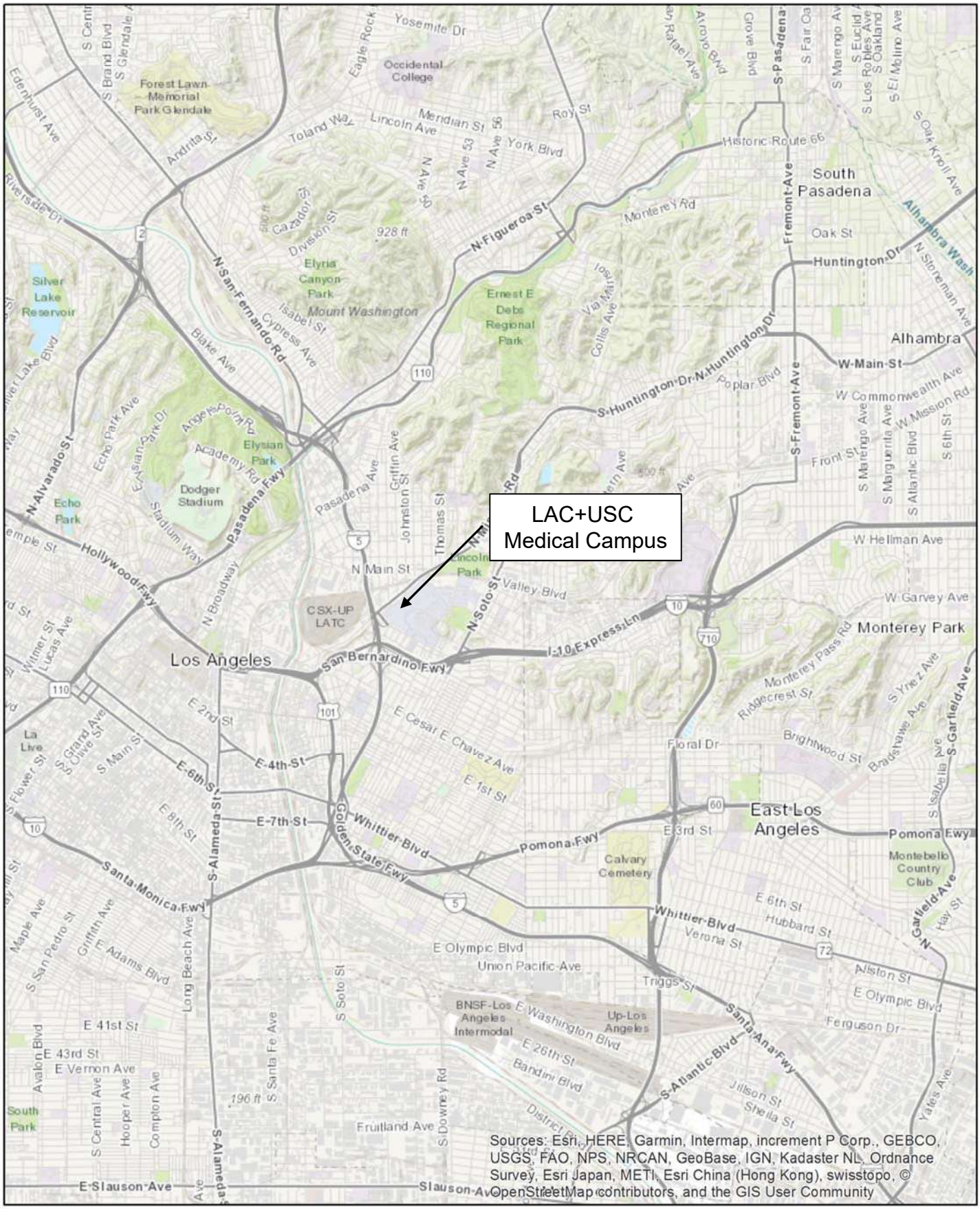


S. Neville Su, P.E., G.E.
Project Manager



Garry Lay, P.E., G.E.
Principal Engineer

FIGURES



N

0 0.5 1 2

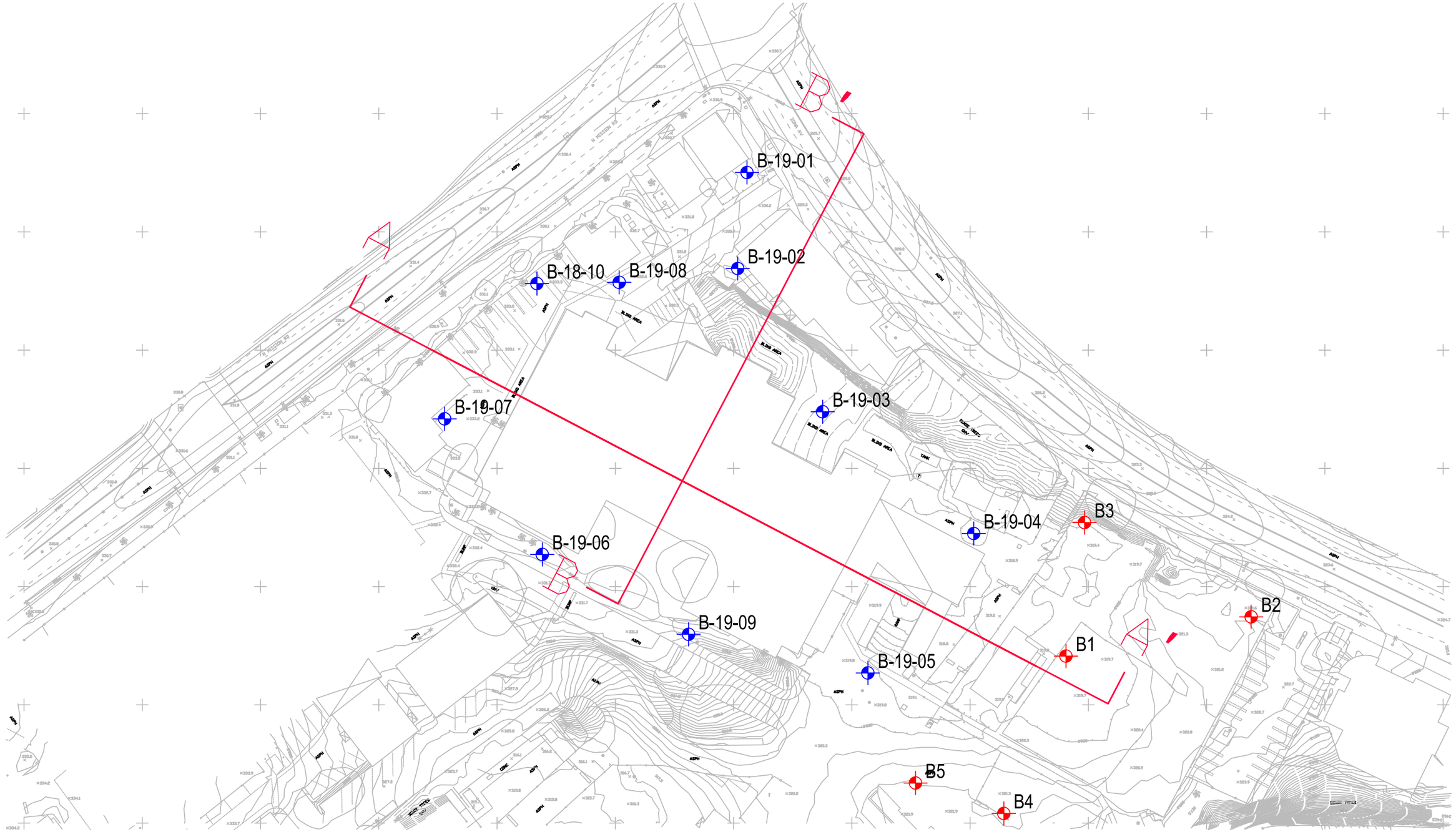
Miles

VICINITY MAP

LAC+USC WC Hospital Demolition

LAC+USC Medical Center

Los Angeles, California



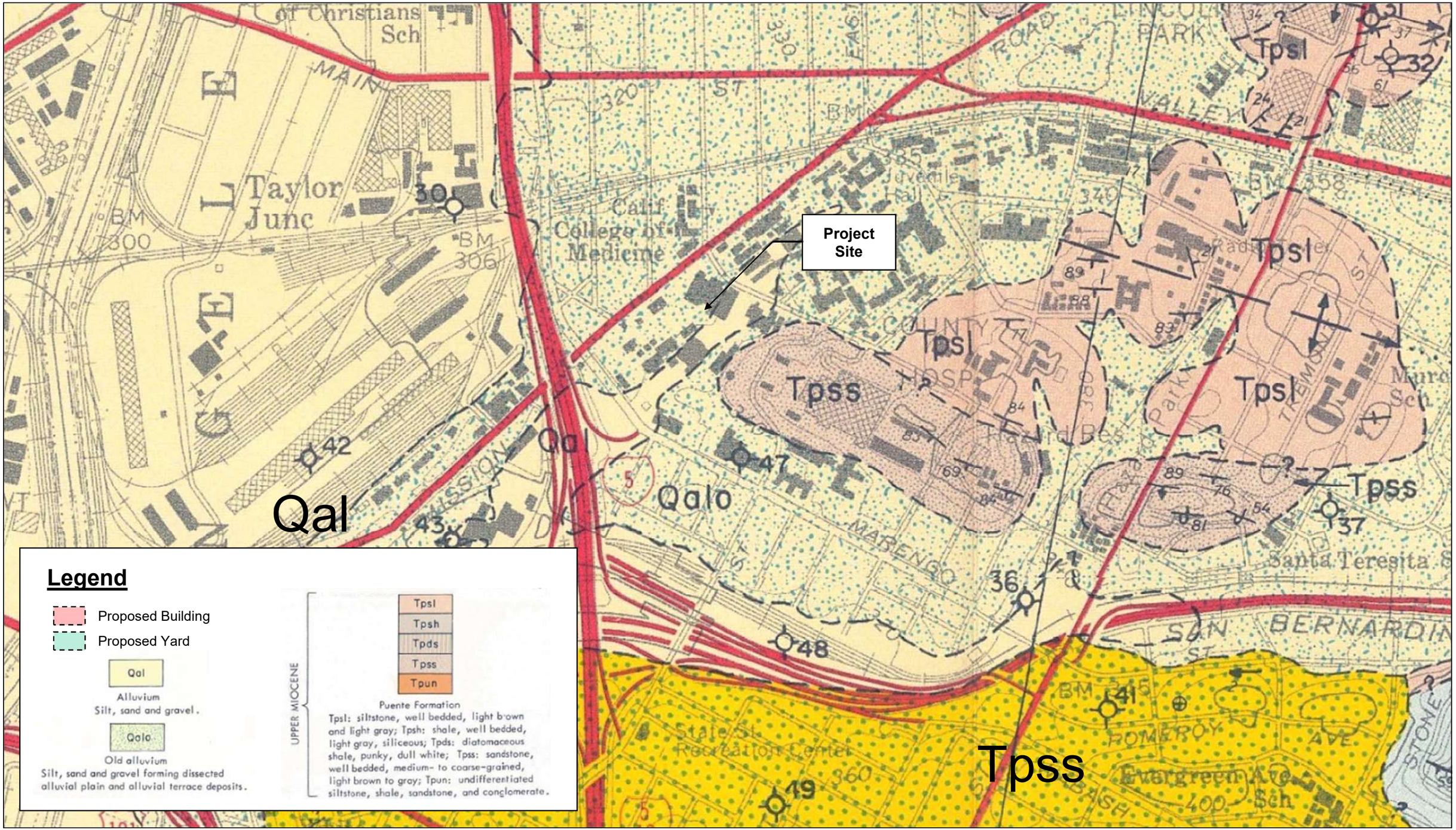
LEGEND

- AECOM 2019 Boring Location
- Geokon 2018 Boring Location

40’ 0’ 80’

SCALE: 1’’= 80’

PLOT PLAN
LAC+USC WC Hospital Demolition
LAC+USC Medical Center
Los Angeles, California



REGIONAL GEOLOGIC MAP
LAC+USC WC Hospital Demolition
LAC+USC Medical Center
Los Angeles, California

Legend

- Proposed Building
- Proposed Yard

Qal

Alluvium
Silt, sand and gravel.

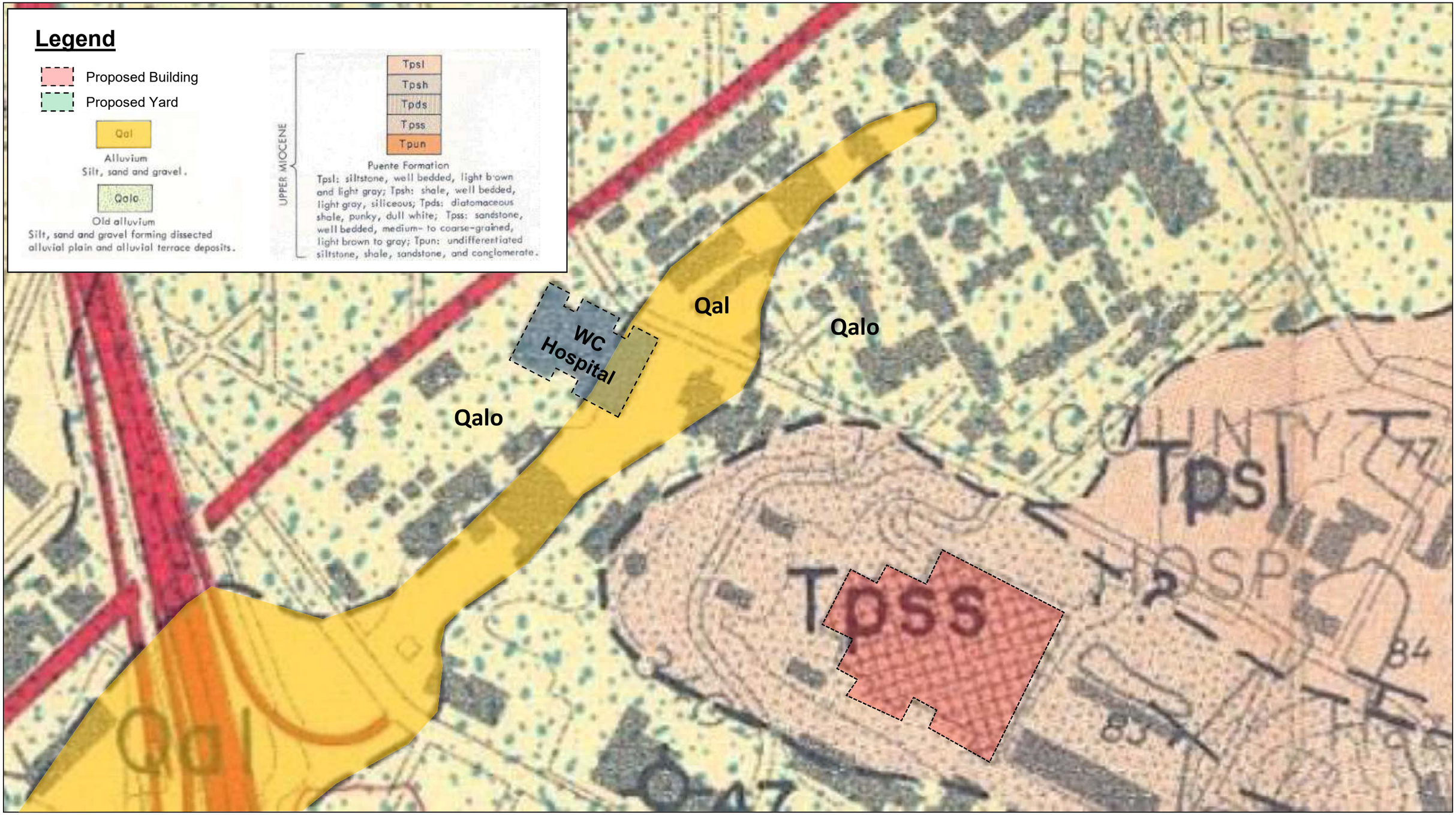
Qalo

Old alluvium
Silt, sand and gravel forming dissected alluvial plain and alluvial terrace deposits.

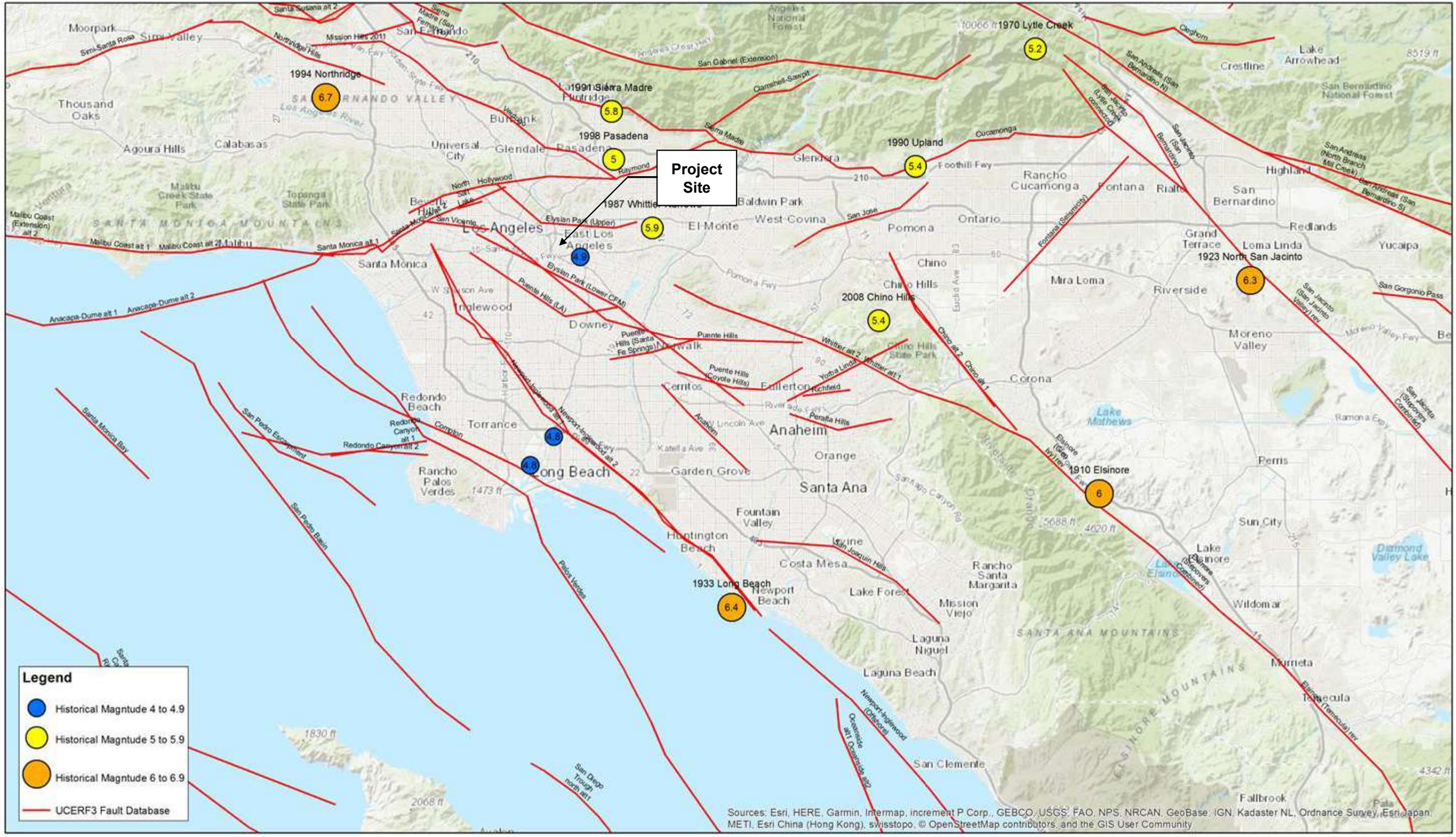
UPPER MIOCENE

Tpsl
Tpsh
Tpds
Tpss
Tpun

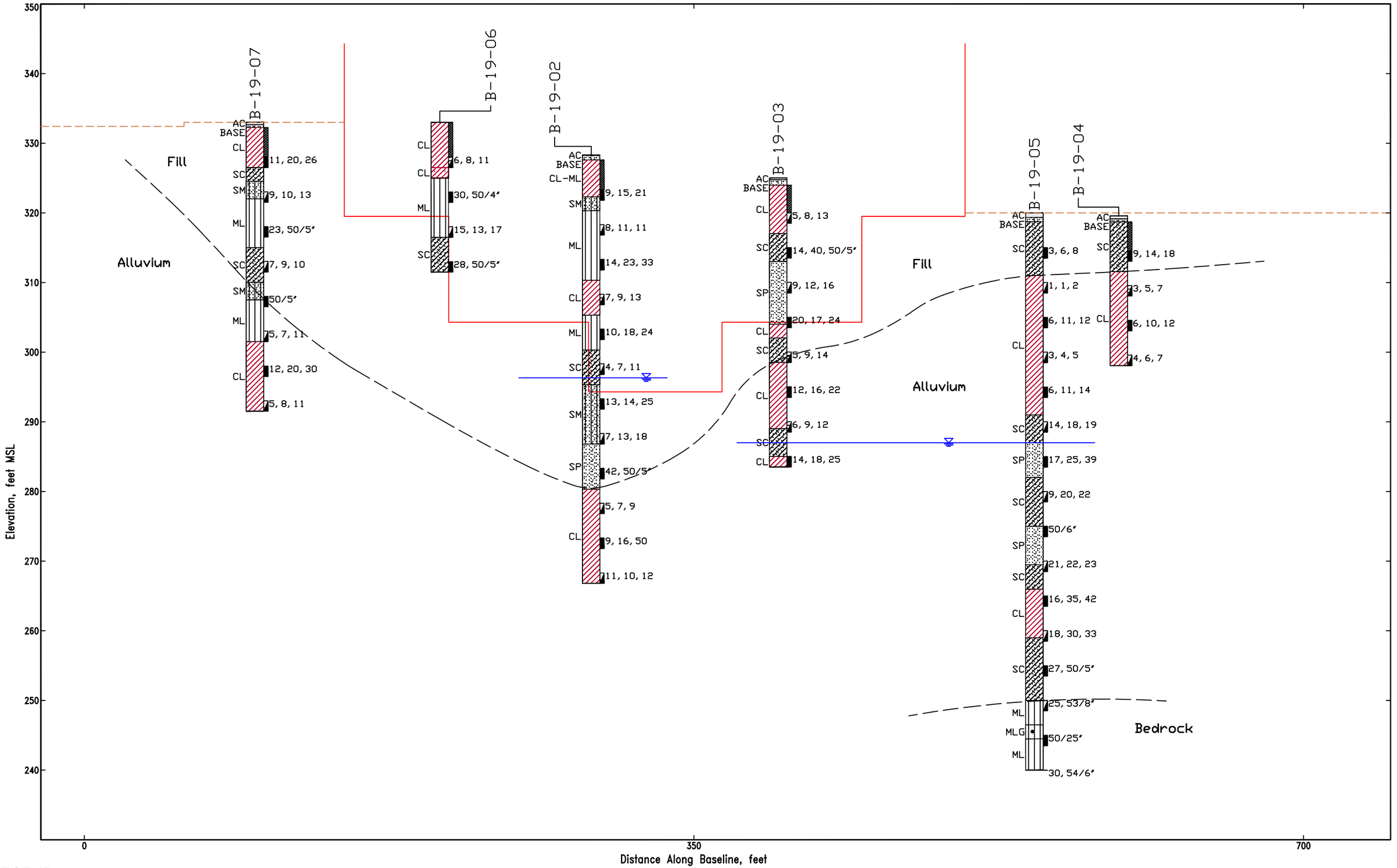
Puente Formation
Tpsl: siltstone, well bedded, light brown and light gray; Tpsh: shale, well bedded, light gray, siliceous; Tpds: diatomaceous shale, punky, dull white; Tpss: sandstone, well bedded, medium- to coarse-grained, light brown to gray; Tpun: undifferentiated siltstone, shale, sandstone, and conglomerate.



SITE GEOLOGIC MAP
LAC+USC WC Hospital Demolition
LAC+USC Medical Center
Los Angeles, California



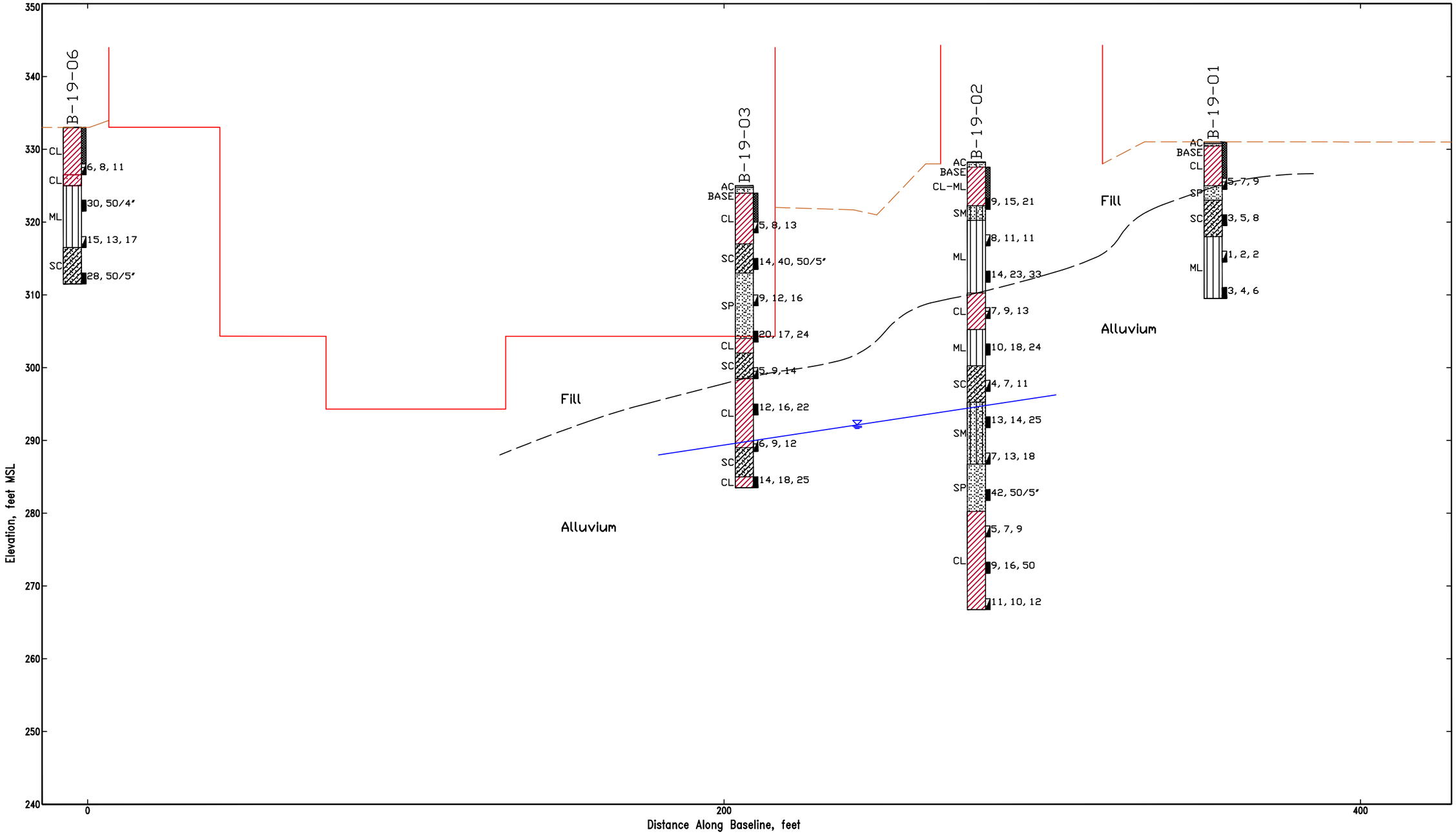
FAULT ZONE MAP
LAC+USC WC Hospital Demolition
LAC+USC Medical Center
Los Angeles, California



LEGEND

- Measured Ground Water Level
- Inferred Geology
- Approximate Building Footprint

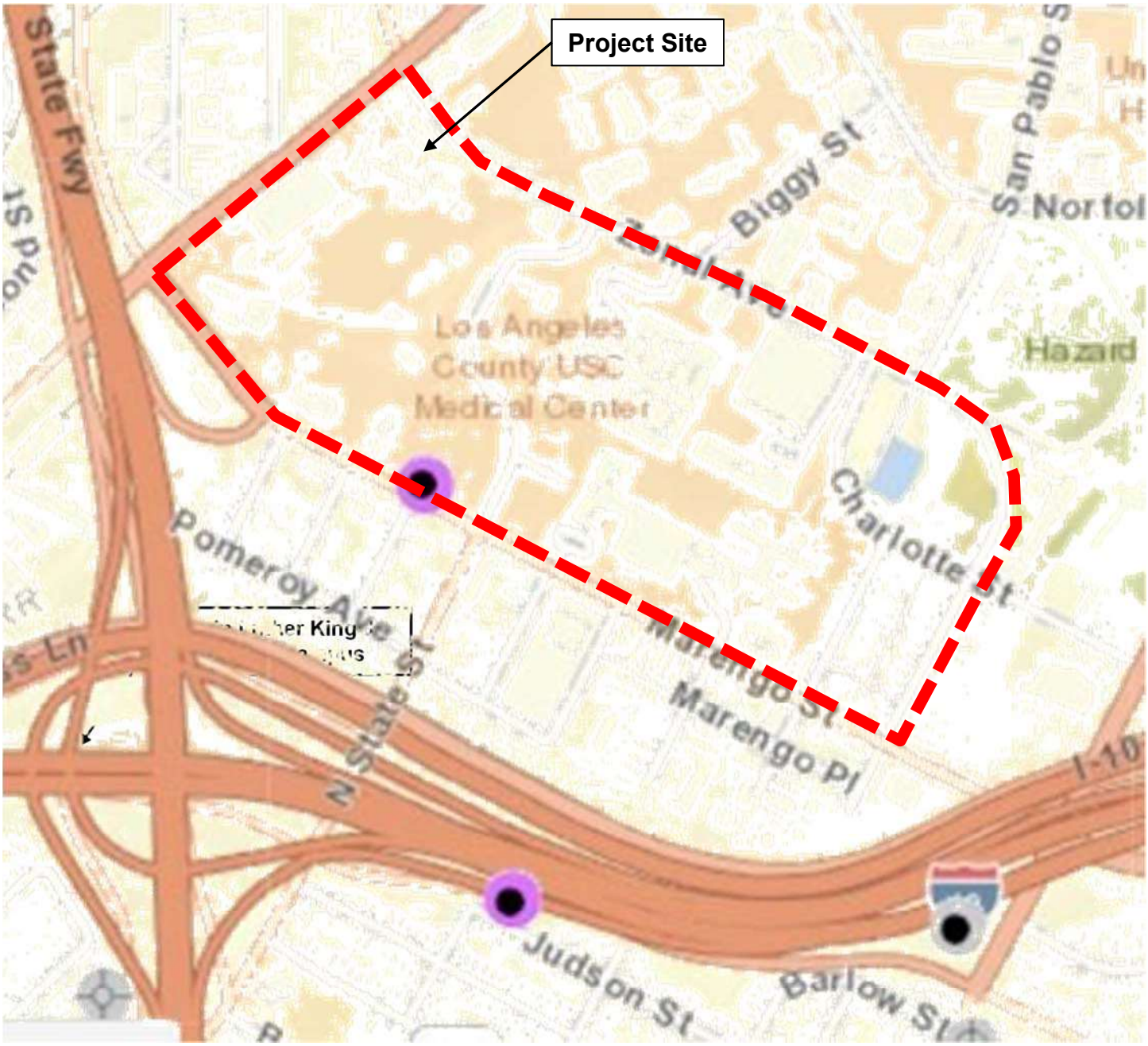
CROSS - SECTION A-A'
LAC+USC WC Hospital Demolition
LAC+USC Medical Center
Los Angeles, California



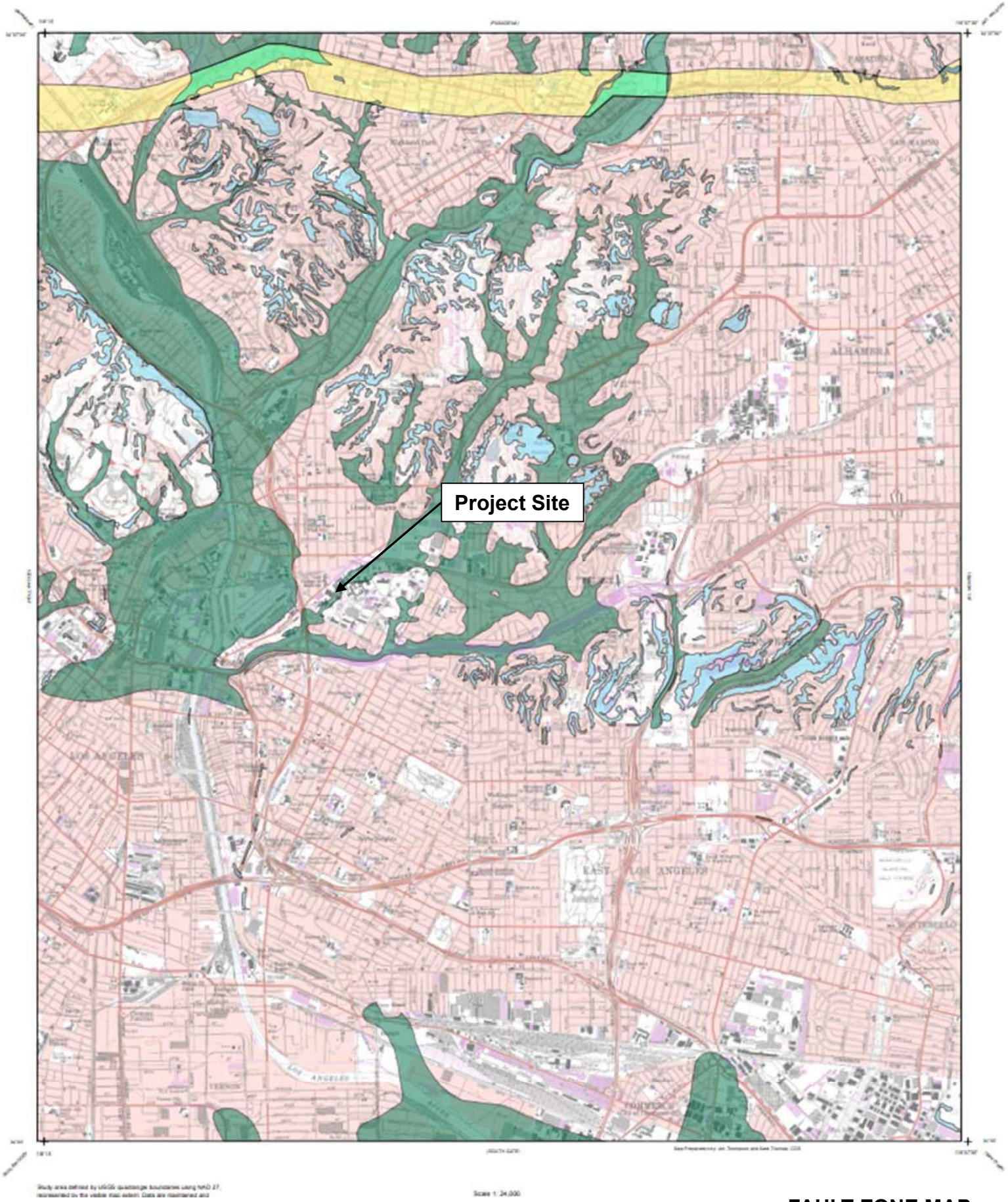
LEGEND

- Measured Ground Water Level
- — — Inferred Geology
- —— Approximate Building Footprint

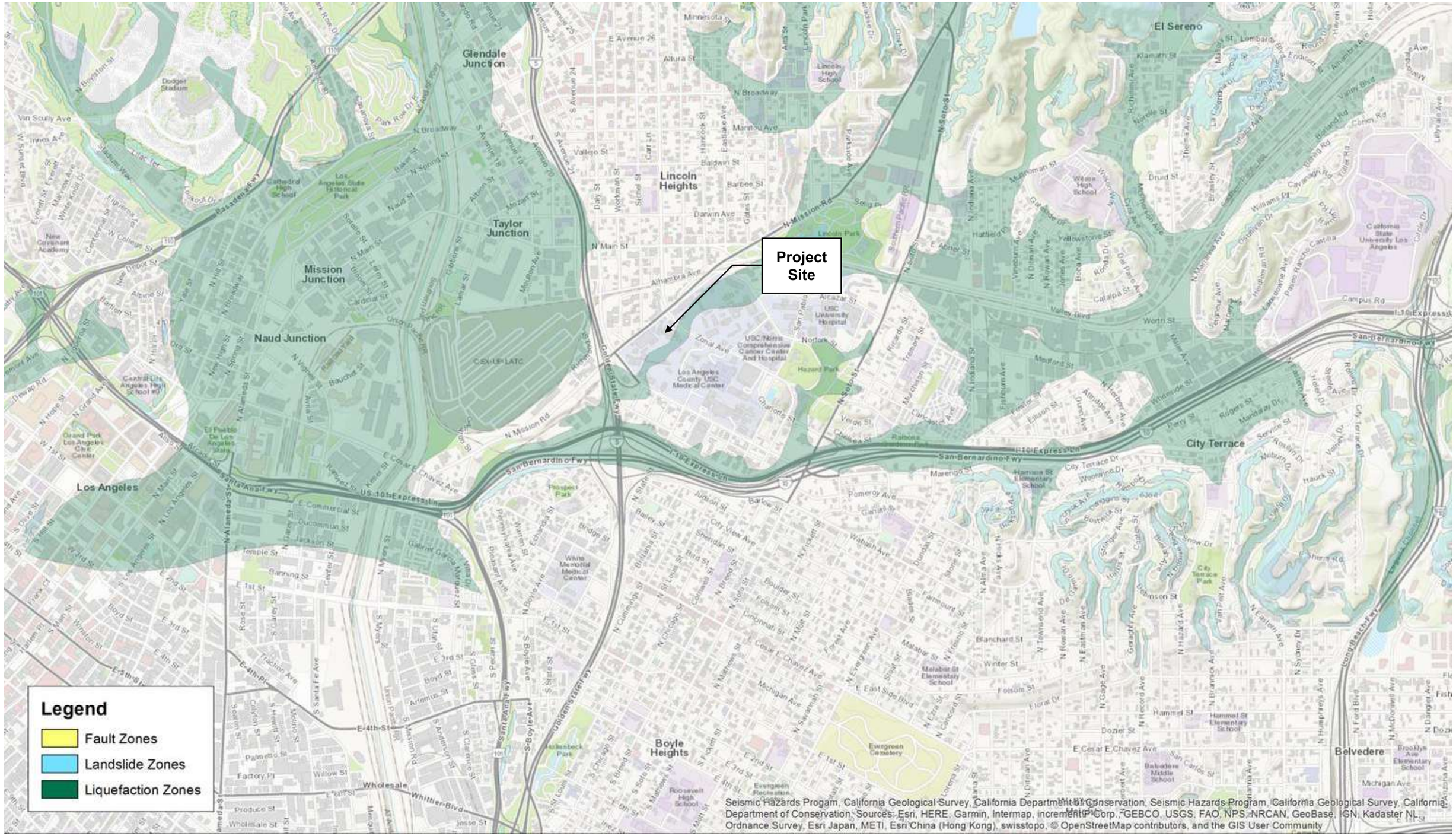
CROSS - SECTION B-B'
LAC+USC WC Hospital Demolition
LAC+USC Medical Center
Los Angeles, California



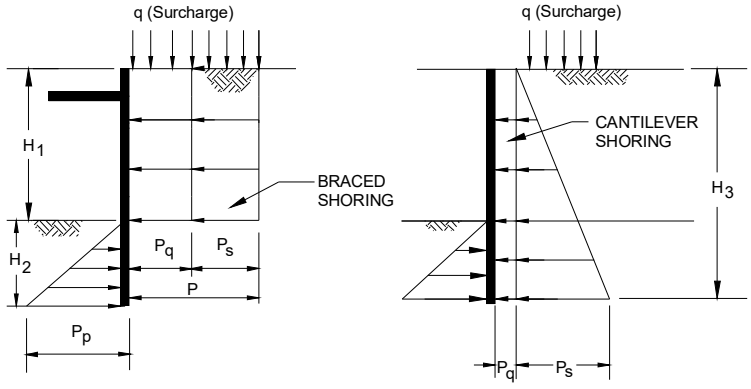
OIL, GAS, & GEOTHERMAL – DISTRICT 1 MAP 119
LAC+USC WC Hospital Demolition
LAC+USC Medical Center
Los Angeles, California



FAULT ZONE MAP
LAC+USC WC Hospital Demolition
LAC+USC Medical Center
Los Angeles, California



SEISMIC HAZARD ZONE MAP
LAC+USC WC Hospital Demolition
LAC+USC Medical Center
Los Angeles, California

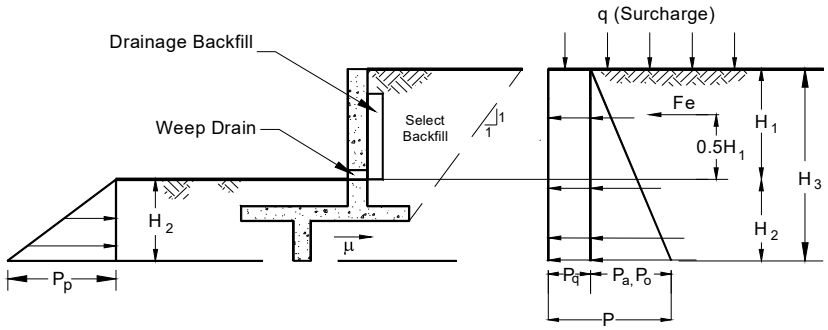


P_P = 300 H_2<3,000 psf	BRACED SHORING $P = P_q + P_s$ $= 0.44\ q + 53\ H_1$ (300 psf minimum)	CANTILEVER SHORING $P = P_q + P_s$ $= 0.28\ q + 31\ H_3$

Notes:

- All values of height (H) in feet, and pressure (P) and surcharge (q) in pounds per square foot (psf).
- Values for temporary excavations using flexible walls.
- For traffic surcharge, assume no less than a 100 psf uniform horizontal pressure along the top 10 feet.
- Earth pressure assumes no hydrostatic pressures. If hydrostatic pressures are allowed to build up, the incremental earth pressures below groundwater level should be reduced by 50 percent and added to hydrostatic pressure for total lateral pressure.

FIGURE 11 - LATERAL EARTH PRESSURES FOR TEMPORARY SHORING

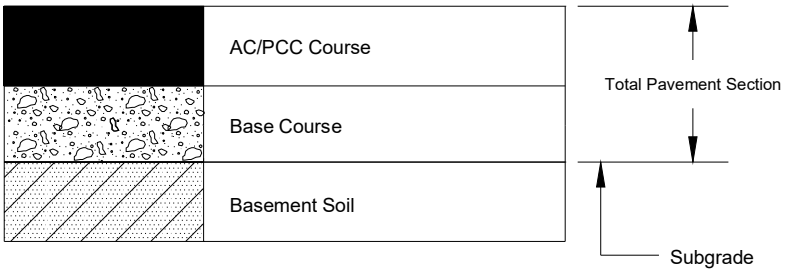


$P_p = 300\ H_2 \leq 3{,}000$ psf $\mu = 0.4$	Cantilever Walls $P = P_a + P_q = 31H_3 + 0.28q$ $Fe = 15H_1^2$	Restrained Walls $P = P_o + P_q = 53H_3 + 0.44q$ $Fe = 29H_1^2$

Notes:

- Select backfill for pressure behind the wall; import sandy fill for passive resistance.
- All values of height (H) in feet, pressure (P), and surcharge (q) in pounds per square foot (psf) and force (F) in pounds.
- P_p, P_a, and P_o are the passive, active, and at-rest earth pressures, respectively; Fe is the ultimate incremental seismic force.
- P_q is the incremental surcharge pressure; μ is the allowable friction coefficient applied to dead normal (buoyant) loads. Fe is in addition to the active and at-rest pressures. Below groundwater, active and at-rest pressure should be reduced by 50 percent and hydrostatic pressure should be added to active and at-rest pressures. P_p should be reduced by 50 percent below the groundwater.
- Adjacent footing bearing pressure should be considered as a surcharge, which applies at the bottom of footing.
- For 2H:1V slopes above the wall, increase the active and at-rest pressures by 50 percent; for 1.5H:1V slope, increase the active and at-rest pressures by 100 percent.
- For 5H:1V slopes below the toe of the retaining wall, decrease the passive pressures by 25 percent; for 3H:1V slope, decrease the passive pressures by 50 percent.
- Neglect the upper 1 foot for passive pressure unless the surface is contained by a pavement or slab.
- Friction and passive resistance may be used in combination, if the passive resistance is reduced by one-half.
- A factor of safety of 1.5 should be considered for ASD.

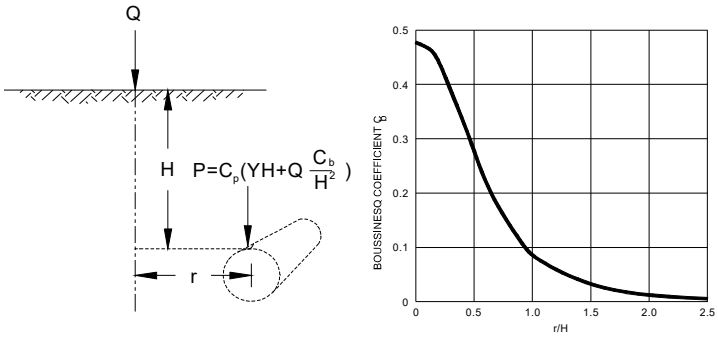
FIGURE 12 - LATERAL EARTH PRESSURES



COURSE		MINIMUM THICKNESS (inches)							
		AC Over Base			Full Depth AC			PCC Over Base	
		TI[1] = 5	TI = 7	TI = 9	TI = 5	TI = 7	TI = 9	TI = 5 to 7	TI = 9
R Value = 50	AC/PCC[2]	2	5	6	6	8	11	6	9
	Base[3]	5	8	10	--	--	--	6	6
	Basement Soil[4]	6	6	6	6	6	6	6	6
R Value = 10	AC/PCC[2]	3	5	7	8	11	14	6	9
	Base[3]	9	13	13	--	--	--	9	12
	Basement Soil[4]	18	12	10	22	19	16	10	10

Notes:
1. TI = Traffic Index.
2. Asphalt concrete (AC) and Portland cement concrete (PCC) should satisfy the requirements of Caltrans Standard Specifications Sections 39 and 40, respectively; or Greenbook Sections 203, and 201 and 302, respectively.
3. Base course = Crushed aggregate base or crushed miscellaneous base, in accordance with Caltrans Standard Specifications Section 26 or Greenbook Sections 200-2.2 and 200-2.4, respectively. The minimum relative compaction is 95 percent.
4. Compacted in-place natural soil or fill, except organic materials (i.e., peat) to be removed; the minimum is 95 percent relative compaction.

FIGURE 13 - PAVEMENT SECTION DESIGN



Notes:

- P = Vertical soil pressure at the top of the conduit (pounds per square foot [psf]).
- C_p = Concentration factor, assume 1.5.
- Y = Unit weight of soil (pounds per cubic foot [pcf]), assume 125 pcf.
- H = Height of fill above the top of the pipe (feet).
- Q = Concentrated surface load (pounds).
- C_b = Boussinesq coefficient obtained from figure above.
- The effects of more than one load are cumulative.

FIGURE 14 - VERTICAL EARTH PRESSURES BURIED OBJECTS

PROJECT NO. A9733-06-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**GRAVEL** **ARTIFICIAL FILL** Silty Sand, loose, dry, brown, fine- to medium-grained, some coarse-grained, some gravel (to 1"), some brick and asphalt fragments.			
2	B1@2.5'			SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained, some coarse-grained.	51	115.7	14.4
4	B1@5'				- trace coarse-grained	15	--	--
6								
8	B1@7.5'			ML	Sandy Silt, hard, slightly moist, dark brown, fine- to medium-grained, some coarse-grained.	63	121.5	15.2
10	B1@10'			SM	Silty Sand, dense, slightly moist, dark brown, fine- to medium-grained, some coarse-grained, some gravel (to 0.5").	41	--	--
12	B1@12.5'				Silt, firm, slightly moist, brown, trace fine- to coarse-grained sand.	20	108.4	19.6
14								
16	B1@15'		▼	ML	- soft, wet, fine- to medium-grained, some coarse-grained	6	--	--
18	B1@17.5'				- firm, light brown to yellowish brown	15	103.4	24.1
20	B1@20'				- saturated, yellowish brown	11	--	--
22	B1@22.5'				**PUENTE FORMATION** Interbedded Siltstone and Sandstone, yellowish brown, fine- to medium-grained, thinly bedded, soft, highly weathered, heavy oxidation.	24	94.4	29.4
24								
26	B1@25'					14	--	--
28	B1@27.5'				- moderately hard	43	97.5	28.5

A9733-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 1 of 2

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 1** ELEV. (MSL.) -- DATE COMPLETED **3/6/18** EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH** MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30'					40	--	--
32	B1@32.5'				- hard	50 (5")	103.3	25.4
34								
36	B1@35'					51	--	--
38								
40	B1@40'					50 (6")	--	--
42								
44								
46	B1@45'					54	--	--
48								
50	B1@50'					50 (4")	--	--
52								
54								
56					Total depth of boring: 56 feet Fill to 1.5 feet. Groundwater encountered at 17 feet. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

A9733-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 2 of 2

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

BORING 2

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**GRAVEL** **ARTIFICIAL FILL** Silty Sand, loose, dry, brown, fine- to medium-grained, some coarse-grained, some gravel (to 1"), fragments of brick and asphalt.			
2	B2@2'					65	101.2	15.3
4					**ALLUVIUM** Silty Sand, dense, slightly moist, brown, fine- to medium-grained, some coarse-grained.			
6	B2@5'			SM	- decrease in medium- to coarse-grained	65	103.9	12.9
8	B2@7'				- medium dense, increase in silt content	49	112.7	20.8
10	B2@10'			ML	Sandy Silt, stiff, slightly moist, brown, fine- to medium-grained.	33	109.7	18.8
12	B2@12'				- some coarse-grained, increase in sand content	34	100.9	21.1
14	B2@15'			SM	Silty Sand, dense, slightly moist, light brown, fine- to medium-grained, some coarse-grained.	57	110.1	15.7

Total depth of boring: 15.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9733-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

BORING 3

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			**GRAVEL**			
			ARTIFICIAL FILL			
2	B3@2'		Silty Sand, loose, dry, brown, fine- to medium-grained, some coarse-grained, some gravel (to 1").	20	104.6	19.2
		ML	**ALLUVIUM**			
4			Sandy Silt, firm, slightly moist, dark brown, fine- to coarse-grained.			
	B3@5'		- stiff, increase in sand content	28	101.4	15.8
6			Silty Sand, medium dense, slightly moist, light brown, fine-grained, some medium-grained, trace gravel (to 0.5").			
	B3@7'			28	111.9	16.4
8		SM				
10	B3@10'		- some oxidation staining	39	109.5	20.1
12	B3@12'		Sandy Silt, stiff, slightly moist, olive brown, fine-grained, trace medium- to coarse-grained.	32	94.7	27.6
		ML				
14						
	B3@15'		- decrease in sand content	40	114.9	18.0

Total depth of boring: 15.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9733-06-01 BORING LOGS.GPJ

Figure A3,
Log of Boring 3, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

BORING 4

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT**			
					ARTIFICIAL FILL			
2	B4@2'			ML	Silty Sand, loose, dry, dark brown, fine- to medium-grained, some coarse-grained, some gravel (to 1").	18	104.0	16.2
					ALLUVIUM			
4					Sandy Silt, firm, slightly moist, brown, fine- to coarse-grained.			
					Silty Sand, loose, slightly moist, light yellowish brown, fine-grained.			
6	B4@5'			SM		14	112.4	19.8
	B4@7'				- some bedrock framents	14	100.8	23.7
8								
10					Sandy Silt, firm, slightly moist, dark brown, fine- to medium-grained.			
	B4@10'					17	103.7	24.5
12	B4@12'			ML	- soft, moist, some coarse-grained	8	98.7	26.7
14								
	B4@15'				- firm, brown to yellowish brown, fine- to coarse-grained	13	97.1	26.6

Total depth of boring: 15.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9733-06-01 BORING LOGS.GPJ

Figure A4,
Log of Boring 4, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

BORING 5

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ASPHALT**			
					ARTIFICIAL FILL Silty Sand, loose, dry, brown, fine- to medium-grained, some coarse-grained, some gravel (to 1").			
2	B5@2'					12	99.9	21.0
					ALLUVIUM Sandy Silt, firm, slightly moist, light brown, fine- to coarse-grained.			
4								
	B5@5'				- yellowish brown, fine- to medium-grained	14	100.9	22.1
6				ML				
	B5@7'				- dark brown	18	110.5	16.5
8								
10	B5@10'				- soft, dark yellowish brown, trace coarse-grained	9	86.7	16.8
					Sand with Silt, well-graded, medium dense, slightly moist, brown to yellowish brown, fine- to coarse-grained, some gravel (to 0.5").			
12	B5@12'			SW-SM		27	119.0	10.3
14					**PUENTE FORMATION** Interbedded Siltstone and Sandstone, yellowish brown, fine- to medium-grained, thinly bedded, soft to moderately hard, highly weathered, oxidation.			
	B5@15'					53	100.1	23.7

Total depth of boring: 15.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9733-06-01 BORING LOGS.GPJ

Figure A5,
Log of Boring 5, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

APPENDIX A-A

FIELD INVESTIGATION

The site was explored on March 6, 2018, by excavating five 8-inch-diameter borings utilizing a truck-mounted hollow-stem auger drilling machine. The borings were excavated to depths of approximately 15½ feet and 56 feet below the existing ground surface. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2 3/8-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained. Standard Penetration Tests (SPTs) were performed in boring B1.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A5. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the boring logs were revised based on subsequent laboratory testing. The location of the borings are shown on Figure 2.

APPENDIX A-B

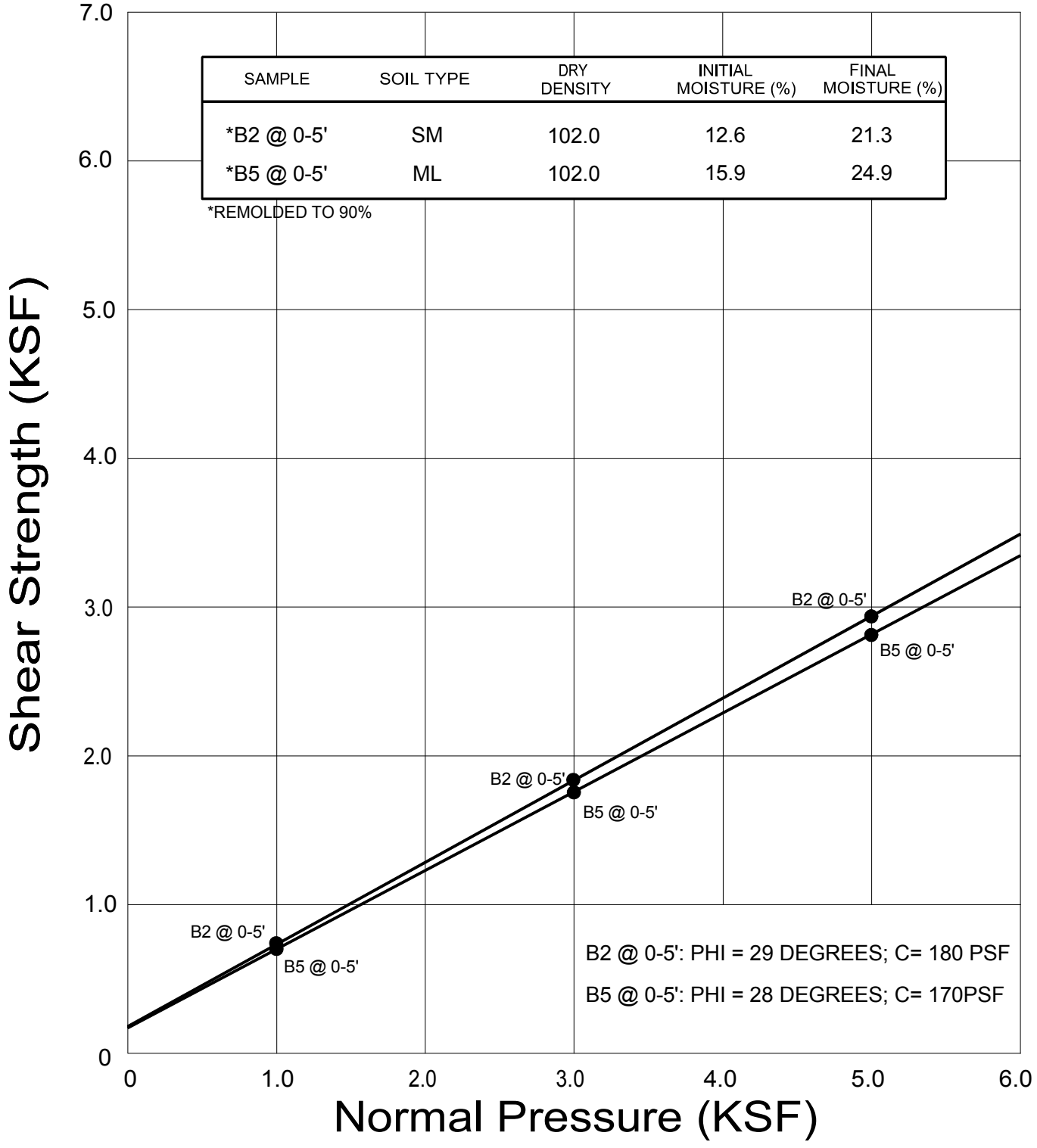
APPENDIX A-B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the "American Society for Testing and Materials (ASTM)", or other suggested procedures. Selected samples were tested for direct shear strength, consolidation and expansion characteristics, plasticity indices, grain size, corrosivity, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B8. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A-A.

SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
*B2 @ 0-5'	SM	102.0	12.6	21.3
*B5 @ 0-5'	ML	102.0	15.9	24.9

*REMOLDED TO 90%



● Direct Shear, Saturated

GEOCON
WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

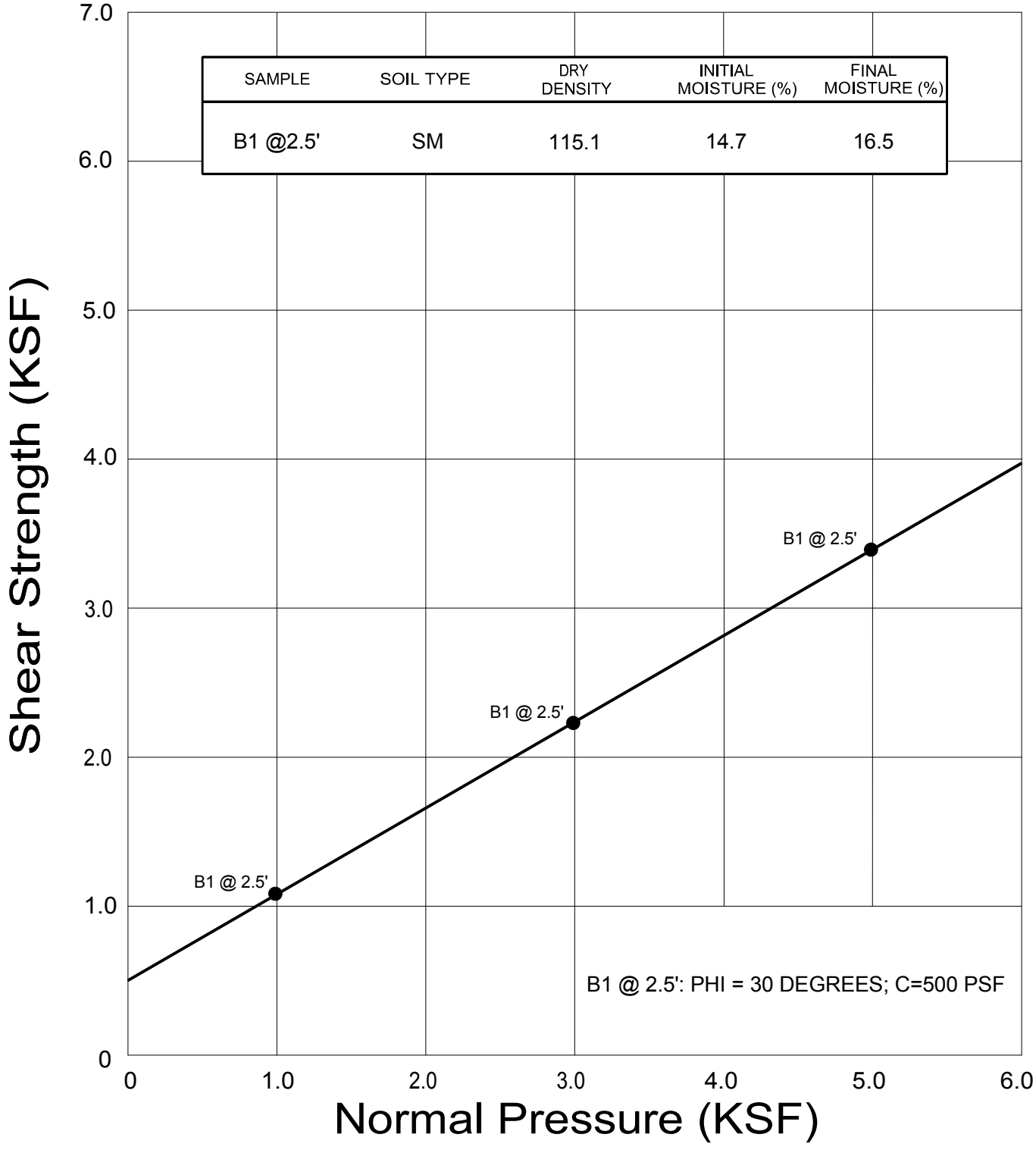
DRAFTED BY: RP | CHECKED BY: HHD

DIRECT SHEAR TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. B1

SAMPLE	SOIL TYPE	DRY DENSITY	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1 @2.5'	SM	115.1	14.7	16.5



● Direct Shear, Saturated

GEOCON
WEST, INC.

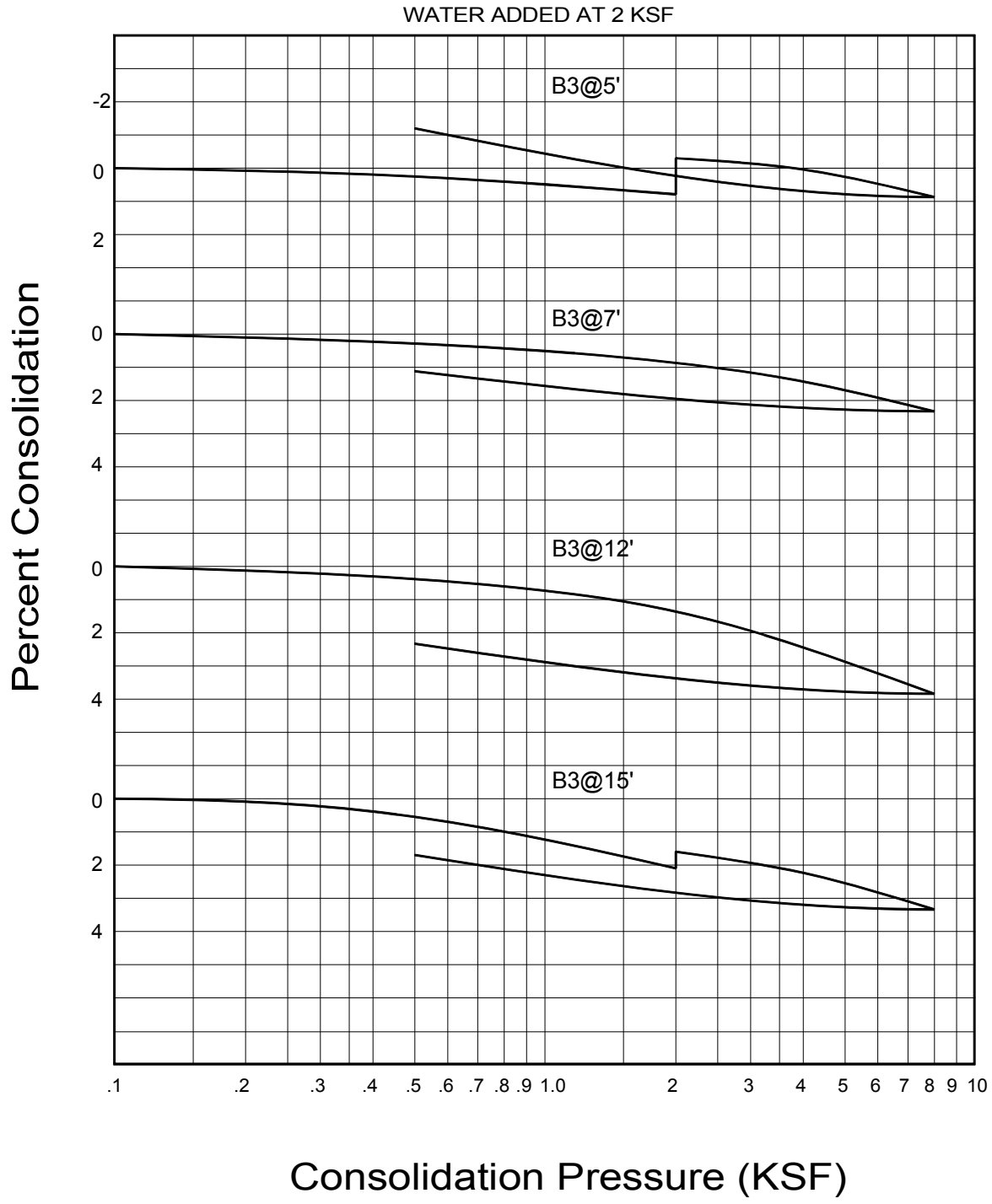
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMH | CHECKED BY: HHD

DIRECT SHEAR TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. B2



GEOCON
WEST, INC.

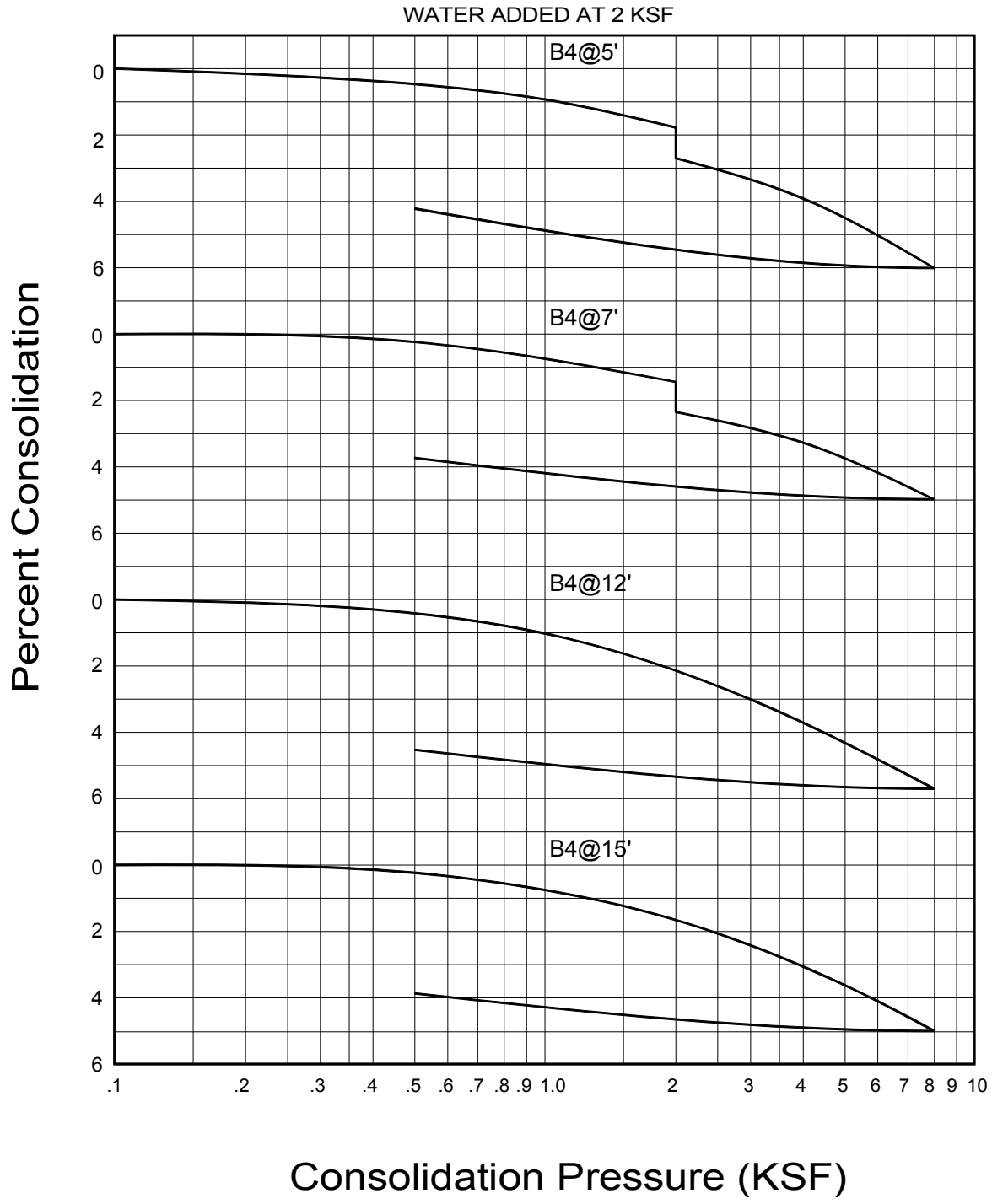
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RP	CHECKED BY: HHD

CONSOLIDATION TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018	PROJECT NO. A9733-06-01	FIG. B3



GEOCON
WEST, INC.

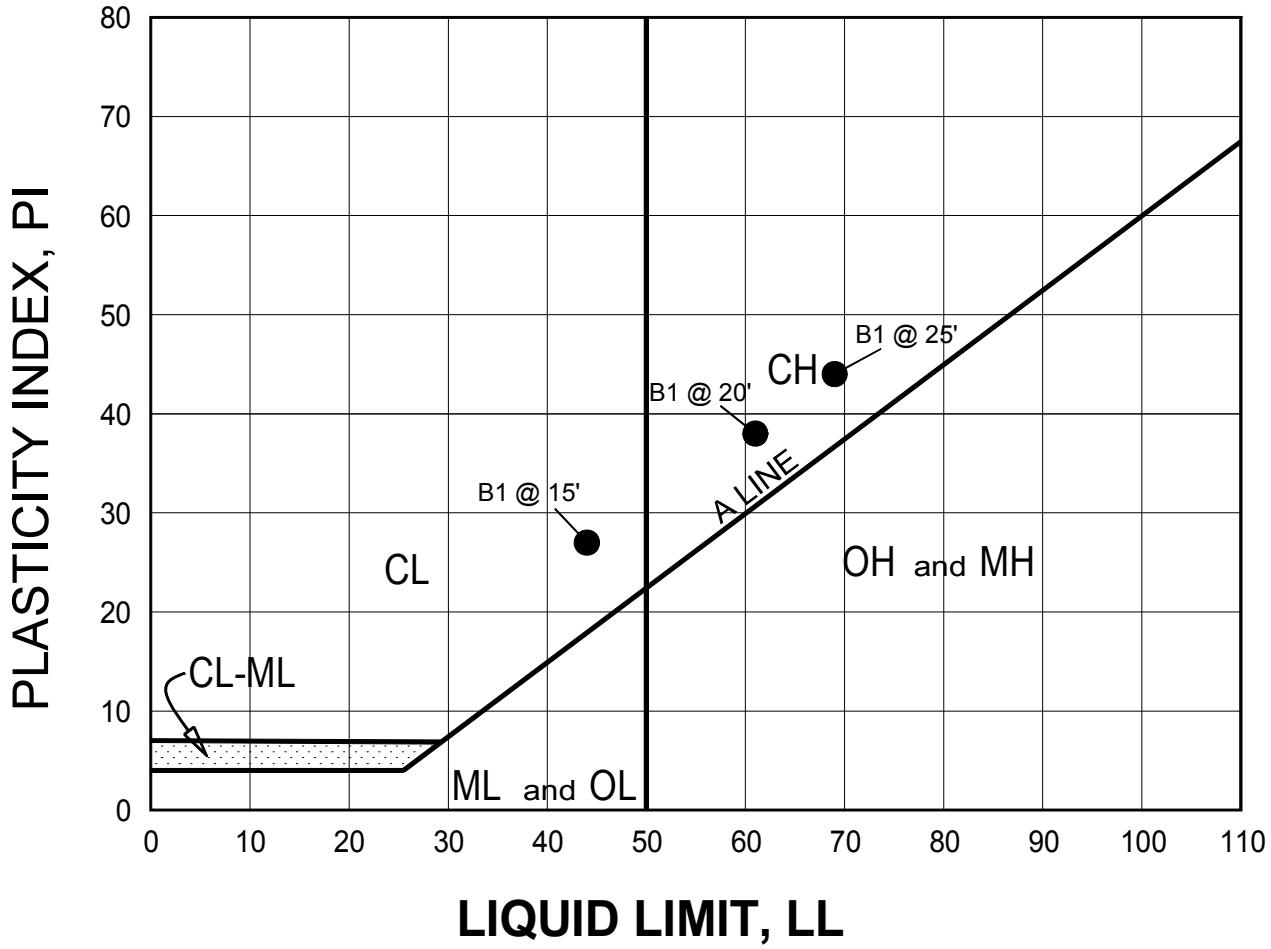
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RP	CHECKED BY: HHD

CONSOLIDATION TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018	PROJECT NO. A9733-06-01	FIG. B4



BORING NUMBER	DEPTH (FEET)	LL	PL	PI	MOISTURE CONTENT AT SATURATION (%)	SOIL BEHAVIOR
B1	5.0	24.0	18.0	6.0	---	*N/P
B1	15.0	44.0	17.0	27.0	15.1	CL
B1	20.0	61.0	23.0	38.0	16.1	CH
B1	25.0	69.0	25.0	44.0	19.1	CH

*N/P indicates Non-Plastic

GEOCON WEST, INC.

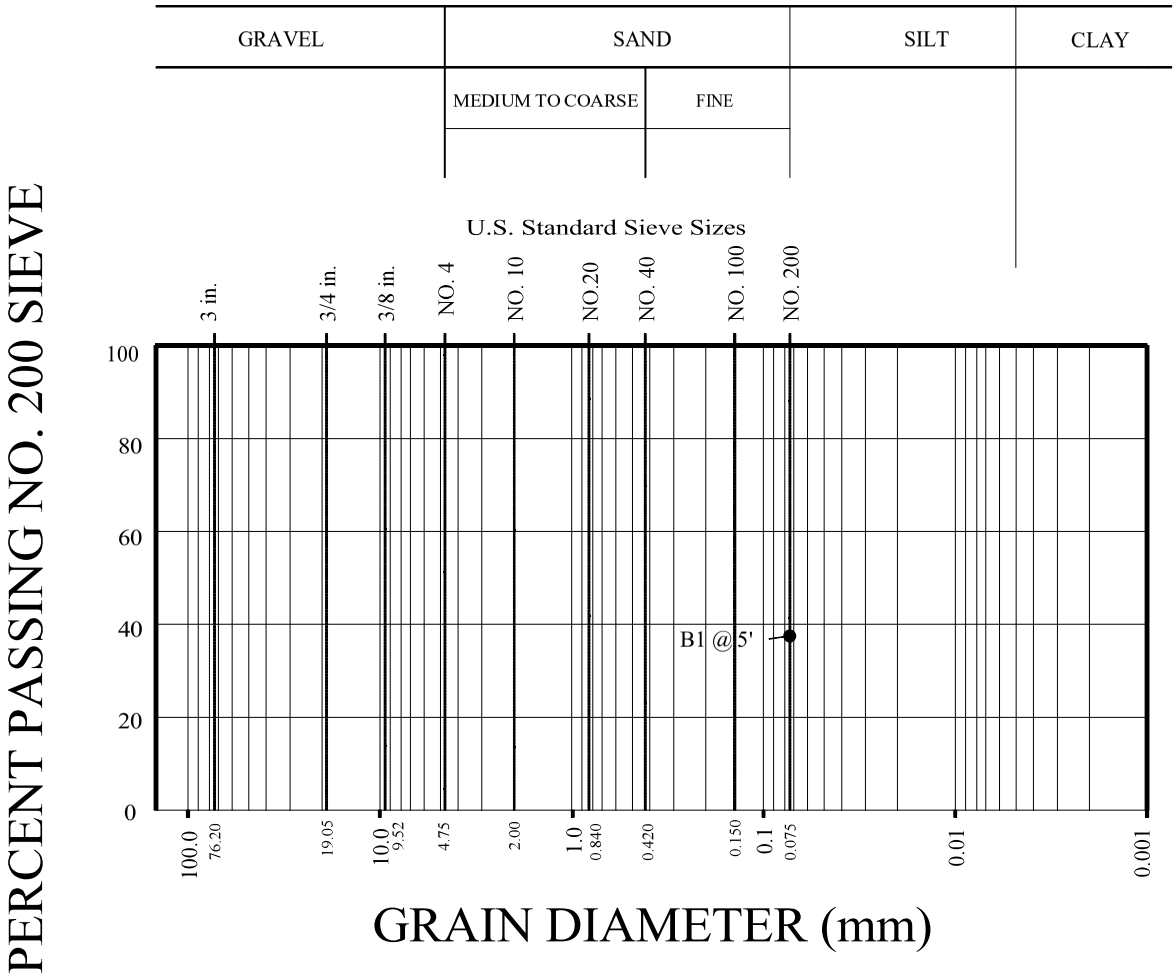
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMH | CHECKED BY: HHD

ATTERBERG LIMIT

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. B5



SAMPLE	PERCENT PASSING NO. 200 SIEVE
B1 @ 5'	37.6

GEOCON
WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: RP	CHECKED BY: HHD

GRAIN SIZE ANALYSIS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018	PROJECT NO. A9733-06-01	FIG. B6

SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D 4829-11

Sample No.	Moisture Content (%)		Dry Density (pcf)	Expansion Index	*UBC Classification	**CBC Classification
	Before	After				
B2 @ 0-5'	10.4	24.1	106.1	81	Moderate	Expansive
B5 @ 0-5'	13.1	29.1	100.0	101	High	Expansive

* Reference: 1997 Uniform Building Code, Table 18-I-B.

** Reference: 2016 California Building Code, Section 1803.5.3

SUMMARY OF LABORATORY MAXIMUM DENSITY AND AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-12

Sample No.	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture (%)
B2 @ 0-5'	Brown Silty Sand	120.0	12.0
B5 @ 0-5'	Yellowish Brown Sandy SIlt	113.0	16.0

GEOCON WEST, INC.

ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMH | CHECKED BY: HHD

LABORATORY TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. B7

SUMMARY OF LABORATORY POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS CALIFORNIA TEST NO. 643

Sample No.	pH	Resistivity (Ohm Centimeters)
B2 @ 0-5'	7.55	740 (Severely Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS EPA NO. 325.3

Sample No.	Chloride Ion Content (%)
B2 @ 0-5'	0.013

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS CALIFORNIA TEST NO. 417

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure*
B2 @ 0-5'	0.498	Severe

* Reference: 2016 California Building Code, Section 1904.3 and ACI 318-11 Section 4.3.

GEOCON WEST, INC.

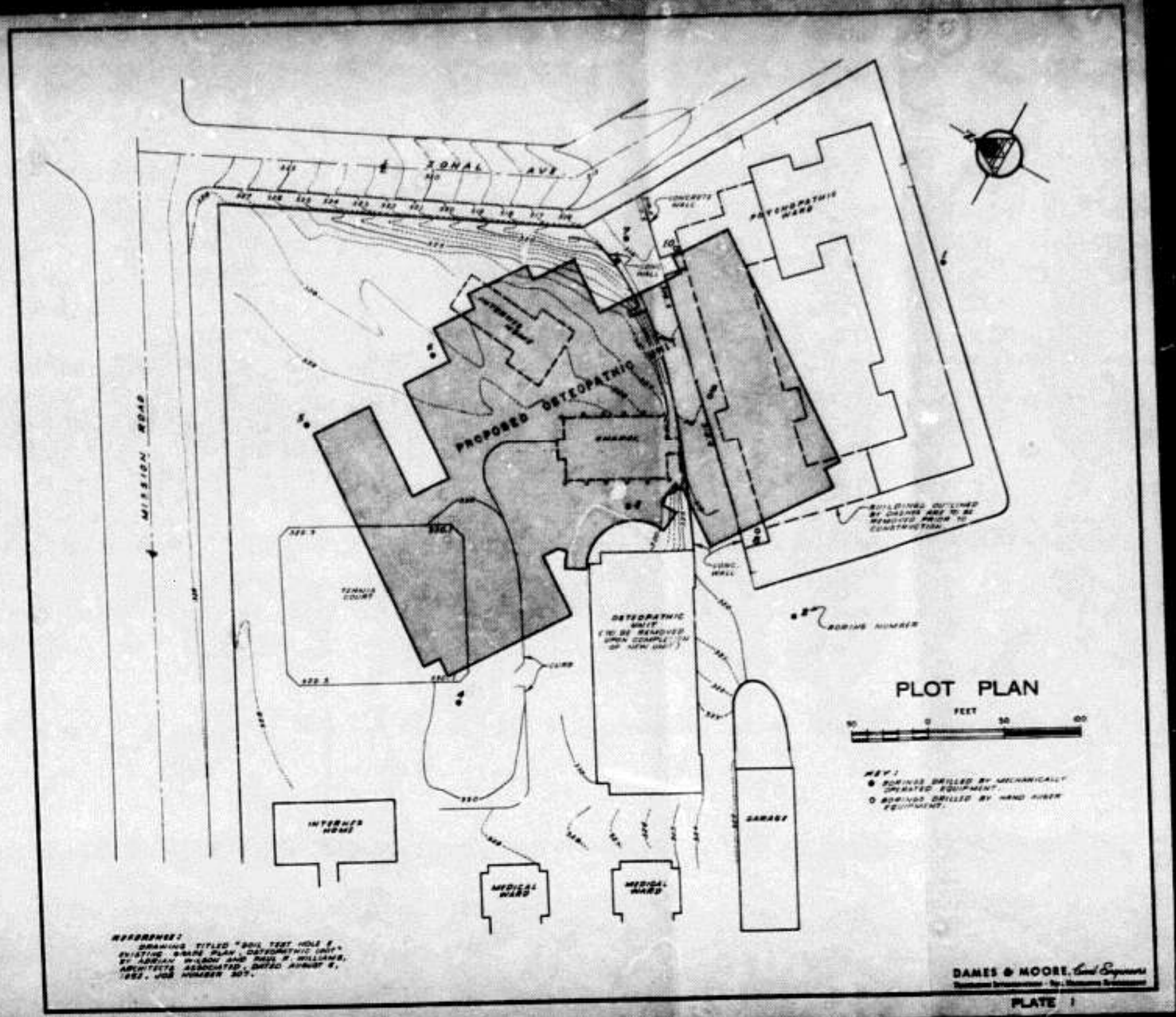
ENVIRONMENTAL GEOTECHNICAL MATERIALS
3303 N. SAN FERNANDO BLVD. - SUITE 100 - BURBANK, CA 91504
PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JMH | CHECKED BY: HHD

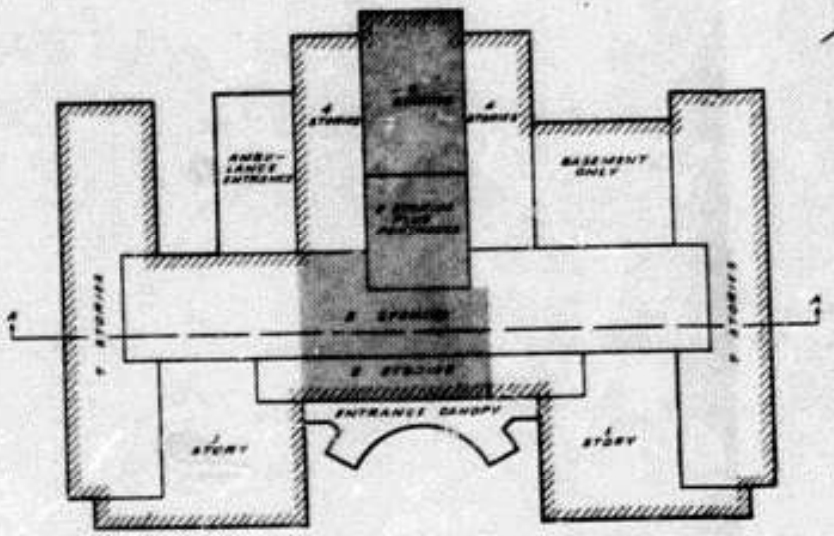
CORROSIVITY TEST RESULTS

HEALTH AGENCY
RECUPERATIVE CAMPUS
1774 ZONAL AVENUE
LOS ANGELES, CALIFORNIA

APRIL 2018 | PROJECT NO. A9733-06-01 | FIG. B8


ZONAL AVE.
MISSION ROAD
PROPOSED OSTEOPATHIC
PSYCHOPATHIC WARD
CONCRETE WALL
TENNIS COURT
OSTEOPATHIC UNIT
(TO BE REMOVED UPON COMPLETION OF NEW UNIT)
BUILDINGS OUTLINED BY DASHES ARE TO BE REMOVED PRIOR TO CONSTRUCTION
BORING NUMBER
INTERNES' HOME
MEDICAL WARD
MEDICAL WARD
GARAGE
PLOT PLAN
FEET
KEY:
● BORINGS DRILLED BY MECHANICALLY OPERATED EQUIPMENT.
○ BORINGS DRILLED BY HAND AUGER EQUIPMENT.
REFERENCE:
DRAWING TITLED "SOIL TEST HOLE &
EXISTING GRADE PLAN, OSTEOPATHIC UNIT"
BY ADRIAN WILSON AND PAUL R. WILLIAMS,
ARCHITECTS ASSOCIATED, DATED AUGUST 6,
1952, JOB NUMBER 309.
DAMES & MOORE, Soil Engineers
PLATE 1

BUILDING PLAN



KEY:

BASEMENT AREA

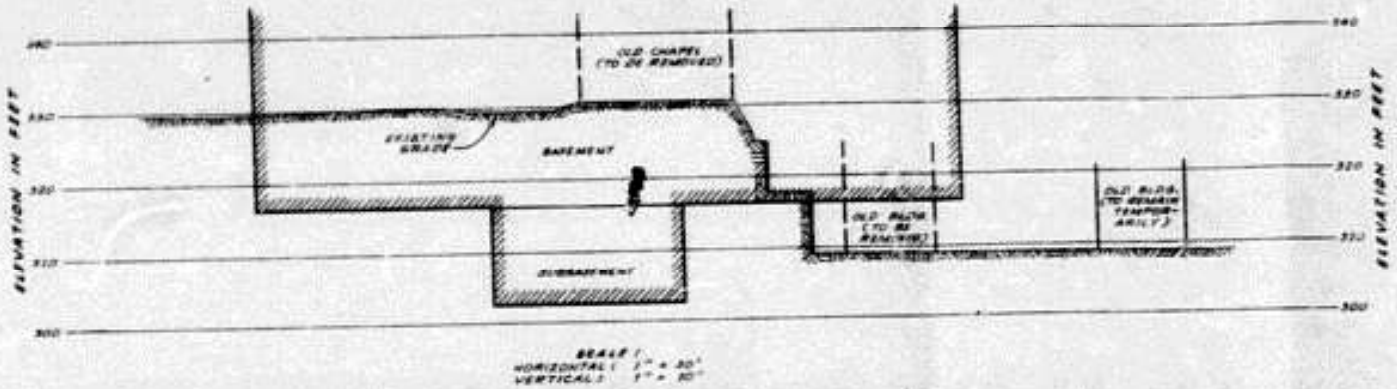
BASEMENT AND SUBBASEMENT AREA

REFERENCE:

DRAWINGS OF THE PROPOSED NEW OSTEOPATHIC UNIT FOR LOS ANGELES COUNTY GENERAL HOSPITAL, COUNTY OF LOS ANGELES, CALIFORNIA, BY ADRIAN WILSON AND PAUL P. WILLIAMS, ARCHITECTS ASSOCIATED TITLED "SOIL TEST HOLE PLAN" REVISED 2-25-53, "BASEMENT PLAN" REVISED 8-5-53, AND SUBBASEMENT DATED 4-53.

SECTION A —— A

SECTION SHOWING GAS VENT GRADES

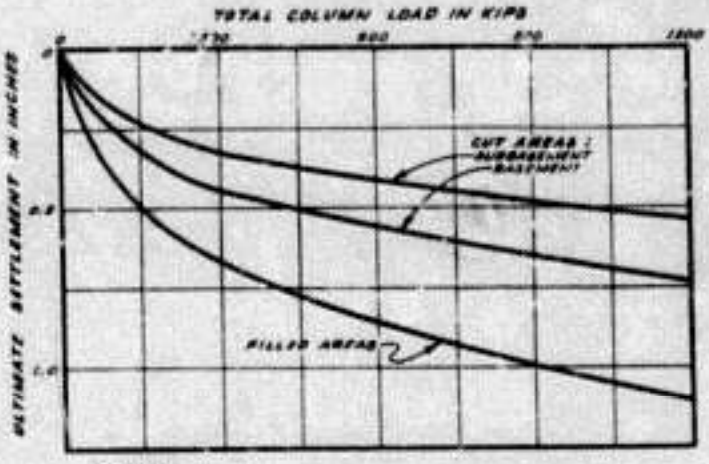


SCALE:
HORIZONTAL: 1" = 30'
VERTICAL: 1" = 30'

DATA ON PROPOSED BUILDING

DAMES & MOORE, Soil Engineers

PLATE 2



NOTES :

SETTLEMENT ANALYSES WERE PERFORMED FOR FOOTINGS FOUNDED AT A DEPTH OF THREE (3) FEET INTO UNDISTURBED NATURAL SOILS AND DESIGNED FOR A TOTAL BEARING VALUE OF 5,500 POUNDS PER SQUARE FOOT.

AREAL SUBSIDENCE IN FILL AREAS WILL BE ADDITIVE TO THE SETTLEMENTS SHOWN FOR COLUMNS IN THOSE AREAS.

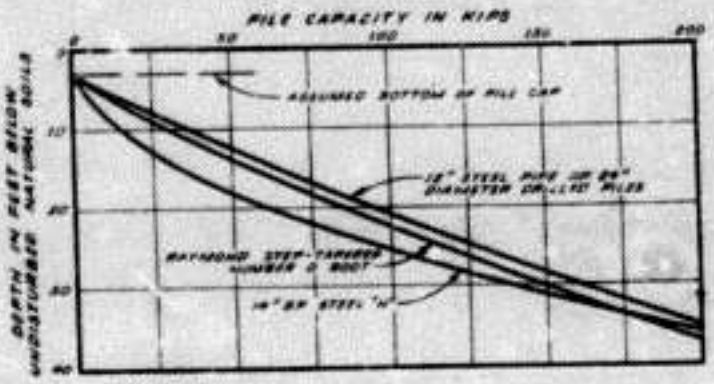
THE RATIO OF REAL LOAD TO TOTAL DESIGN LOAD IS ASSUMED TO BE NINE-TENTHS; SETTLEMENTS FOR OTHER RATIOS WOULD BE PROPORTIONAL.

SPREAD FOUNDATION SETTLEMENTS

DAMES & MOORE, Soil Engineers

PLATE 3

PILE CAPACITIES



NOTES:

THE VALUES SHOWN ARE LIMITED BY THE STRESSES IN THE SOIL. THE STRESSES IN THE PILE ITSELF MAY IMPOSE MORE SEVERE LIMITATIONS.

THE INDICATED CAPACITIES APPLY TO THE TOTAL OF ALL LOADS, DEAD, LIVE, AND SEISMIC.

THE TOTAL OF ALL PERMANENTLY AND/OR FREQUENTLY APPLIED REAL LOADS SHOULD BE LIMITED TO TWO-THIRDS OF THE INDICATED CAPACITIES.

DRIVEN PILES SHOULD NOT BE SPACED CLOSER THAN THREE (3) FEET ON CENTERS. DRILLED PILES SHOULD BE SPACED AT MINIMUM OF 3 DIAMETERS ON CENTERS.

PILE FOUNDATIONS

DAMES & MOORE, Soil Engineers

PLATE 4

APPENDIX

EXPLORATIONS AND LABORATORY TESTS

EXPLORATIONS

The site was explored by drilling a total of ten exploration borings to depths ranging from 9½ to 66½ feet below the ground surface. Initially, Borings 1 through 7 were drilled with 16-inch-diameter rotary bucket-type drilling equipment. Soils of poor supporting quality were encountered in the upper 20 feet of Boring 2. This boring was drilled at a location some 60 feet from the proposed building location since nearer locations were inaccessible to truck-mounted drilling equipment required to perform the deep borings. In order to ascertain whether these poorer soil conditions extended under part of the proposed building, three shallow borings (8 through 10) were drilled to the firm soils using hand operated portable equipment.

The soils encountered were classified by visual and textural examination in the field, supplemented by inspection in the laboratory of samples obtained during drilling. Core samples of the various soils penetrated by the exploration borings were obtained in a soil sampler of the type illustrated on Page A-I of this Appendix.

Graphical representations of the soils encountered in the exploration borings are presented on Plates A-1A through A-1E, Log of Borings. The nomenclature used in soil classification is defined by the Soil Classification Chart shown on Plate A-1E. The elevations of the borings refer to Mean Sea

level. Ground water was not encountered in any of the exploration borings. However, water seepage occurred in a number of the borings at depths ranging from 18 to 42½ feet below the ground surface. Where such water seepage occurred, it is noted on the appropriate log of borings.

LABORATORY TESTS

SHEAR TESTS:

To determine the shear strength of the soils encountered, shear tests were performed on selected undisturbed samples in the manner described on Page A-II, Method of Performing Shear and Friction Tests. Samples were tested at their field moisture content at a surcharge pressure equal to the pressure of the overburden existing at the time of exploration, and at reduced values of surcharge in order to evaluate the shear strength of the soils after basement excavations. Representative samples were also tested at artificially increased moisture content in order to evaluate their shear strengths under conditions of moisture infiltration. The results of the shear tests, and of the associated moisture and density determinations, are presented on the left of the appropriate boring logs in the manner described by the Key to Test Data shown on Plate A-1E. The results of the shear tests have also been summarized and are presented on the left graph on Plate A-2, Summary of Test Data. The values used in the analyses are presented by the line shown on the Shear Summary.

CONSOLIDATION TESTS:

To provide data for the determination of settlement, consolidation tests were performed on five representative undisturbed samples of the soils encountered. The tests were performed in accordance with the Method of Performing Consolidation Tests described on Page A-II of this Appendix. All

but one of the samples were tested in one loading cycle. One sample was tested in two loading cycles, in order to evaluate the effect of evacuation on the consolidation characteristics of the soil. The load-versus-consolidation curves obtained from the consolidation tests are presented on the right hand graphs on Plate A-2.

-oOo-

The following Plates are attached and complete this Appendix:

Plate A-1A - Log of Borings (Boring 1)

Plate A-1B - Log of Borings (Boring 2)

Plate A-1C - Log of Borings (Boring 3)

Plate A-1D - Log of Borings (Boring 4)

Plate A-1E - Log of Borings (Boring 5)

Plate A-1F - Log of Borings (Boring 6)

Plate A-1G - Log of Borings (Boring 7)

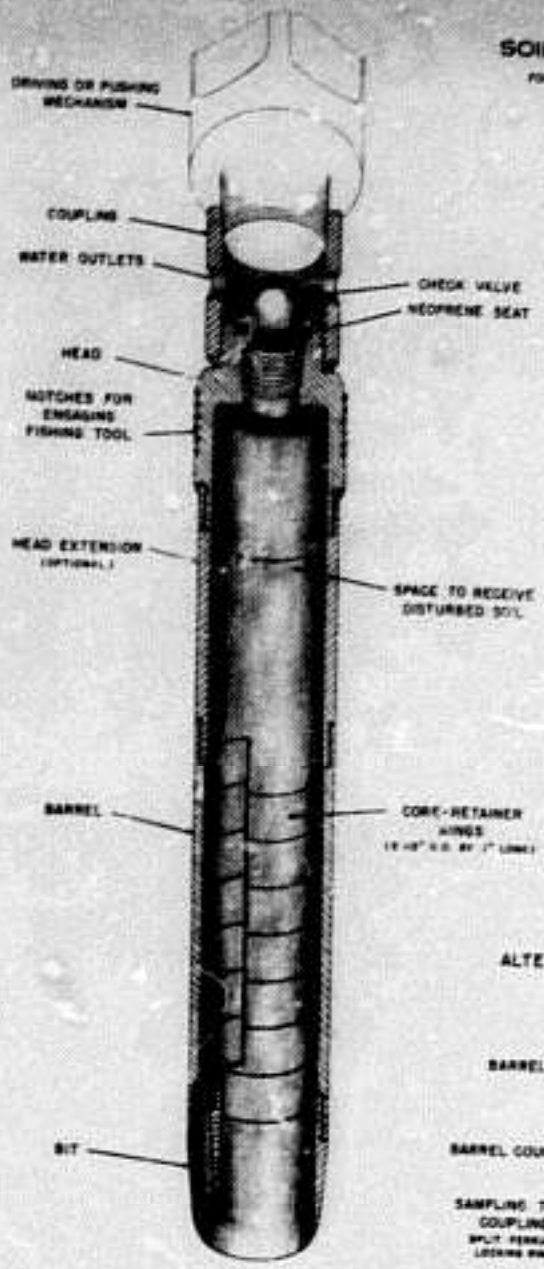
Plate A-1H - Log of Borings (Borings 8 through 10 - Soil Classification Chart and Key to Test Data)

Plate A-2 - Summary of Test Data.

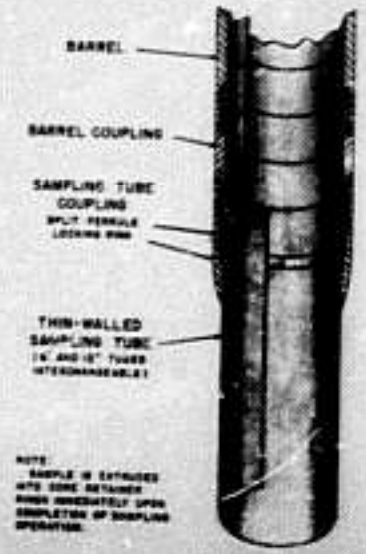
A-I

SOIL SAMPLER TYPE D

FOR SOILS EASY TO RETAIN IN SAMPLER



ALTERNATE ATTACHMENTS

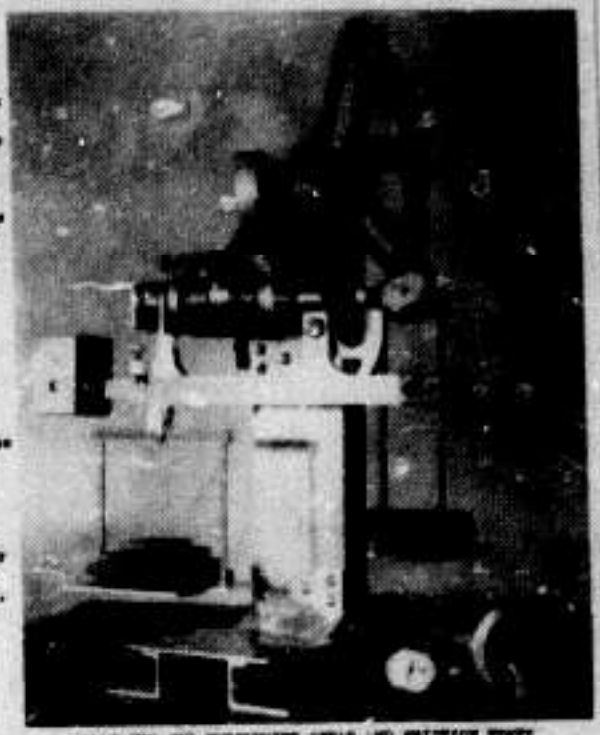


NOTE:
SAMPLE IS EXTRUDED INTO CORE RETAINER RINGS IMMEDIATELY UPON COMPLETION OF SAMPLING OPERATION.

A-I (C)

METHOD OF PERFORMING SHEAR AND FRICTION TESTS

In the performance of the shear tests, a three-inch length of the core sample is tested in double shear while confined with a three of the retainer rings in which the sample was obtained. The sample is placed in the shearing apparatus under the surcharge pressure at which it is desired to determine the shearing strength of the soil and then, at equal time intervals, equal increments of load are applied in a direction perpendicular to the axis of the sample to the center ring of the three-ring sample of soil, while all transverse movements of the outer rings are prevented. The deflections of the center ring and the axial movements of the sample are measured for each load increment by micrometer dial gauges. These deflections are plotted against the shearing load, giving a stress-strain curve which is used to determine the yield point shearing strength of the core, this shearing strength being related to the bearing value in the ratio 1.1*. The bearing value so computed is that yield point bearing value which would be established by a properly conducted field load test performed at the same conditions of moisture and confinement as the shear tests.



APPARATUS FOR PERFORMING SHEAR AND FRICTION TESTS
Shows lower half of sample carriage including shear dial removed to expose partly sheared core sample of soil.

In order to determine the decrease in strength which may occur with future increases in moisture content, the strength of the soil at possible future moisture conditions is determined by performing the shear test at an artificially increased moisture content. The change in moisture content is accomplished by immersing the sample in water for a period of at least 18 hours (The period is varied depending on the characteristics of the soil and the probable amount of moisture infiltration which may occur in the field.) while the sample is confined under the desired surcharge pressure.

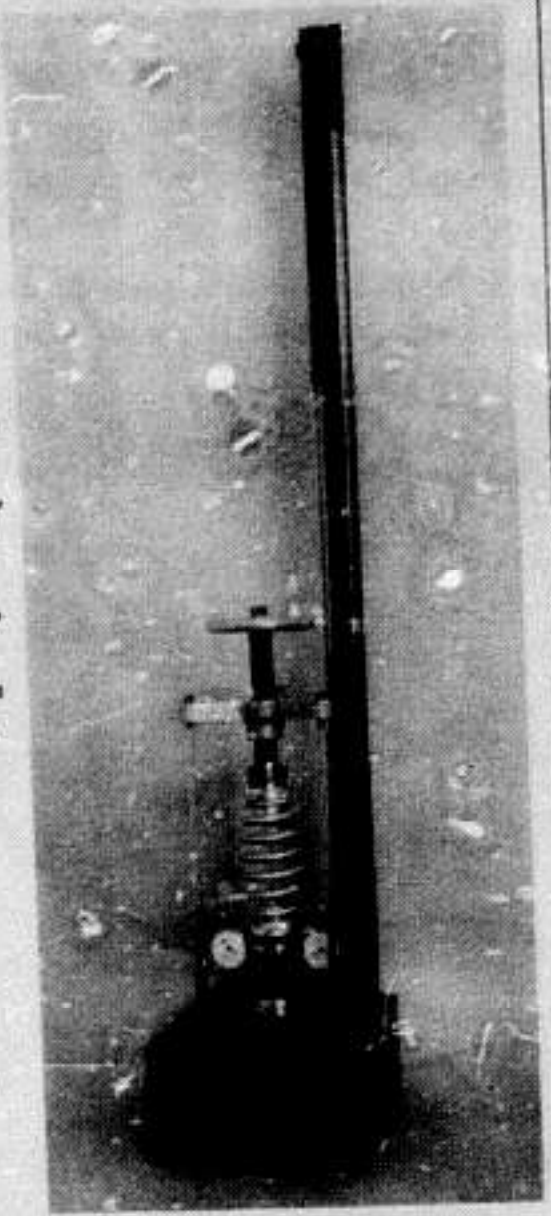
Friction tests of the soil in contact with steel, wood, or concrete, representing the surfaces of the various types of driven piling, are performed in the same apparatus and at the same surcharge pressures, moisture contents, and densities as the corresponding shear tests. The method of performing the test is identical to that of the shear tests, the only difference being that the center ring of soil is replaced by a disk of steel, wood, or concrete.

---

*"Practical Shear Tests for Foundation Design", Civil Engineering, December, 1942, Pages 738 and 739.

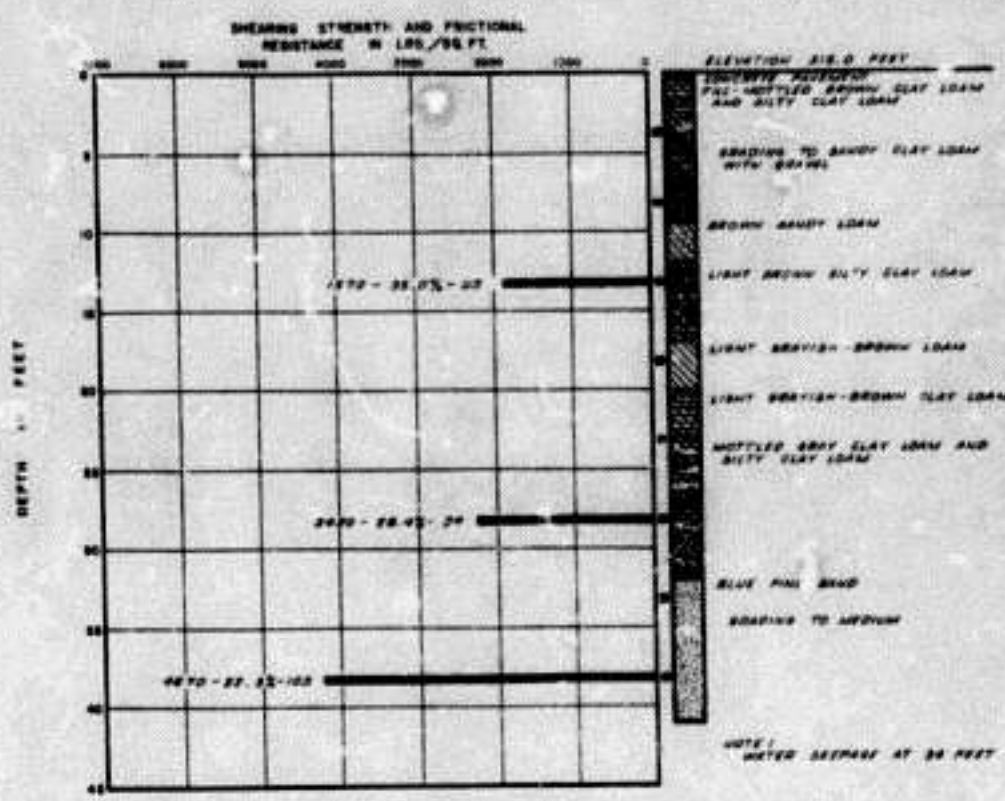
METHOD OF PERFORMING CONSOLIDATION TESTS

To determine the consolidation characteristics of the foundation soils so that settlement analyses may be performed, consolidation tests are performed in the consolidation apparatus on confined one-inch-thick core samples compressed between two porous stones. Axial loads are applied to the sample in increments and observations of the consolidation under the various load increments, as measured by micrometer dial gauges, are recorded at various time intervals. In general, each load is maintained constant until the rate of consolidation is observed to cease substantially, and then an additional load increment is applied and the observation repeated. From these data, time-versus-consolidation curves and load-versus-consolidation curves are obtained which enable estimates to be made of the probable magnitude and rate of settlement of foundations supported by the soil tested.



APPARATUS FOR PERFORMING CONSOLIDATION TESTS
Shows one-inch length of soil sample confined in retainer ring and under axial load.

BORING 1



LOG OF BORINGS

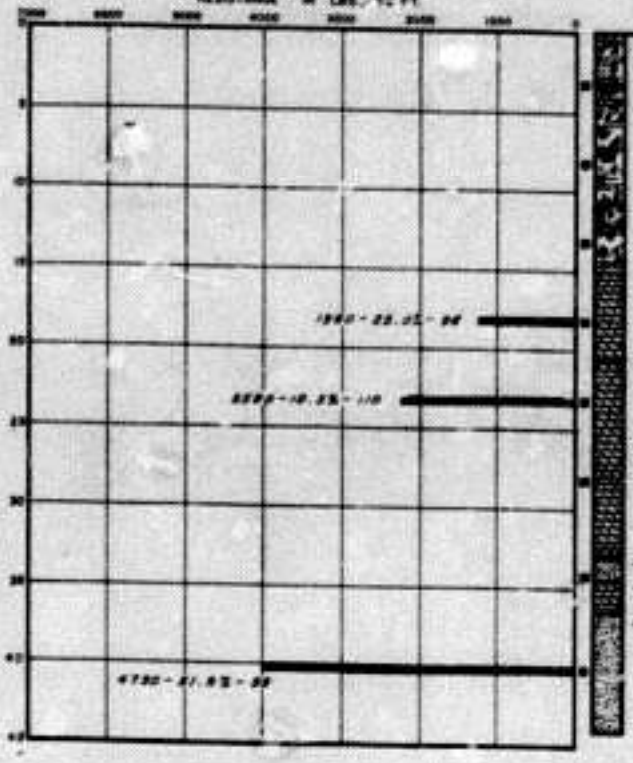
ELEVATION REFERENCE: MEAN SEA LEVEL

DAMES & MOORE, Soil Engineers

PLATE A-12

BORING 2

SHEARING STRENGTH AND FRICTIONAL RESISTANCE IN LBS./SQ FT

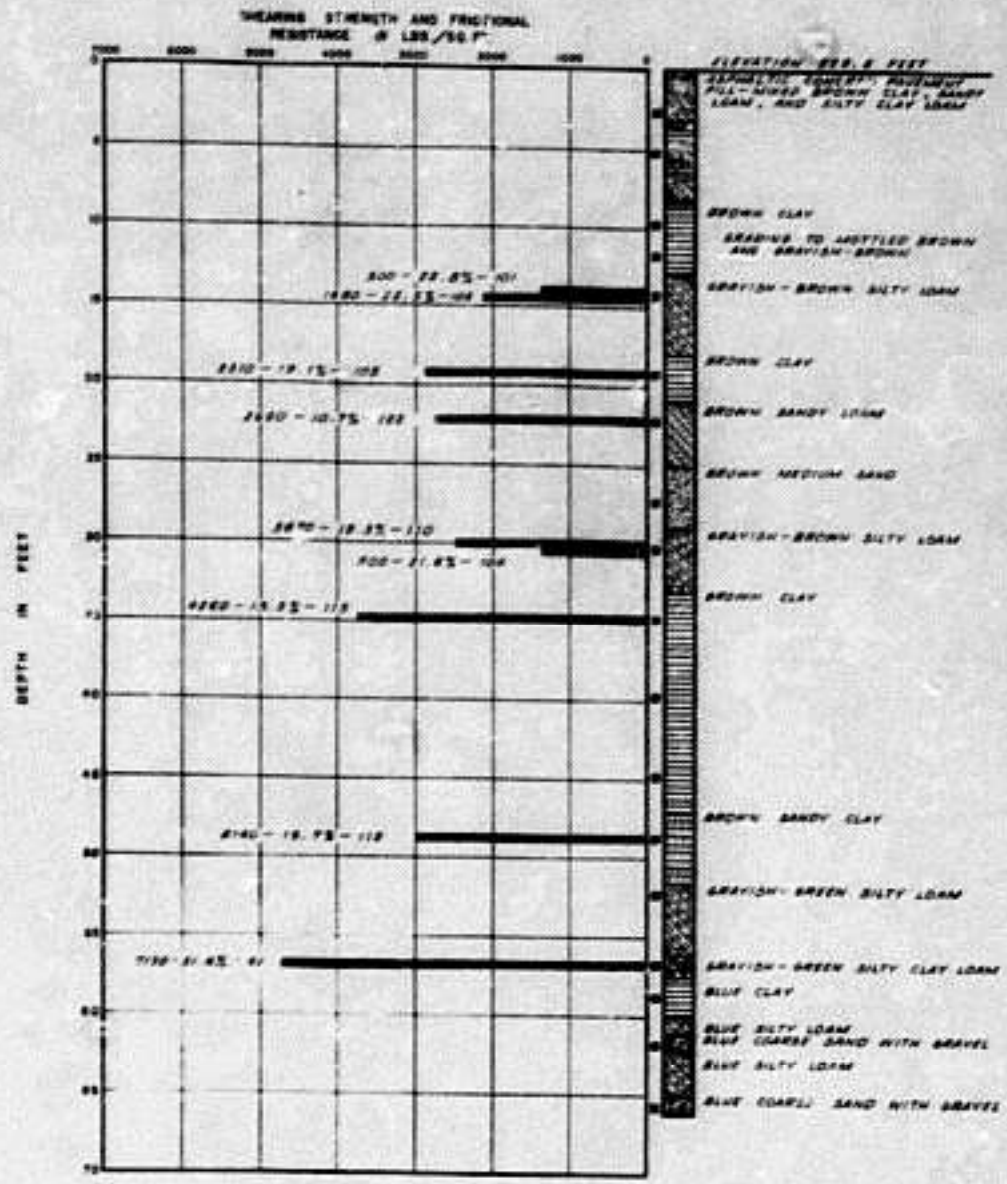


LOG OF BORINGS

EAMES & MOORE, Soil Engineers

PLATE A-18

BORING 5

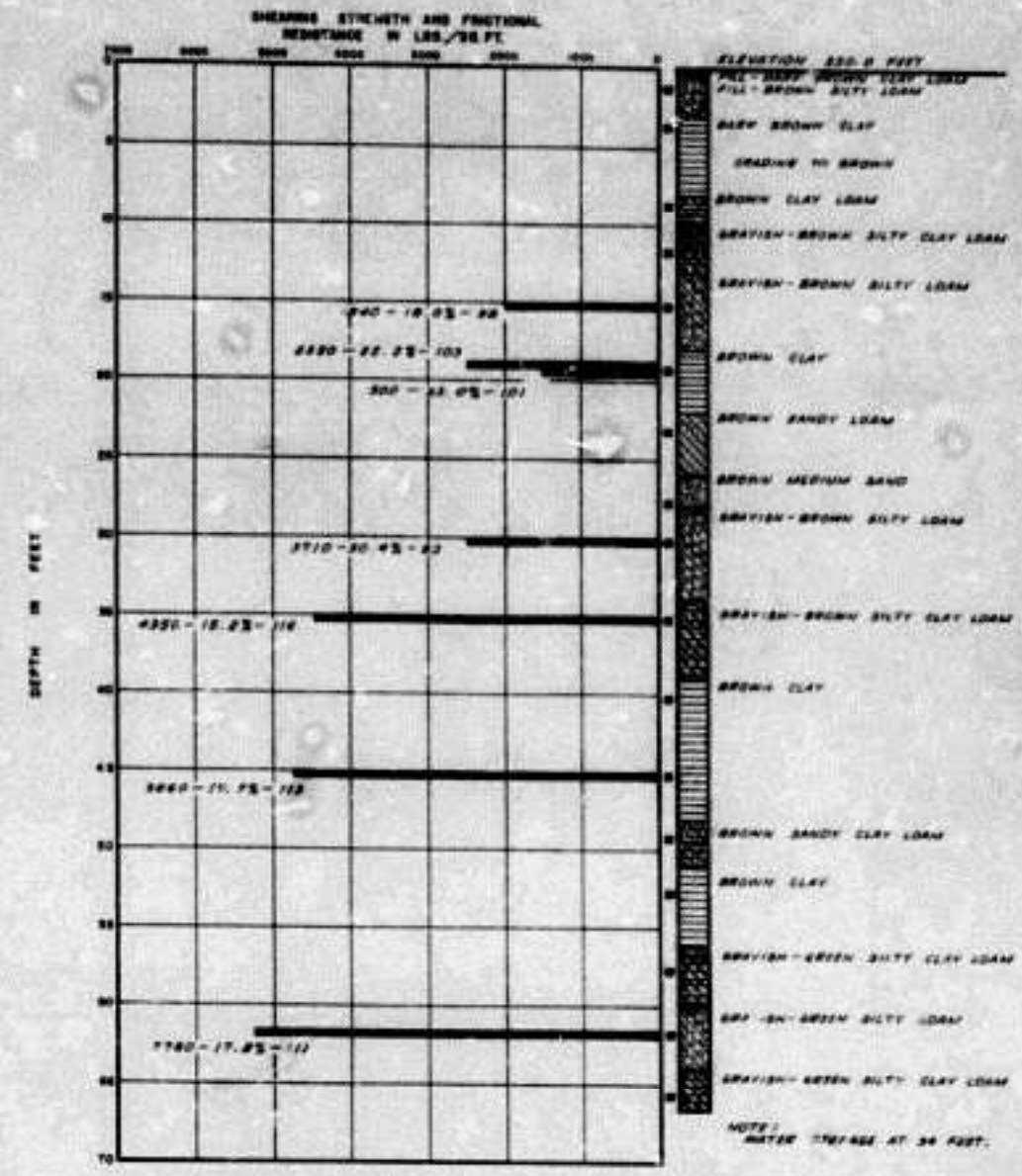


LOG OF BORINGS

DAMES & MOORE, Soil Engineers

PLATE A-1C

BORING 4

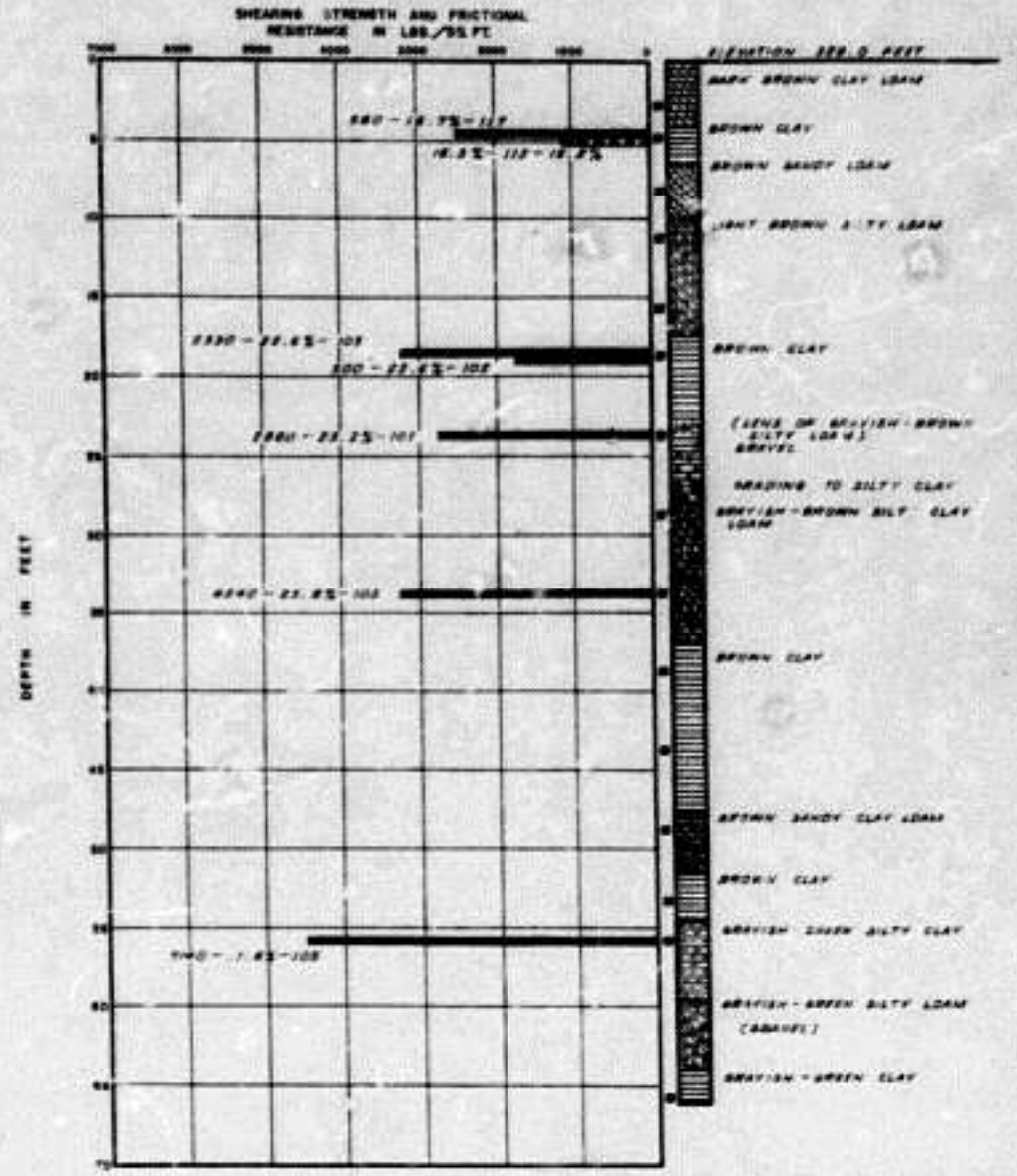


LOG OF BORINGS

DAMES & MOORE, Soil Engineers

PLATE A-18

BORING 5

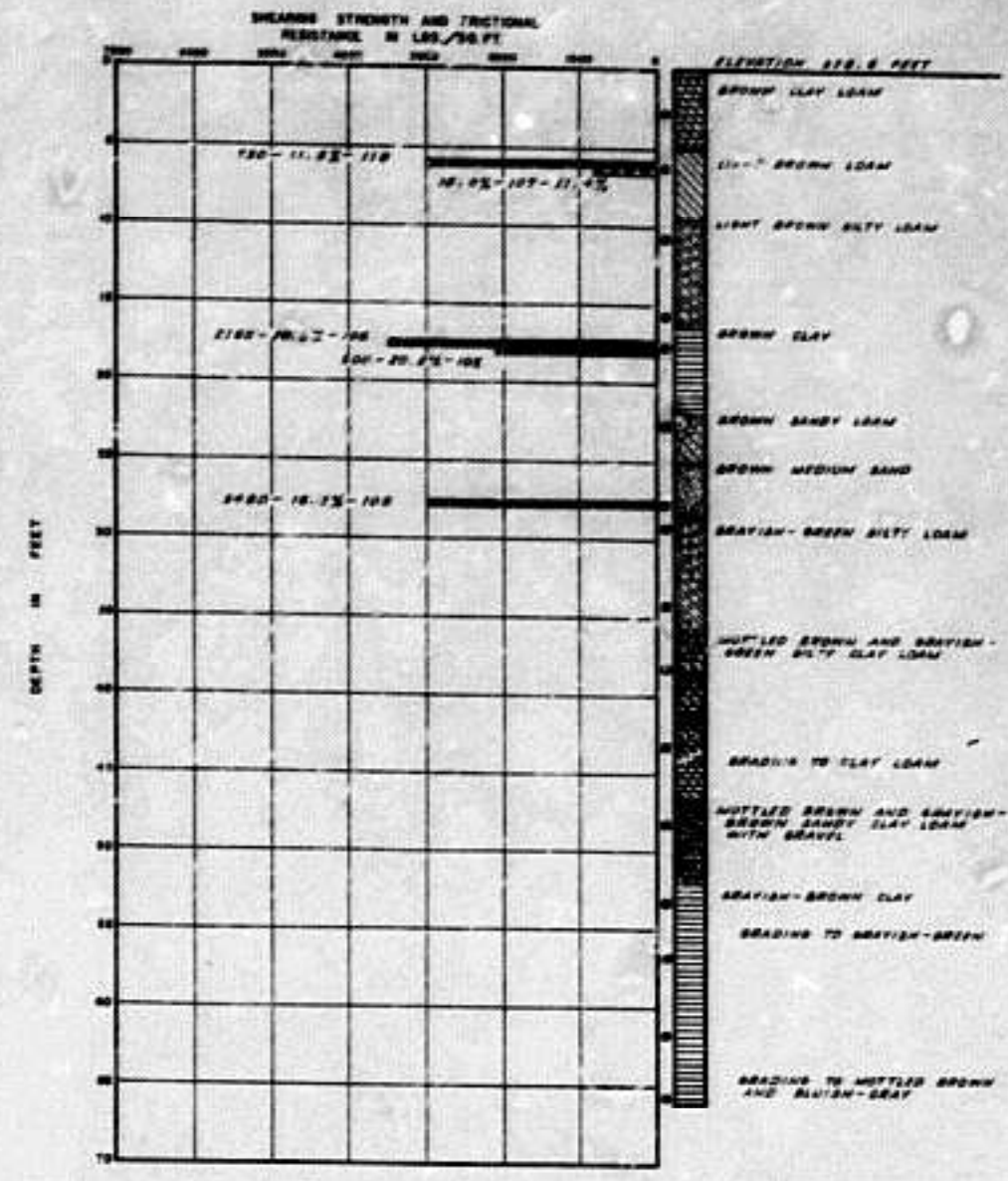


LOG OF BORINGS

DAMES & MOORE, Soil Engineers

PLATE A-15

BORING 6



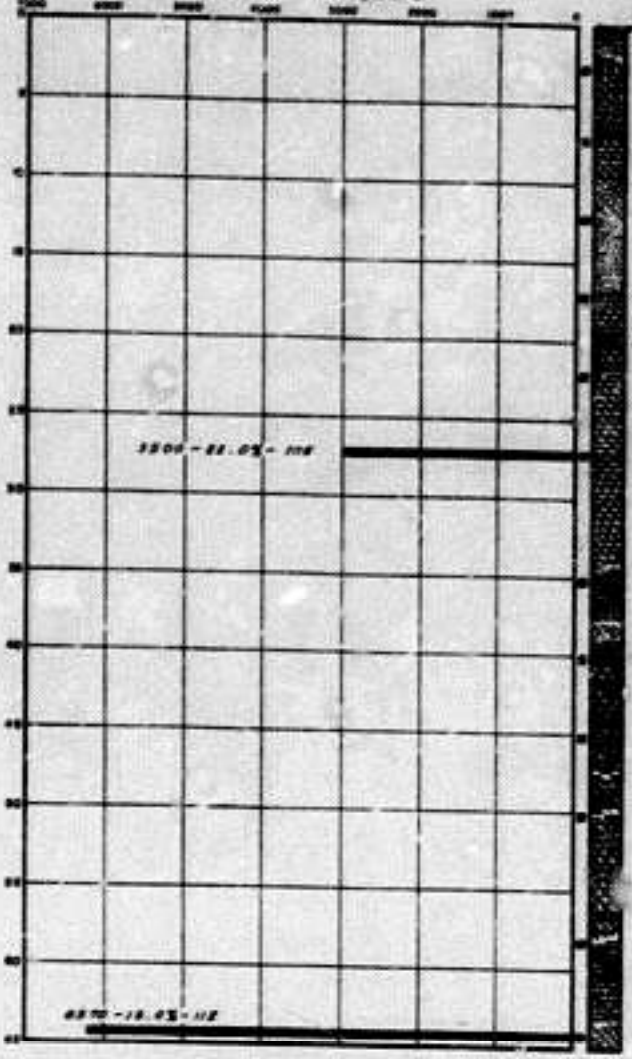
LOG OF BORINGS

DAMES & MOORE, Soil Engineers

PLATE A-17

BORING 7

SHEARING STRENGTH AND FRICTIONAL RESISTANCE IN LBS./SQ.FT.



BROWN CLAY LOAM

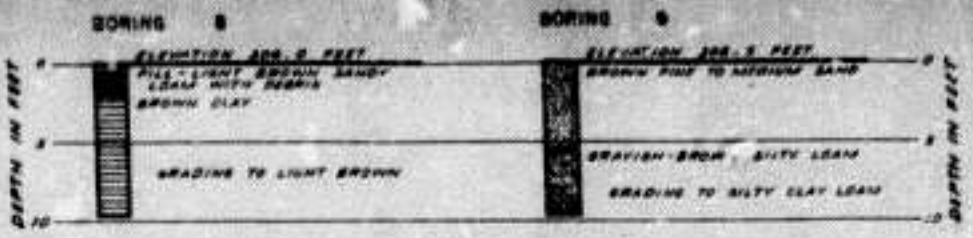
BROWN SANDY LOAM

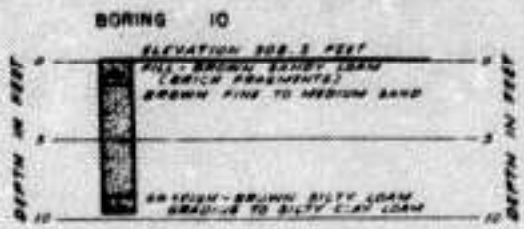
NOTE : WATER SEEPAGE AT 10 FEET.

LOG OF BORINGS

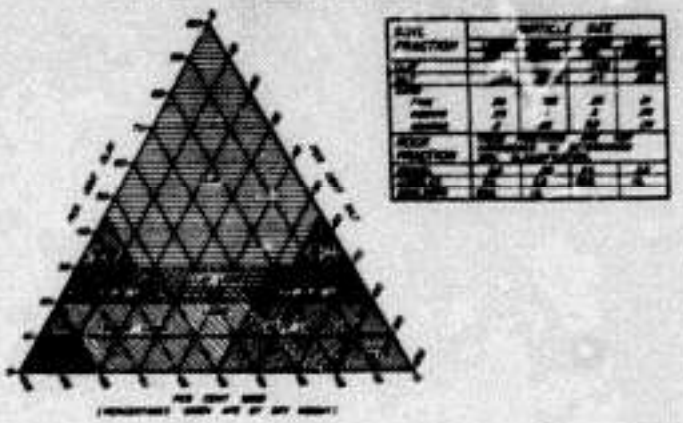
DAMES & MOORE, Soil Engineers

PLATE A-10

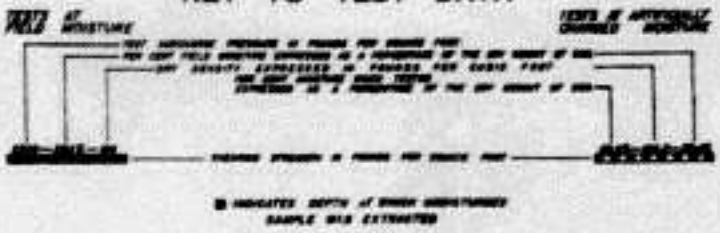




SOIL CLASSIFICATION CHART



KEY TO TEST DATA

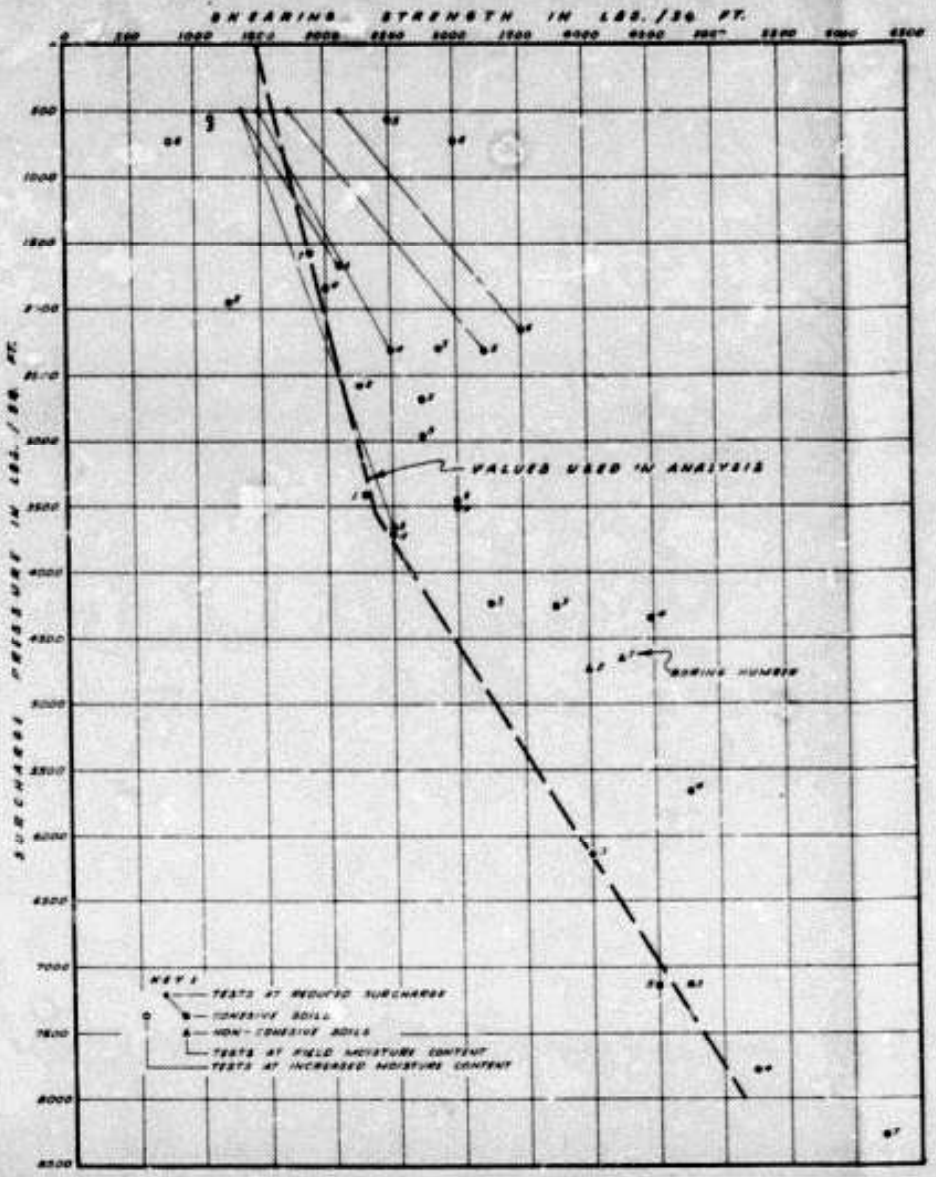


LOG OF BORINGS

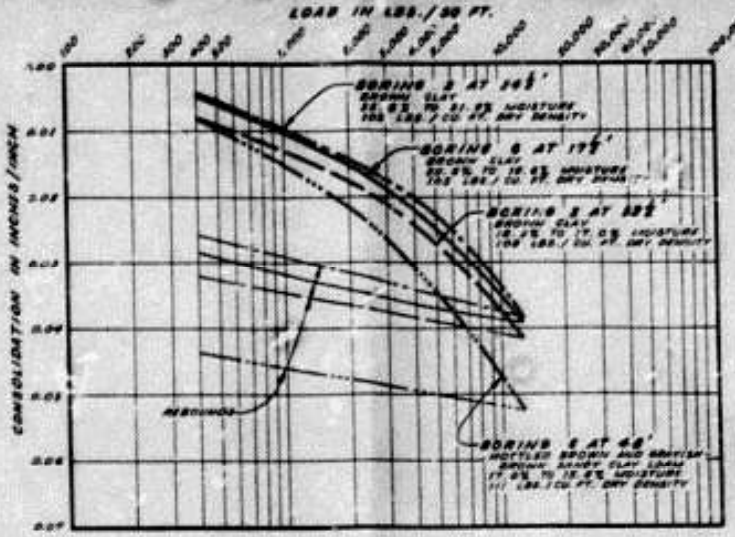
DAMES & MOORE, Soil Engineers

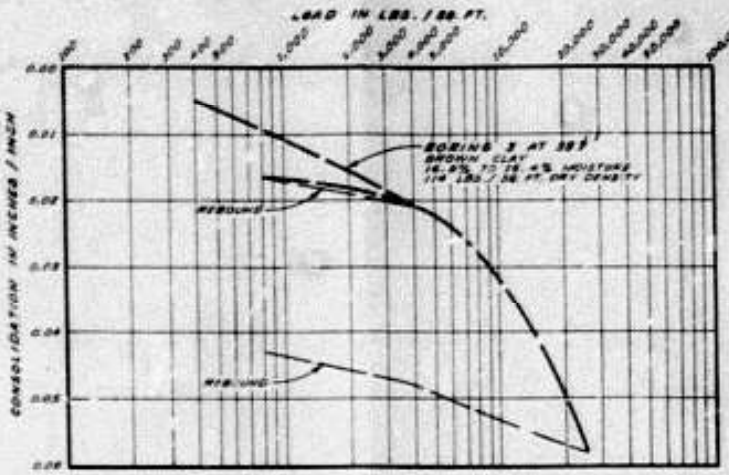
PLATE A-1H

SHEAR TEST DATA



CONSOLIDATION TEST DATA





SUMMARY OF TEST DATA

DAMES & MOORE, Soil Engineers

PLATE A-2

This appendix describes the geotechnical boring program conducted by AECOM for the proposed Women's and Children's Hospital project (Project) within the LAC+USC Medical Center, located at 1240 N Mission Rd, Los Angeles, CA 90033. The exploratory boring locations were first marked in the field and checked through Underground Services Alert (USA) for clearance of potential conflicts with underground utilities. When necessary, the marked locations were adjusted in order to avoid encountering underground utilities.
Our original exploratory boring program was initiated on January 28, 2019 and completed on February 7, 2019 under the supervision of an AECOM engineer.

Subsurface exploration included drilling and sampling ten borings to a maximum depth of 80 feet below the existing ground surface by our subcontractor, , 2R Drilling Inc. of Chino, California and SoCal Drilling of La Habra, California. Nine borings were drilled using a truck-mounted CME-75 drill rig with hollow stem auger and one boring was drilled using a truck-mounted mud-rotary drill rig. The borings were backfilled with cement-bentonite grout and the surrounding ground surface was reinstated following borehole completion. The locations of the borings are shown on Figure 4.

AECOM geotechnical representative maintained field boring logs and visually classified the soils according to the Unified Soil Classification System. The boring logs and other pertinent information are presented in following pages. When subsurface conditions permitted, relatively undisturbed samples were recovered with the Modified California Soil Sampler (with 2.42-inch I.D.). Standard Penetration Testing (SPT) and testing with the Modified California Soil Sampler were performed in the subsurface soils using a 140-pound automatic-tripped hammer with a 30-inch drop. The number of blows required to drive the sampler was recorded at 6-inch intervals for each sample taken. SPT was performed in accordance with ASTM D1586 procedures. The total number of blows required to drive the sampler every 6 inches is recorded on the boring logs presented in this Appendix.

Geotechnical samples obtained in the field were carefully sealed and packaged to reduce moisture loss and disturbance and were transported to our laboratory for further testing.

The blow count for the final 12 inches of sampler penetration is commonly referred to as the "N Value". This value generally reflects the resistance to penetration of the soil at the sample depth. The degree of relative density of granular soils and the degree of consistency of cohesive soils are generally described on the boring logs according to the conventional correlation presented in the following table.

The relative density and consistency descriptions on the attached boring log are based on unadjusted blow counts recorded in the field. These numbers are considered useful in providing an estimate of the relative density or consistency of soils. The relative density and consistency descriptions on the log may deviate from the correlation for a number of reasons, including reliance on other test results or the engineer's judgment based on manual manipulation of the sample.

Granular Soils			Cohesive Soils	
SPT Blow Count	**Description**		**SPT Blow Count**	**Description**
< 4	Very Loose		< 2	Very Soft
4 – 10	Loose		2 – 4	Soft
11 – 30	Medium Dense		5 – 8	Firm
31 – 50	Dense		9 – 15	Stiff
> 50	Very Dense		16 - 30	Very Stiff
			> 30	Hard

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	*GRAVEL AND GRAVELLY SOILS* MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	*CLEAN GRAVELS* (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING NO. 4 SIEVE	*CLEAN SANDS* (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	*SILTS AND CLAYS*	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: Dual symbols are used to indicate gravels or sand with 5-12% fines and soils with fines classifying as CL-ML. Symbols separated by a slash indicate borderline soil classifications.

Other Material Symbols

Concrete Asphalt Artificial Fill [AF]

Sampler and Symbol Descriptions

- Modified California Sample
- Standard Penetration Test
- No Recovery
- Bk Bulk sample
- Disturbed Modified California Sample
- Pitcher Tube Sample
- Shelby Tube Sample
- Rock Core Sample
- Approximate depth of perched water or groundwater

Note: Number of blows required to advance driven sample 12" (or length noted) is recorded; blow count recorded for seating interval (initial 6" of drive) is indicated by an asterisk.

Laboratory and Field Test Abbreviations

CBR	California Bearing Ratio Test
COL	Collapse Potential test (test result in parentheses)
COMP	Compaction test
CON	Consolidation test
CORR	Corrosivity test
DSCD	Consolidated drained direct shear test (normal pressure and shear strength results shown)
EI	Expansion Index test (test result in parentheses)
LL=29	Liquid limit (Atterberg limits test)
PI=11	Plasticity Index (Atterberg limits test)
PP	Pocket Penetrometer test
R-Value	Resistance Value test
SA	Sieve Analysis (-200 result in parentheses)
SE	Sand Equivalent test (test result in parentheses)
SWELL	Swell Load test (test result in parentheses)
TXCU	Triaxial Compression Test, Consolidated Undrained
-200	Percent passing #200 sieve (test result in parentheses)

KEY TO LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Date(s) Drilled	1/24/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Not Encountered		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,703.3 East 1,845,147.4

Boring B-19-01

Sheet 1 of 1

Job Number	60595699
Total Depth Drilled (ft)	21.5
Approximate Ground Surface Elevation (ft)	331.0 MSL

Elevation (ft)	Depth (ft)	Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	0							2.5-inch thick ASPHALT CONCRETE							
330			B-1				CL	4-inch thick BASE **FILL:** **Lean CLAY with SAND** dark yellowish brown			73				
	5		S-1 S-2	16	5 7 9			hard, dark brown, moist, with fine to coarse GRAVEL	14						
							SM	**ALLUVIUM:** **Silty SAND** medium dense, dark yellowish brown, moist, fine grained							
							SC	**Clayey SAND** dense, olive brown, moist, fine grained							
	10		S-3	13	3 5 8				16	112					DSCD
320							ML	**SILT** stiff, olive brown, moist, little CLAY and fine SAND							
	15		S-4	4	1 2 2										
	20		S-5	10	3 4 6				21	103					
310								Bottom of borehole at 21.5 feet Borehole backfilled with cement bentonite grout and soil cuttings with concrete cap to exisiting grade. Groundwater was not encountered.							
	25														
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	1/23/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Groundwater encountered at 32 feet below ground surface		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,695.3 East 1,845,069.5

Boring B-19-02

Sheet 1 of 2

Job Number	60595699
Total Depth Drilled (ft)	61.5
Approximate Ground Surface Elevation (ft)	330.5 MSL

Elevation (ft)	Depth (ft)	Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
330	0							1-inch thick ASPHALT CONCRETE							
			B-1				CL	7.5-inch thick BASE **FILL:** **Lean CLAY with SAND** dark brown with yellowish brown mottling, moist, fine grained			77				
	5		S-1	36	9 15 21			stiff	15	117					
							SM	**Silty SAND** dense, dark yellowish brown, moist, fine grained							
320	10		S-2	22	8 11 11		ML	**SILT with Sand** stiff, light olive brown, moist, fine grained							
	15		S-3	56	14 23 33			some CLAY	23	103					
							CL	**ALLUVIUM:** **Lean CLAY** hard, olive brown, moist, some very fine grained SAND			95				
310	20		S-4 S-5	22	7 9 13										
							CH	**Fat CLAY** hard, olive brown, moist, little fine SAND, high plasticity							
	25		S-6	42	10 18 24				28	97		61	41		
							SC	**Clayey SAND** medium dense, olive brown, moist, fine grained							
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

LAC+USC Medical Center

FOR: Los Angeles County, Department of Public Works

Boring B-19-02

Sheet 2 of 2

Elevation (ft)	Depth (ft)	Sample Type	Sample Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	30	S	S-7	18	4 7 11		SC	**Clayey SAND***(continued)* medium dense, olive brown, moist, fine grained							
-300							SM	**Silty SAND** dense, olive brown, moist, fine grained							
	35	S	S-8	39	13 14 25		CL	**Lean CLAY with SAND** olive brown, moist, fine grained	22	106	87				
	40	S	S-9	31	7 13 18			light olive brown, medium to fine grained							
-290							SM	**Silty SAND** dense, light olive brown, wet, fine to medium grained, some coarse, trace fines							
	45	S	S-10	42	10 18 24				19	113					
							CL	**Sandy Lean CLAY** hard, olive, moist, little coarse GRAVEL							
-280	50	S	S-11	18	4 7 11										
	55	S	S-12	39	13 14 25			some fine SAND							
	60	S	S-13	31	7 13 18			trace fine SAND	23						
-270								Bottom of borehole at 61.5 feet Borehole backfilled with cement bentonite grout to 5 feet above GW, and soil cuttings with concrete cap to existing grade							
	65														

LOG OF BORING

LAC+USC WC Hospital Demolition

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	1/23/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Groundwater encountered at 38 feet below ground surface		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,762.6 East 1,844,944.2

Boring B-19-03

Sheet 1 of 2

Job Number	60595699
Total Depth Drilled (ft)	41.5
Approximate Ground Surface Elevation (ft)	322.5 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	0							3-inch thick ASPHALT CONCRETE							
								9.5-inch thick BASE							
			B-1				CL	**FILL:** **Lean CLAY with SAND** stiff, dark yellowish brown, moist			74				
320															
			S-1	21	5 8 13			becomes hard							
	5														
							SC	**Clayey SAND** dense, dark yellowish brown, moist							
	10		S-2	100+	14 40 50/5"				10	128					
310							SM	**Silty SAND** dense, light olive brown, moist, fine grained, trace fines							
	15		S-3	28	9 12 16			some CLAY							
	20		S-4 S-5	41	20 17 24				16	118	22				
300							CL	**Lean CLAY** hard, olive brown, moist							
							SC	**ALLUVIUM:** **Clayey SAND** dense, light olive brown, moist, fine grained							
	25		S-6 S-7	23	5 9 14				16						
							CL	**Lean CLAY** olive brown, moist							
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

LAC+USC Medical Center

FOR: Los Angeles County, Department of Public Works

Boring B-19-03

Sheet 2 of 2

Elevation (ft)	Depth (ft)	SAMPLES: Type	SAMPLES: Number	SAMPLES: Blows per foot	SAMPLES: Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	30	S	S-8	38	12 16 22		CL	**Lean CLAY with SAND** *(continued)* olive brown, moist	19	111	77	47	33		
290															
	35	S	S-9 S-10	21	6 9 12		SC	**Clayey SAND** hard, dark yellowish brown, moist, fine grained							
	40	S	S-11	43	14 18 25		CL	**Lean CLAY** hard, olive brown, moist	26	100					
280								Bottom of borehole at 41.5 feet Borehole backfilled with cement bentonite grout to 5 feet above GW, and soil cuttings with concrete cap to existing grade							
	45														
	50														
270															
	55														
	60														
260															
	65														

LOG OF BORING

LAC+USC WC Hospital Demolition

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	1/23/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Not Encountered		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,906.2 East 1,844,843.1

Boring B-19-04

Sheet 1 of 1

Job Number	60595699
Total Depth Drilled (ft)	21.5
Approximate Ground Surface Elevation (ft)	318.5 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	0							5-inch thick ASPHALT CONCRETE							CORR (0-5')
			B-1				SC	5.5-inch thick BASE			78				
								FILL: **Lean CLAY with SAND** dark yellowish brown, moist, fine grained							
	5		S-1	32	9 14 18			stiff	13	118					El(101)
310							CL	**ALLUVIUM:** **Lean CLAY** hard, very dark brown, moist, little GRAVEL and trace fine SAND							
	10		S-2 S-3	12	3 5 7										
	15		S-4	22	6 10 12			stiff, light olive brown, moist, little fine SAND and GRAVEL	19	111					
300															
	20		S-5	13	4 6 7			light olive brown with yellowish brown mottling							
								Bottom of borehole at 21.5 feet Borehole backfilled with cement bentonite grout and soil cuttings with concrete cap to exisiting grade. Groundwater was not encountered.							
	25														
290															
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	2/7/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Groundwater encountered at 33 feet below ground surface		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,809.7 East 1,844,729.0

Boring B-19-05

Sheet 1 of 3

Job Number	60595699
Total Depth Drilled (ft)	81.0
Approximate Ground Surface Elevation (ft)	320.0 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
320	0							8-inch thick ASPHALT CONCRETE 7-inch thick BASE							
							SC	**FILL:** **Clayey SAND** dark brown, moist, fine grained							
	5	S-1		14	3 6 8				23	106					
310	10	S-2		3	1 1 2		CL	**ALLUVIUM:** **Lean CLAY with SAND** stiff, olive brown, moist, fine grained	24						
	15	S-3		23	6 11 12				26	100	84	44	31		CON
300	20	S-4		9	3 4 5										SE
	25	S-5		25	6 11 14			becomes very dark grey (5Y, 3/1)	27	97	89				DSCD
290	30						SC	**Clayey SAND** con't							

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	SAMPLES Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
290	30		S-6	37	14 18 19		SC	**Clayey SAND***(continued)* dark greenish gray, moist, fine grained, little clay							
							SM	**Silty SAND** olive gray, moist, fined grained, trace silt							
	35		S-7	64	17 25 39				23						
							SC	**Clayey SAND** dense, olive gray, moist, fine grained							
280	40		S-8	42	9 20 22										
	45			100+	50/6"		SM	**Silty SAND** dense, olive grey, wet							
270	50		S-9	45	21 22 23				20						
							SC	**Clayey SAND** medium dense, olive gray, moist, fine grained, little gravel							
							CL	**Silty CLAY with SAND** hard, dark grey, moist							
	55		S-11	77	16 35 42				22	103	89	75	57		CON
260	60		S-12	63	18 30 33										
							SC	**Clayey SAND** dense, dark olive brown, moist, fine grained							
	65		S-13	100+	27 50/5"				23	104					

LOG OF BORING

LAC+USC WC Hospital Demolition

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

LAC+USC Medical Center

FOR: Los Angeles County, Department of Public Works

Boring B-19-05

Sheet 3 of 3

Elevation (ft)	Depth (ft)	SAMPLES Type Number	SAMPLES Blows per foot	SAMPLES Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
						SC	**Clayey SAND***(continued)* dense, dark olive brown, moist, fine grained							
250	70	S-14	100+	25 53/8"		ML	**BEDROCK (PUENTE FORMATION):** **Sandy SILT** hard, dark greyish brown, moist, fine gained							
						MLG	**Gravely SILT** hard, dark grayish brown, moist, fine grained, trace of GRAVEL							
	75	S-15	100+	50/25"		ML	**Sandy SILT** hard, dark greyish brown, moist, fine gained	44						
240	80	S-16	100+	30 54/6"										
							Bottom of borehole at 81.0 feet Borehole backfilled with cement bentonite grout to 5 feet above groundwater, and soil cuttings with concrete cap to existing grade							
	85													
230	90													
	95													
220	100													
	105													

LOG OF BORING

LAC+USC WC Hospital Demolition

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	1/23/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Not Encountered		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,532.5 East 1,844,832.2

Boring B-19-06

Sheet 1 of 1

Job Number	60595699
Total Depth Drilled (ft)	20.9
Approximate Ground Surface Elevation (ft)	332.5 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	0		B-1				CL	**FILL:** **Lean CLAY with SAND** very dark brown, moist, little fine SAND and GRAVEL	17						
330															
	5		S-1 S-2	19	6 8 11		CLS	**Sandy Lean CLAY** hard, yellowish brown, dry, with fine SAND			83				
							ML	**SILT** hard, light yellowish brown, moist, with little CLAY							
320	10		S-3	100+	30 50/4"				13	111					
	15		S-4	30	15 13 17		SC	**Clayey SAND** dense, dark yellowish brown, moist, fine grained							
	20		S-5	100+	28 50/5"		CL	**Lean CLAY** dark yellowish brown, moist	10	123					DSCD
310								Bottom of borehole at 21.5 feet Borehole backfilled with cement bentonite grout and soil cuttings with concrete cap to exisiting grade. Groundwater was not encountered.							
	25														
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	1/23/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Not Encountered		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,457.3 East 1,844,944.1

Boring B-19-07

Sheet 1 of 2

Job Number	60595699
Total Depth Drilled (ft)	41.5
Approximate Ground Surface Elevation (ft)	333.0 MSL

Elevation (ft)	Depth (ft)	Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	0							4-inch thick ASPHALT CONCRETE							
								5-inch thick BASE							
330		B	1				CL	**FILL:** **Lean CLAY with SAND** very dark brown, moist, little fine SAND			73	36	22		
	5			46	11 20 26			hard	16	117					
		S	1				SC	**Clayey SAND** dense, dark yellowish brown, moist							
							SM	**Silty SAND** dense, dark yellow, moist							
	10	S	2	23	9 10 13										
		S	3				ML	**SILT** hard, light olive brown, moist, few fine grained SAND							
320															
	15	S	4	100+	23 50/5"				13	115	89				
							SC	**Clayey SAND** dense, dark yellowish brown, moist, fine grained, fines decrease with depth							
	20	S	5	19	7 9 10										
310							SM	**Silty SAND** dense, light olive brown, moist, little SILT							
	25	S	6	100+	50/5"				8	116					
							ML	**ALLUVIUM:** **SILT** olive, moist, little fine SAND							
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

LAC+USC Medical Center

FOR: Los Angeles County, Department of Public Works

Boring B-19-07

Sheet 2 of 2

Elevation (ft)	Depth (ft)	Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	30		S-7	18	5 7 11		ML	**SILT** *(continued)* very stiff, olive, moist, little fine SAND							
300							CL	**Lean CLAY** hard, olive brown, moist, some SILT and fine SAND							
	35		S-8	50	12 20 30				23	102					DSCD
	40		S-9	19	5 8 11			dark yellowish brown							
290								Bottom of borehole at 41.5 feet Borehole backfilled with cement bentonite grout and soil cuttings with concrete cap to exisiting grade. Groundwater was not encountered.							
	45														
	50														
280															
	55														
	60														
270															
	65														

LOG OF BORING

LAC+USC WC Hospital Demolition

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	1/23/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Not Encountered		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,591.1 East 1,845,058.5

Boring B-19-08

Sheet 1 of 1

Job Number	60595699
Total Depth Drilled (ft)	21.5
Approximate Ground Surface Elevation (ft)	332.5 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	0							4-inch thick ASPHALT CONCRETE							
330			B-1				CL	5-inch thick BASE **FILL:** **Lean CLAY with SAND** dark brown, moist, fine grained SAND							
	5		S-1	29	6 12 17			hard, dark yellowish brown	15		76				
	10		S-2 S-3	68	18 28 40				14	117					El(111)
320							ML	**SILT** hard, light yellowish brown, moist, with some CLAY							
	15		S-4	35	11 16 19										
							CL	**Sandy Lean CLAY** hard, dark brown, moist, some fine grained							
	20		S-5	51	12 21 30				20	108					
310								Bottom of borehole at 21.5 feet Borehole backfilled with cement bentonite grout and soil cuttings with concrete cap to exisiting grade. Groundwater was not encountered.							
	25														
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	2/6/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Not Encountered		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,524.5 East 1,845,058.6

Boring B-19-09

Sheet 1 of 1

Job Number	60595699
Total Depth Drilled (ft)	11.5
Approximate Ground Surface Elevation (ft)	333.0 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	0							3-inch thick ASPHALT CONCRETE							
			B-1				SC	6-inch thick BASE	15						R-VALUE
330								**FILL:** **Clayey SAND** dense, dark yellowish brown, moist, fine grained							
	5		S-1	39	10 16 23										
							ML	**SILT** hard, pale olive, moist, fine grained, trace SAND							
	10		S-2	18	8 9 9										CORR
320								Bottom of borehole at 11.5 feet Borehole backfilled with filter sand and soil cuttings with concrete cap to exisiting grade. Groundwater was not encountered.							
	15														
	20														
310															
	25														
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	2/6/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Not Encountered		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,681.0 East 1,844,753.3

Boring B-19-10

Sheet 1 of 1

Job Number	60595699
Total Depth Drilled (ft)	17.5
Approximate Ground Surface Elevation (ft)	333.0 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	0		B-1				SM	**FILL:** **Silty SAND** dense, olive brown, moist, fine grained, little CLAY							
330															
	5		S-1	19	5 8 11										
							SC	**Clayey SAND** dense, olive brown, moist, fine grained							
	10		S-2	11	4 5 6										
320															
	15		S-3	16	4 10 6										
								Bottom of borehole at 17.5 feet Borehole backfilled with filter sand and soil cuttings with concrete cap to exisiting grade. Groundwater was not encountered.							
	20														
310															
	25														
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Field infiltration test was conducted at the representative locations determined jointly by the Project Civil Engineer and AECOM, and was performed at upper alluvium materials, which were considered the target depths favorable for the site infiltration. Details of the test procedures and results are provided in the following sections.

TEST HOLE PREPARATION

Upon completion of the drilling activities, the test locations for both borings were prepared by installing a 3-inch diameter PVC casing (combination of blank pipe and slotted pipe with solid end cap at the bottom) to facilitate in-hole infiltration test. The annular space between the pipe and drilled hole was filled with sand up to the bottom of the blank pipe then filled with bentonite chips to the existing ground surface. Test holes were pre-soaked prior to testing.

IN-SITU PERCOLATION TESTING

Base on observed water flow rate into the test holes during presoaking, a 30-minute time interval was chosen as the standard time interval to measure the water drop readings for this percolation test program per GS200.2 guidelines. Upon completion of the presoak, drop in water level in the test hole and time were recorded in accordance with GS200.2 guidelines. During testing, the test holes were refilled with water to the depth of soil to be tested. Recording the drops of water level was repeated until a stabilized rate of water drop was obtained during percolation testing. Data recorded during the tests included drawdown at given interval (approximately 30 minutes). The infiltration test results are summarized in the following pages.

The data presented herein is based on subsurface conditions encountered at our test locations only. The data should not be extrapolated to other areas, or used for other facilities without our prior review.

AECOM

Boring/Excavation Percolation Testing Field Log

Date 2/7/2019

Project Location	LAC+USC WC Demolition	**Boring/Test Number**	B-19-09
Earth Description	Silt (ML)	**Diameter of Boring (in)**	8
Tested by	M. Black	**Diameter of Casing (in)**	3
Liquid Description	Water	**Depth of Boring (ft)**	30
Measurement Method	Water Level Meter	**Depth to Invert of BMP (ft)**	5
		Depth to Initial Water Depth (d_1) (ft)	8.8
		Depth to Original Water Table (ft)	N/A
		Screen Depths (ft) Top	6
		Bottom	11
Time Interval Standard			
Start Time for Pre-Soak	10:53:00 AM	**Water Remaining In Boring (Yes/No)**	Yes
Start Time for Standard	12:52:00 PM	**Standard Time Interval Between Readings**	30-min

Reading Number	Time Start / End (hh:mm)	Elasped Time Δtime (mins)	Water Drop During Standard Time Interval Δd (inches)	Percolation Rate for Reading (in/hr)	Soil Description/Notes/ Comments
1	12:52:00 PM 1:22:00 PM	30	41.76	2.79	
2	1:22:00 PM 1:52:00 PM	30	39.84	2.73	
3	1:52:00 PM 2:22:00 PM	30	40.44	2.75	
4	2:22:00 PM 2:52:00 PM	30	40.20	2.74	
5	2:52:00 PM 3:22:00 PM	30	39.96	2.73	
6	3:22:00 PM 4:10:00 PM	30	39.96	2.73	
7	3:52:00 PM 4:22:00 PM	30	39.84	2.73	
8	4:22:00 PM 4:52:00 PM	30	39.72	2.72	

d_1 = Initial water depth (in.)
Δd = Water drop of final period (in.)
DIA = Diameter of boring (in.)

Measure Percolation Rate = 2.72 in/hr

AECOM

Boring/Excavation Percolation Testing Field Log

Date 3/1/2019

Project Location	LAC+USC WC Demolition	**Boring/Test Number**	B-19-10
Earth Description	Clayey Sand (SC)	**Diameter of Boring (in)**	8
Tested by	M. Black	**Diameter of Casing (in)**	2
Liquid Description	Water	**Depth of Boring (ft)**	20
Measurement Method	Water Level Meter	**Depth to Invert of BMP (ft)**	16.0
		Depth to Initial Water Depth (d_1) (ft)	8.8
		Depth to Original Water Table (ft)	N/A
		Screen Depths (ft) Top	11
		Bottom	16

Time Interval Standard			
Start Time for Pre-Soak	10:25:00 AM	**Water Remaining In Boring (Yes/No)**	Yes
Start Time for Standard	12:07:00 PM	**Standard Time Interval Between Readings**	30-min

Reading Number	Time Start / End (hh:mm)	Elasped Time Δtime (mins)	Water Drop During Standard Time Interval Δd (inches)	Percolation Rate for Reading (in/hr)	Soil Description/Notes/ Comments
1	12:07:00 PM 12:37:00 PM	30	7.20	2.64	
2	12:37:00 PM 1:07:00 PM	30	6.72	2.64	
3	1:07:00 PM 1:37:00 PM	30	6.48	2.64	
4	1:37:00 PM 2:07:00 PM	30	6.24	2.64	
5	2:07:00 PM 2:37:00 PM	30	6.12	2.64	
6	2:37:00 PM 4:10:00 PM	30	6.00	2.64	
7	3:07:00 PM 3:37:00 PM	30	6.00	2.64	
8	3:37:00 PM 4:07:00 PM	30	4.80	2.64	

d_1 = Initial water depth (in.)
Δd = Water drop of final period (in.)
DIA = Diameter of boring (in.)

Measure Percolation Rate = 2.64 in/hr

Geotechnical soil samples obtained from the borings were carefully packaged in the field to reduce moisture loss and disturbance. The samples were subsequently delivered to our laboratory where they were further examined and classified. Selected representative samples were tested to evaluate moisture content, in-situ dry density, grain-size distribution, plasticity index, shear strength, corrosivity, and R-value. Tests discussed below were performed in accordance with the latest American Society of Testing and Materials (ASTM) and California Test (CT) standards.

Moisture Content and In-situ Density (ASTM D2216 and D2937) Tests

The moisture content and in-situ density tests were performed on selected soil samples obtained from the borings. Moisture content and in-situ density tests were performed in accordance with ASTM Test Method D2216. The test results are presented on the logs of borings (Appendix B).

Percent Passing No. 200 Sieve

Percent passing no. 20 sieve were performed to determine the fines content of the representative soils. The tests were performed in accordance with ASTM Test Method D1140. The test results are presented on the logs of borings (Appendix B).

Sieve Analyses (ASTM D6913)

Sieve analyses were performed to determine the particle size distribution of representative soils. The tests were performed in accordance with ASTM Test Method D6913. Test results are presented in Appendix D-1 to D-7 (Grain Size Distribution).

Atterberg Limits Tests (ASTM D4318)

Atterberg Limits tests were performed to aid in classification and to evaluate the plasticity characteristics of fine-grained materials encountered in the borings. The tests were performed in accordance with ASTM Test Method D4318. The results of this test are presented in Appendix D-8 and D-11.

Direct Shear Test (ASTM D3080)

One consolidated-drained (saturated) direct shear test was performed on a selected undisturbed sample to evaluate shear-strength parameters of the on-site soils. The direct-shear test was performed in accordance with ASTM Test Method D3080. The results are presented in Appendix D-12 through D-15, respectively.

Compaction Test (ASTM D1557)

A compaction test was performed on a representative bulk sample of the near-surface soils in order to evaluate compaction characteristics of the soils. The test was performed in accordance with ASTM Test Method D1557. Test results are presented in Figures D-16 and D-21.

Consolidation Test (ASTM D2435)

One consolidation tests were performed to determine the magnitude and rate of volume decrease that a laterally confined soil specimen undergoes when subjected to different vertical pressures. The tests were performed in accordance with ASTM Test Method D2435. Test results are presented in Appendix D-22 and D-23.

Expansion Index (ASTM D4829)

Expansion index (EI) tests were performed on selected samples to evaluate expansion characteristics of the soils. The test was performed in accordance with ASTM D4829, and the result of the test is presented in Appendix D-24 and D-25.

Sand Equivalent (ASTM D2419)

Sand equivalent test were performed on selected samples to evaluate the relative abundance of sand versus clay in the soil. The test was performed in accordance with ASTM D2419, and the results of the test is presented in Appendix D-26 through D-27.

Corrosivity Tests (CT 417, CT 422, CT 643)

Selected representative samples obtained from the borings were tested for corrosion. The subject soil samples were processed with the U.S. Standard No. 8 Sieve and tested for pH (CT 643), Sulfate Ion Content (CT 417), Chloride Ion Content (CT 422), and resistivity (CT 643), the result of the test is presented in Appendix D-28 through D-29.

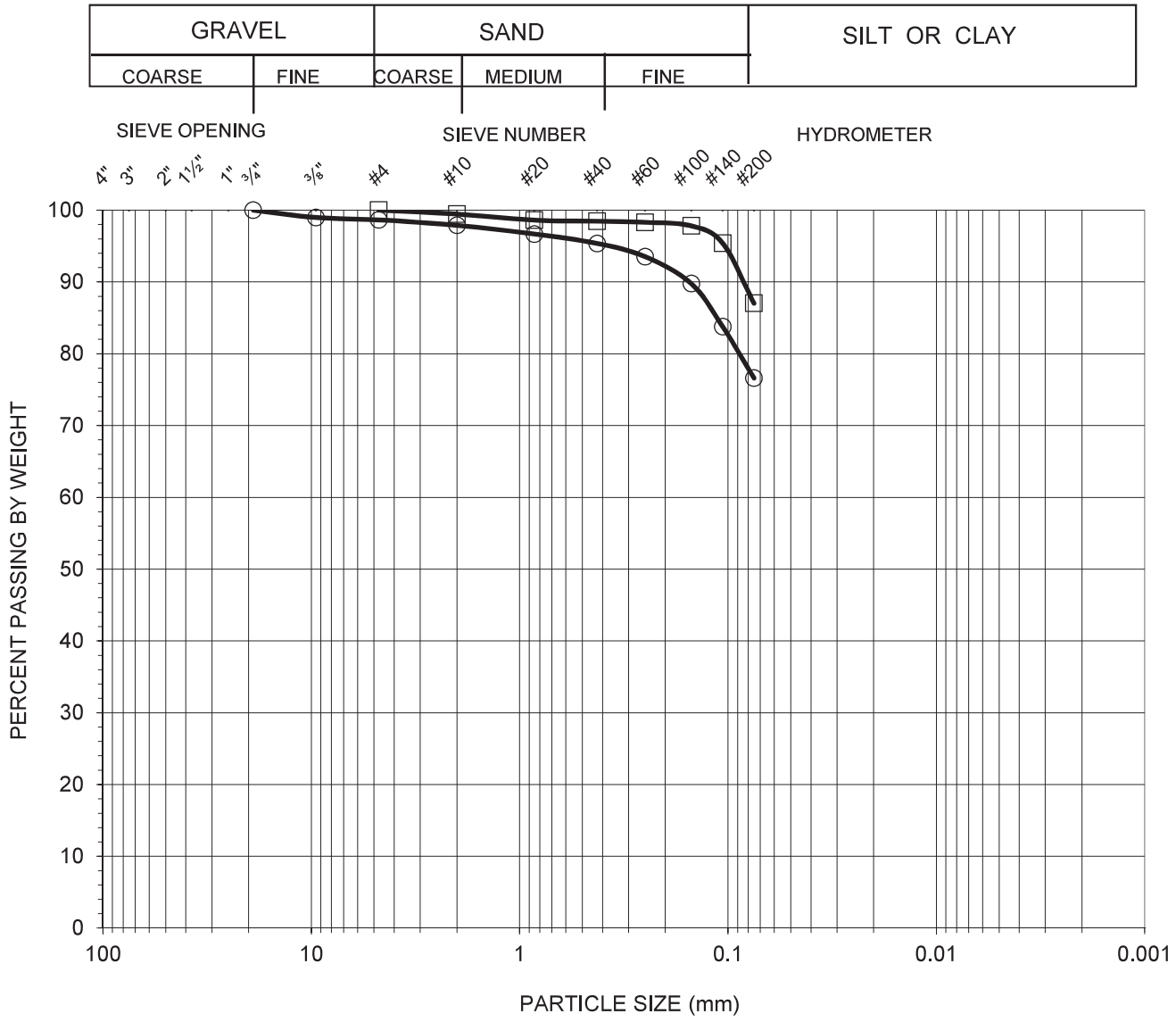
R-Value Test (ASTM D2844)

R-Value test were performed on selected samples to evaluate bearing characteristics of the soils. The test was performed in accordance with ASTM D2844, and the result of the test is presented in Appendix D-30.

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	AECOM	Tested by:	AM	Date:	02/19/19
Project Name:	LAC+USC WC Hospital Demolition	Computed by:	JP	Date:	02/19/19
Project Number:	60597019	Checked by:	AP	Date:	02/19/19



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	B-19-02	B-1	0-5	1	22	77	N/A	CL*
□	B-19-02	S-8	35	0	13	87	N/A	CL*

**Note: Based on visual classification of sample*

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	AECOM	Tested by:	AM	Date:	02/19/19
Project Name:	LAC+USC WC Hospital Demolition	Computed by:	JP	Date:	02/19/19
Project Number:	60597019	Checked by:	AP	Date:	02/19/19



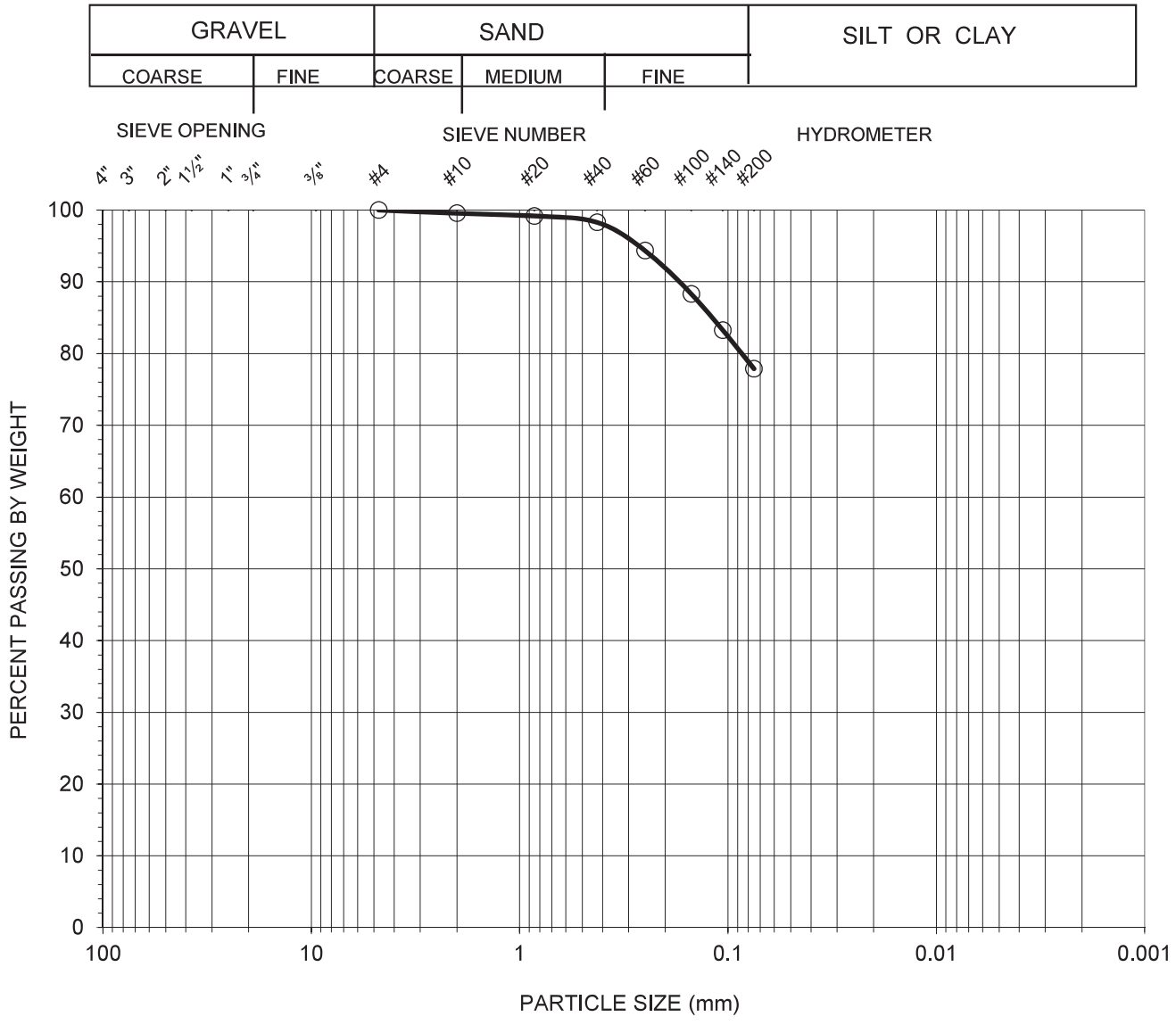
Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	B-19-03	B-1	0-5	7	19	74	N/A	CL*
□	B-19-03	S-4	20.5	0	78	22	N/A	SM

**Note: Based on visual classification of sample*

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	AECOM	Tested by:	AM	Date:	02/19/19
Project Name:	LAC+USC WC Hospital Demolition	Computed by:	JP	Date:	02/19/19
Project Number:	60597019	Checked by:	AP	Date:	02/19/19



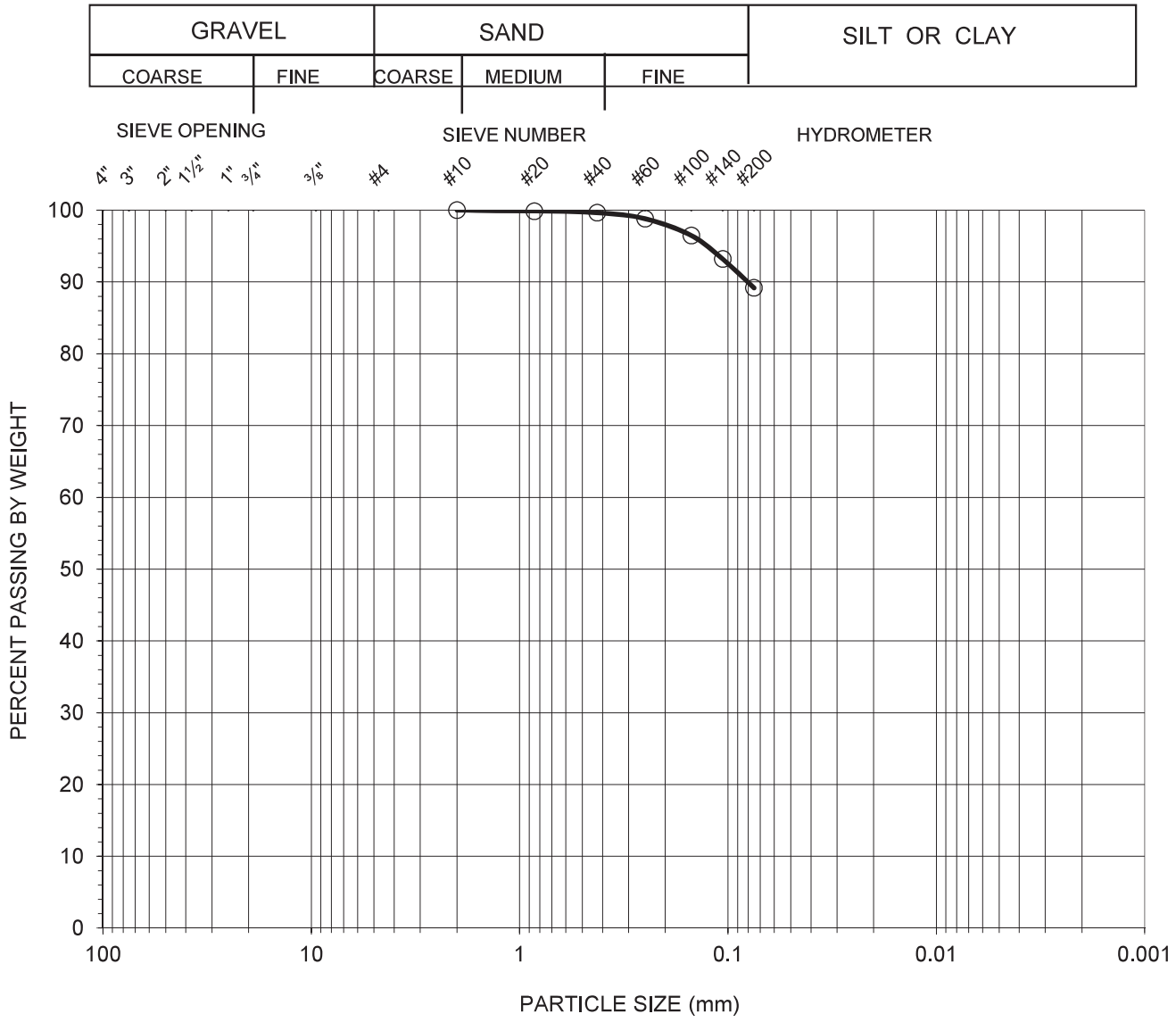
Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	B-19-04	B-1	0-5	0	22	78	N/A	CL*

**Note: Based on visual classification of sample*

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	AECOM	Tested by:	AM	Date:	02/19/19
Project Name:	LAC+USC WC Hospital Demolition	Computed by:	JP	Date:	02/19/19
Project Number:	60597019	Checked by:	AP	Date:	02/19/19



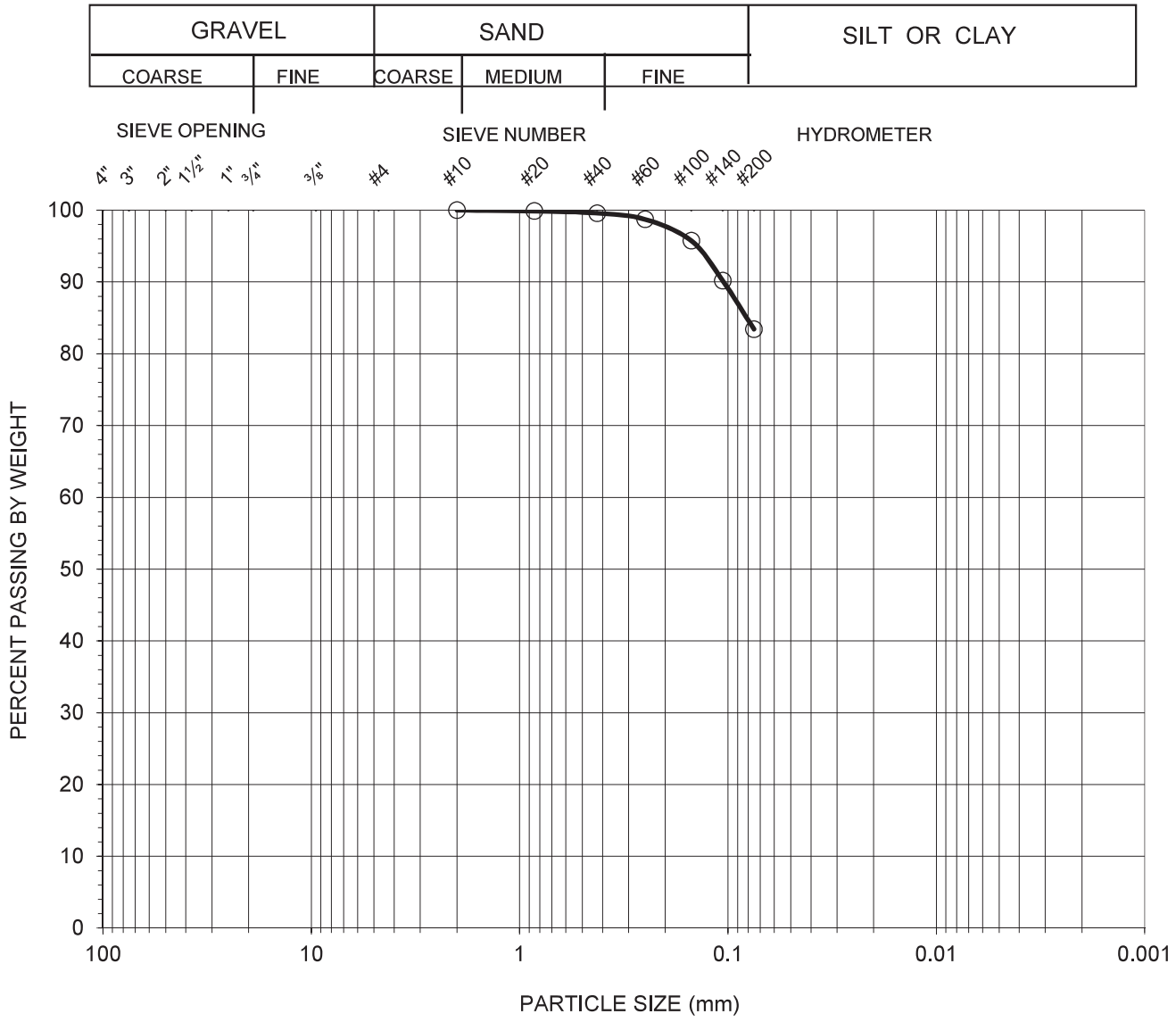
Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	B-19-05	S-5	25	0	11	89	N/A	CL*

**Note: Based on visual classification of sample*

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	AECOM	Tested by:	AM	Date:	02/19/19
Project Name:	LAC+USC WC Hospital Demolition	Computed by:	JP	Date:	02/19/19
Project Number:	60597019	Checked by:	AP	Date:	02/19/19



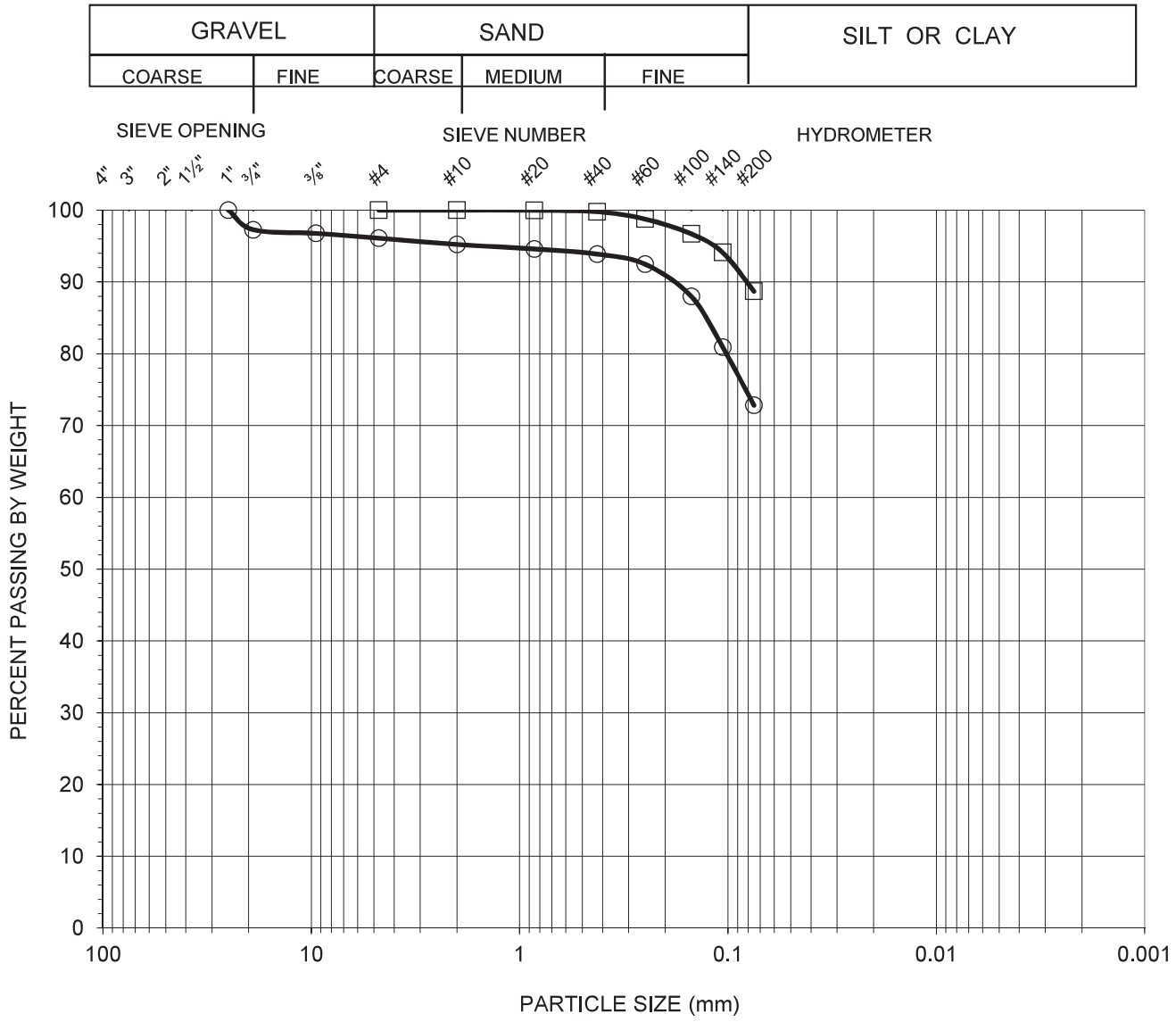
Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	B-19-06	S-1	5	0	17	83	N/A	CL*

**Note: Based on visual classification of sample*

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	AECOM	Tested by:	AM	Date:	02/19/19
Project Name:	LAC+USC WC Hospital Demolition	Computed by:	JP	Date:	02/19/19
Project Number:	60597019	Checked by:	AP	Date:	02/19/19



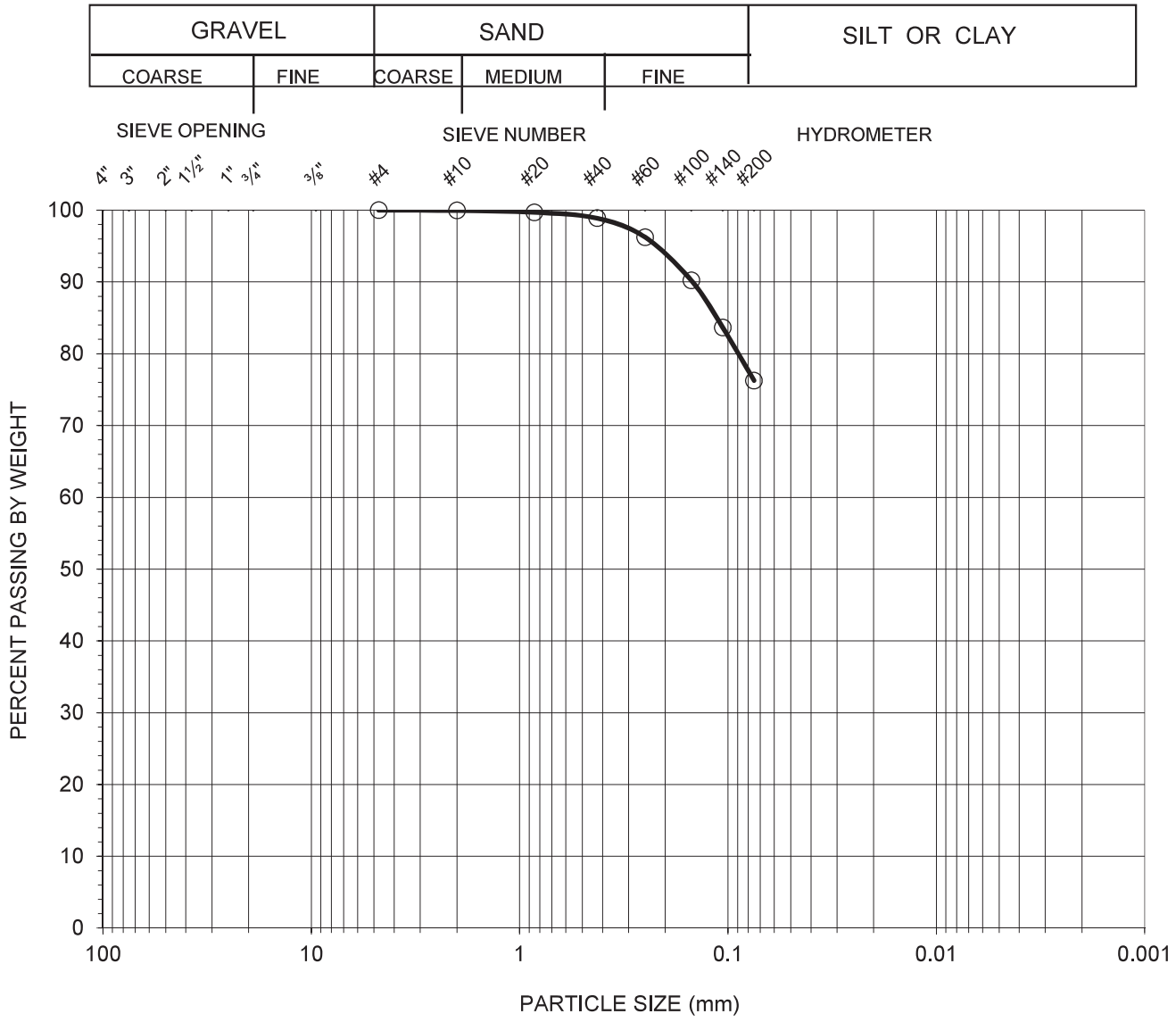
Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	B-19-07	B-1	0-5	4	23	73	36:14:22	CL
□	B-19-07	S-4	15	0	11	89	N/A	CL*

**Note: Based on visual classification of sample*

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	AECOM	Tested by:	AM	Date:	02/19/19
Project Name:	LAC+USC WC Hospital Demolition	Computed by:	JP	Date:	02/19/19
Project Number:	60597019	Checked by:	AP	Date:	02/19/19



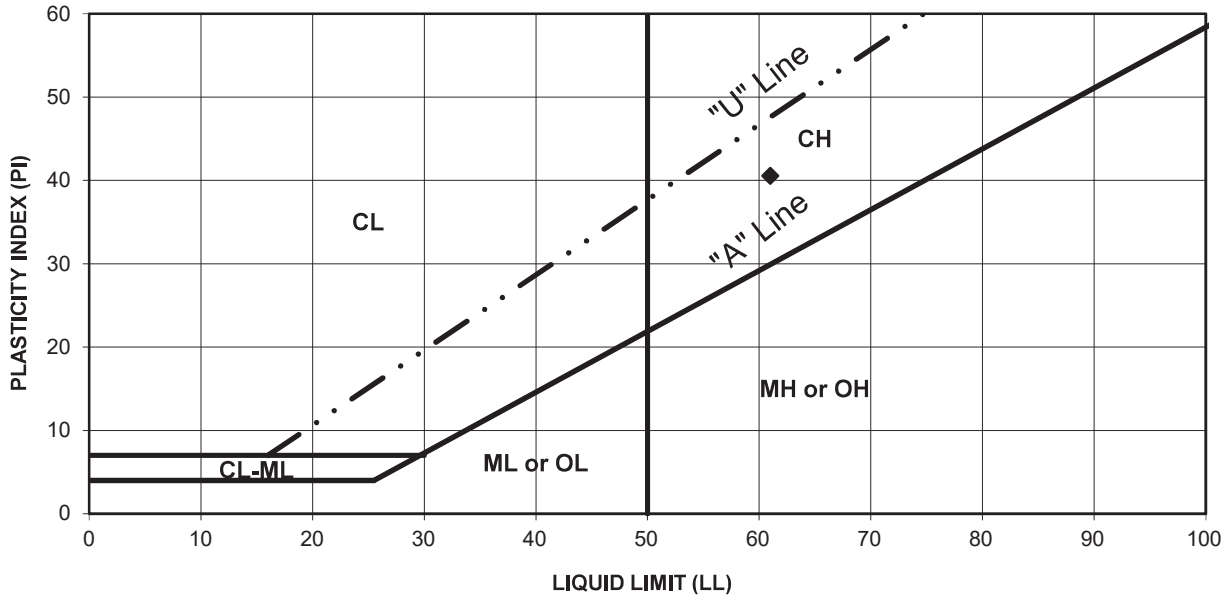
Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	B-19-08	S-1	5	0	24	76	N/A	CL*

**Note: Based on visual classification of sample*

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

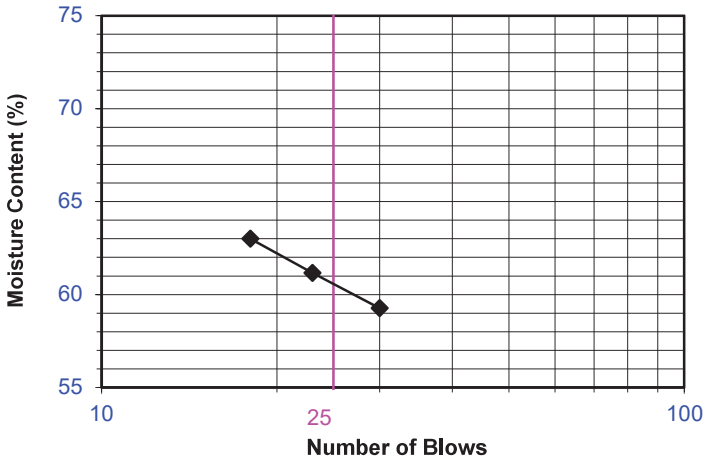
ATTERBERG LIMITS
ASTM D 4318

Project Name: LAC+USC WC Hospital Demolition **Tested By:** LS Date: 02/18/19
Project No.: 60597019 **Checked By:** AP Date: 02/19/19



PROCEDURE USED

- ☐ Wet Preparation
- ☒ Dry Preparation
- ☒ Procedure A Multipoint Test
- ☐ Procedure B One-point Test

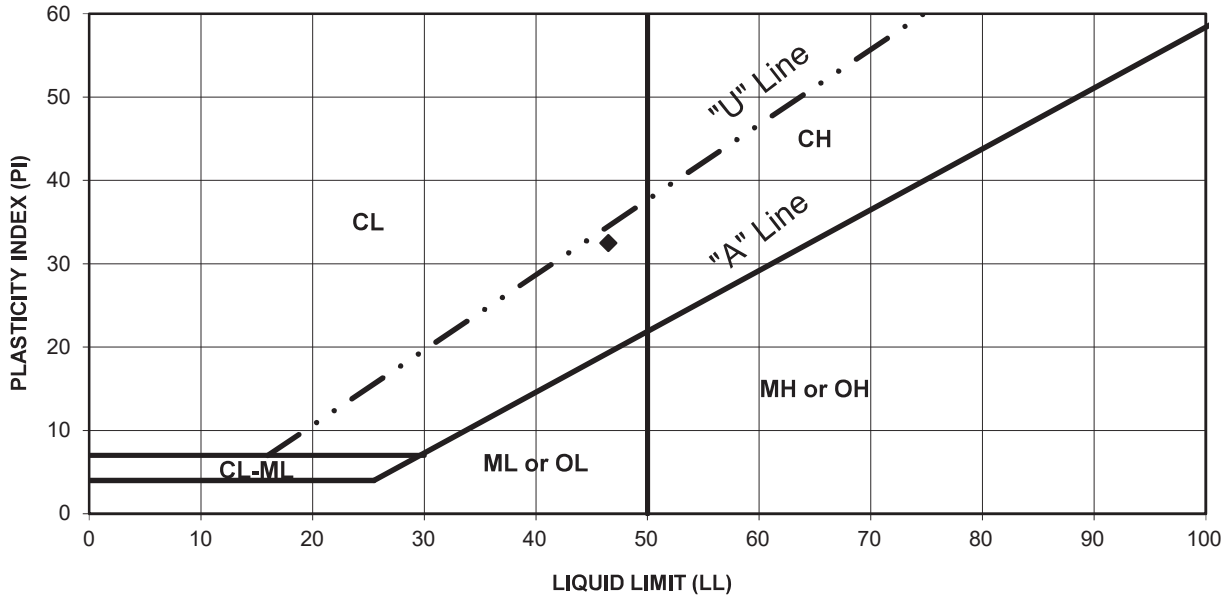


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	B-19-02	S-6	25	61	20	41	CH

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

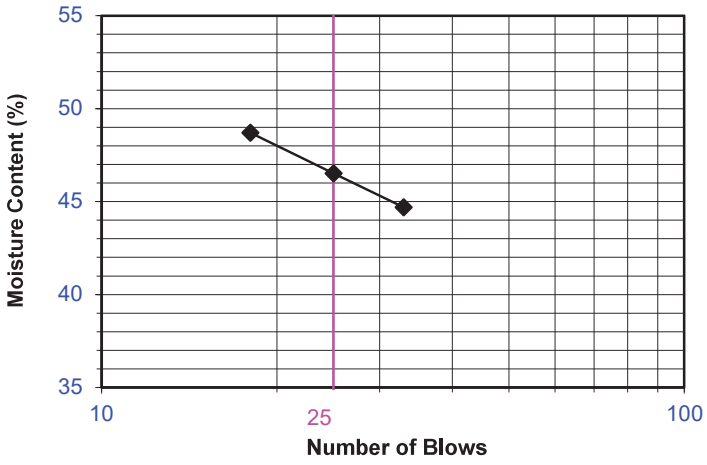
ATTERBERG LIMITS
ASTM D 4318

Project Name: LAC+USC WC Hospital Demolition **Tested By:** LS Date: 02/18/19
Project No.: 60597019 **Checked By:** AP Date: 02/19/19



PROCEDURE USED

- ☐ Wet Preparation
- ☒ Dry Preparation
- ☒ Procedure A Multipoint Test
- ☐ Procedure B One-point Test

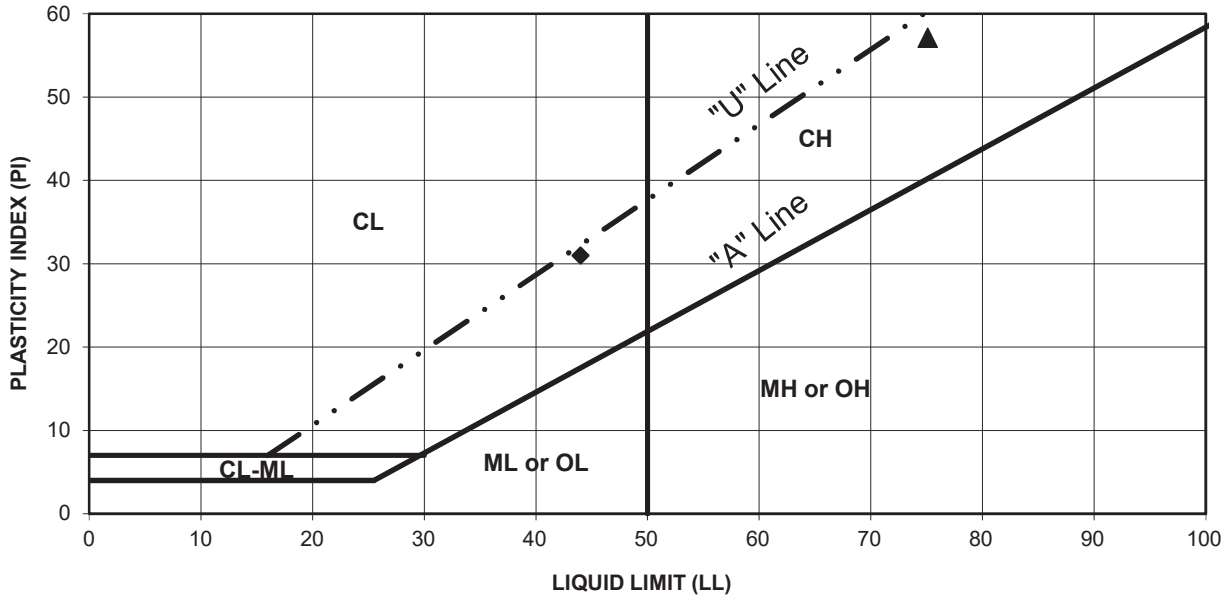


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	B-19-03	S-8	30	47	14	33	CL

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

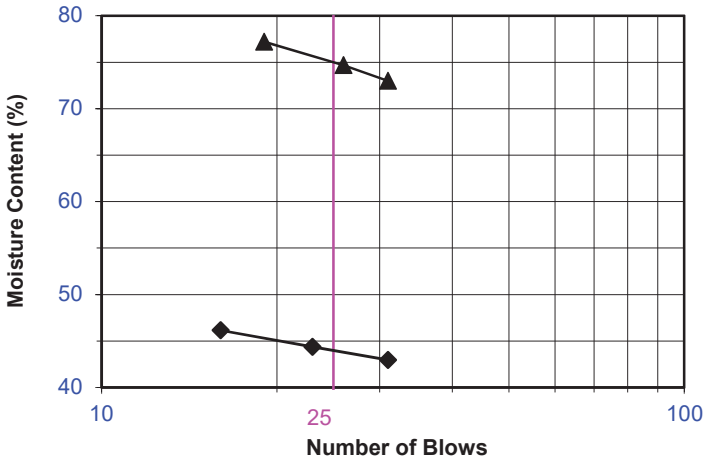
ATTERBERG LIMITS
ASTM D 4318

Project Name: LAC+USC WC Hospital Demolition **Tested By:** LS Date: 02/18/19
Project No.: 60597019 **Checked By:** AP Date: 02/19/19



PROCEDURE USED

- ☐ Wet Preparation
- ☒ Dry Preparation
- ☒ Procedure A Multipoint Test
- ☐ Procedure B One-point Test

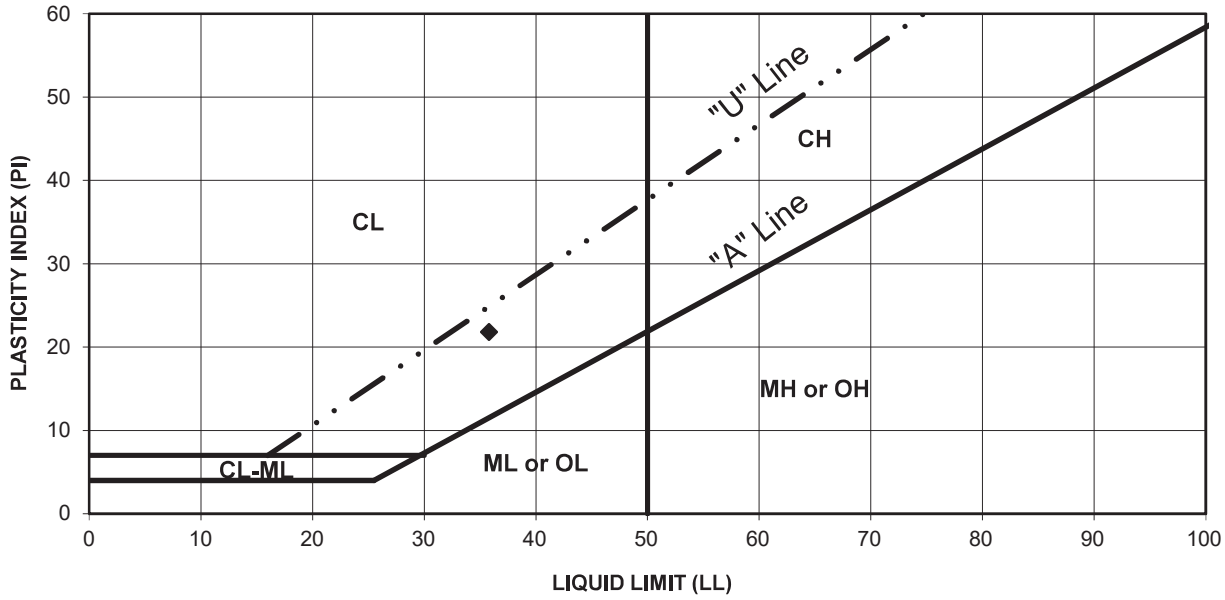


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	B-19-05	S-3	15	44	13	31	CL
▲	B-19-05	S-11	55	75	18	57	CH

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

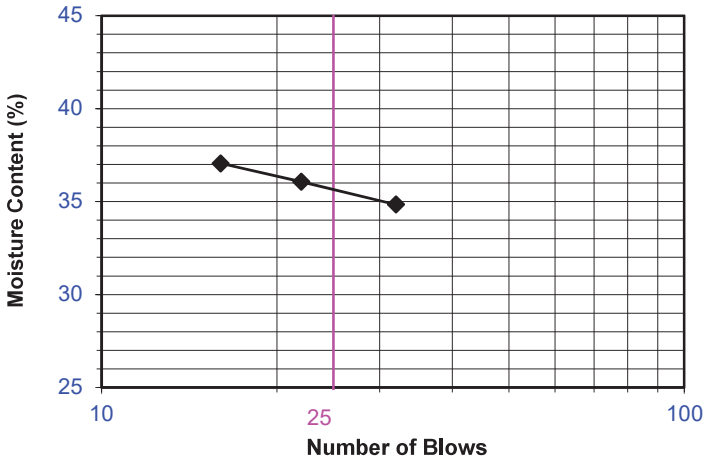
ATTERBERG LIMITS
ASTM D 4318

Project Name: LAC+USC WC Hospital Demolition **Tested By:** LS Date: 02/18/19
Project No.: 60597019 **Checked By:** AP Date: 02/19/19



PROCEDURE USED

- ☐ Wet Preparation
- ☒ Dry Preparation
- ☒ Procedure A Multipoint Test
- ☐ Procedure B One-point Test



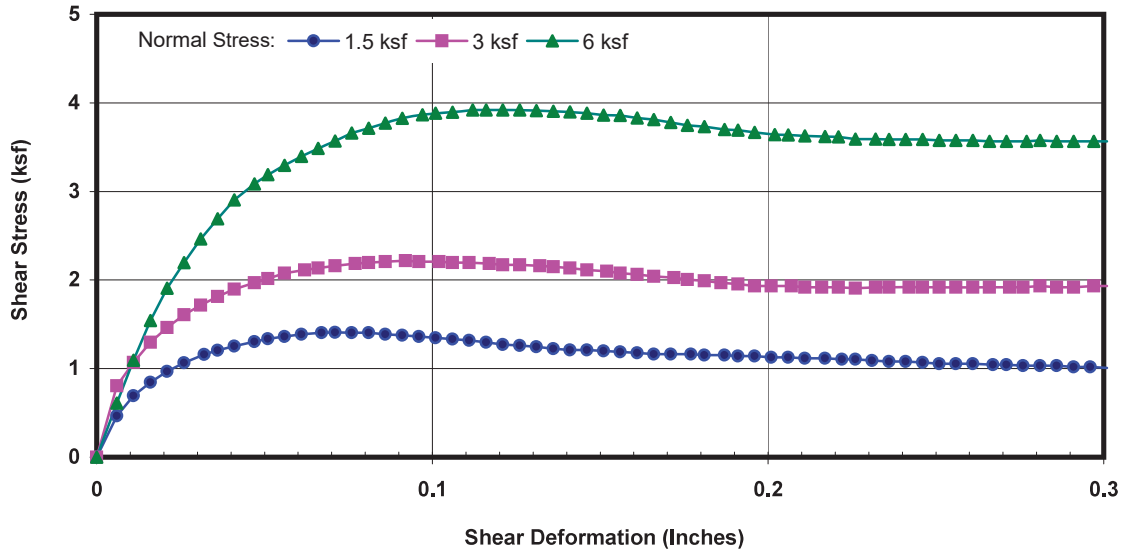
Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	B-19-07	B-1	0-5	36	14	22	CL

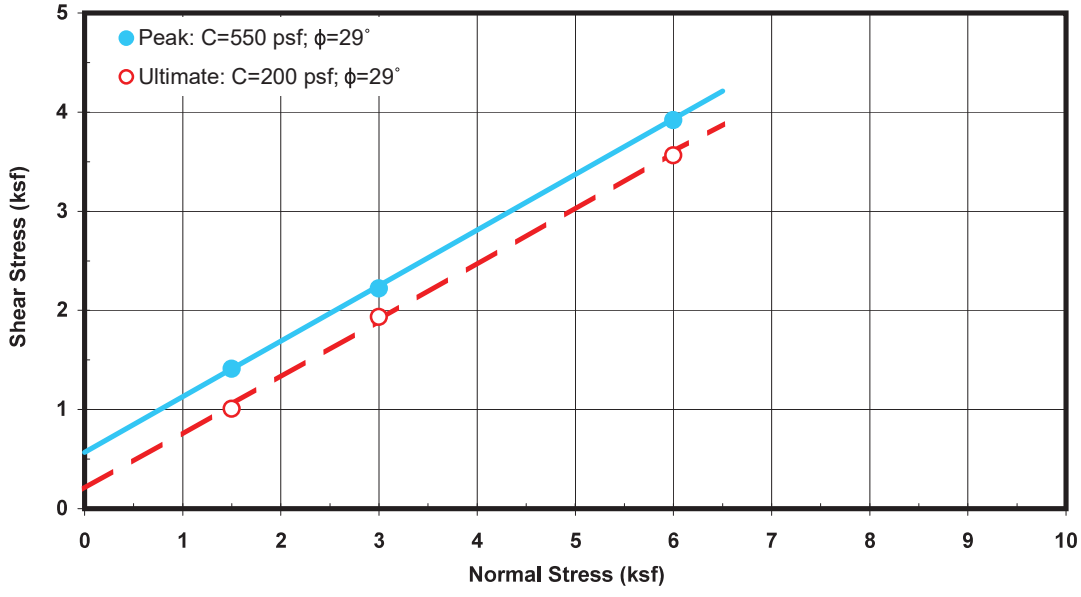
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	LAC+USC WC Hospital Demolition		**Tested By:**	ST	**Date:** 02/15/19
Project No.:	60597019		**Computed By:**	JP	**Date:** 02/20/19
Boring No.:	B-19-01		**Checked by:**	AP	**Date:** 02/20/19
Sample No.:	S-3	**Depth (ft):** 10			
Sample Type:	Mod. Cal.				
Soil Description:	Clayey Sand				
Test Condition:	Inundated	**Shear Type:** Regular			

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
129.7	112.0	15.8	17.4	84	93	1.5	1.410	1.008
						3	2.220	1.932
						6	3.920	3.564



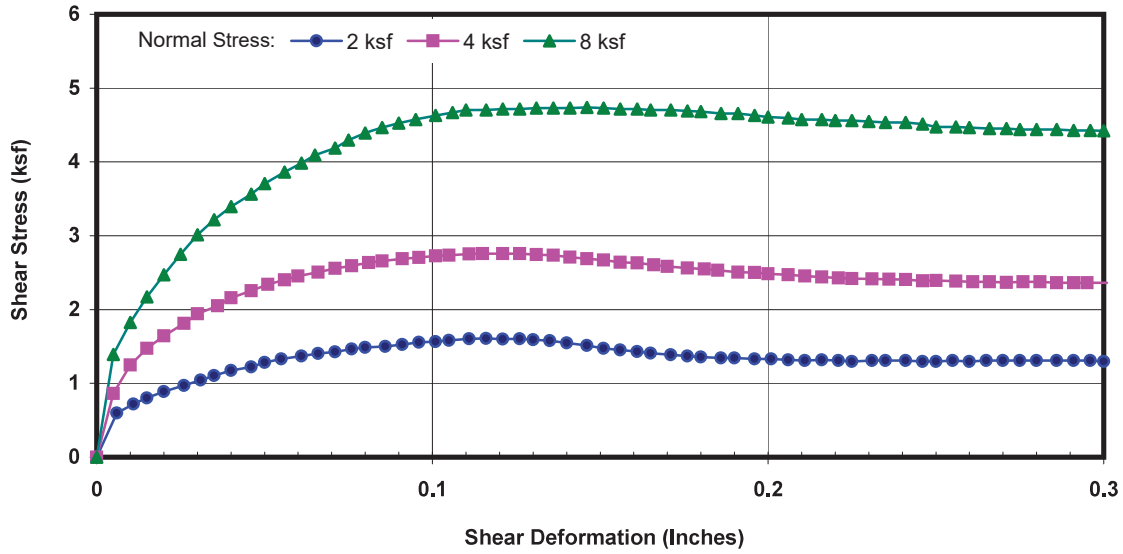


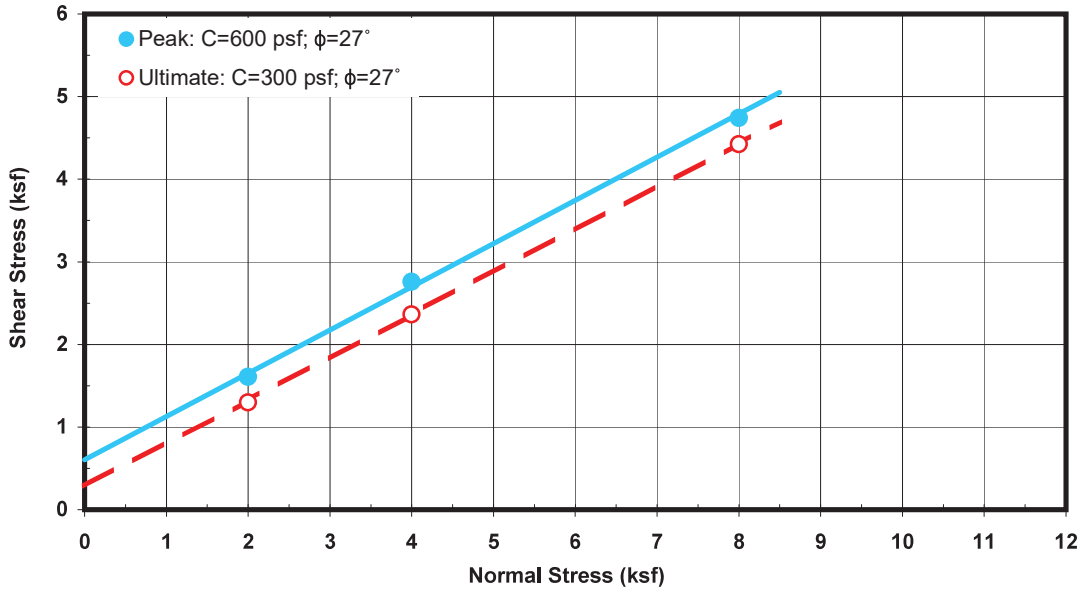
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	LAC+USC WC Hospital Demolition		**Tested By:**	ST	**Date:** 02/15/19
Project No.:	60597019		**Computed By:**	JP	**Date:** 02/20/19
Boring No.:	B-19-05		**Checked by:**	AP	**Date:** 02/20/19
Sample No.:	S-5	**Depth (ft):** 25			
Sample Type:	Mod. Cal.				
Soil Description:	Clay				
Test Condition:	Inundated	**Shear Type:** Regular			

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
123.2	96.9	27.1	27.4	99	100	2	1.609	1.296
						4	2.756	2.364
						8	4.740	4.423



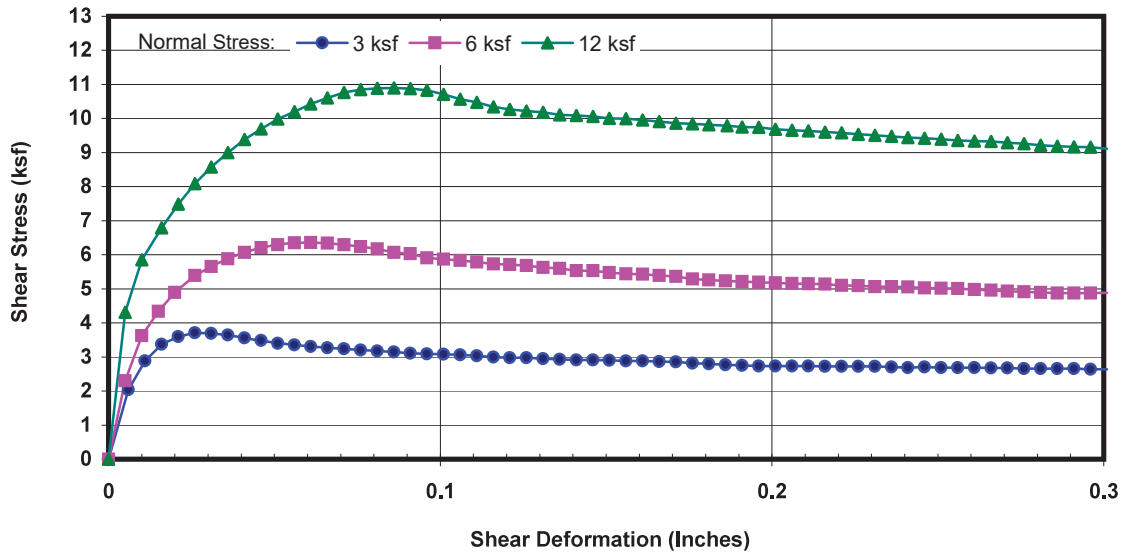


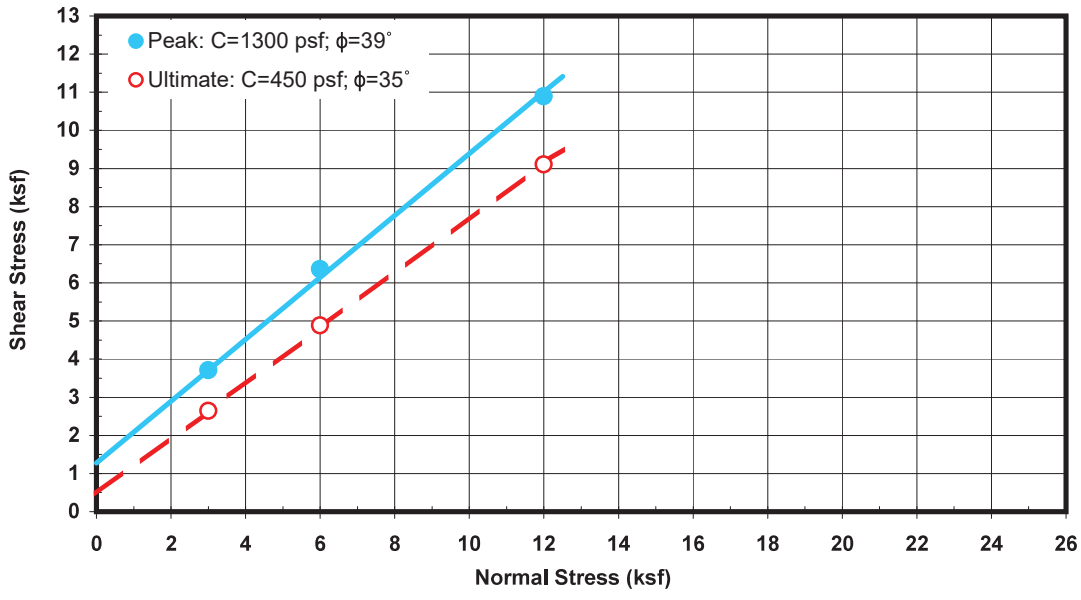
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	LAC+USC WC Hospital Demolition		**Tested By:**	ST	**Date:** 02/15/19
Project No.:	60597019		**Computed By:**	JP	**Date:** 02/20/19
Boring No.:	B-19-06		**Checked by:**	AP	**Date:** 02/20/19
Sample No.:	S-5	**Depth (ft):** 20			
Sample Type:	Mod. Cal.				
Soil Description:	Clay				
Test Condition:	Inundated	**Shear Type:** Regular			

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
136.3	123.4	10.4	13.4	77	99	3	3.710	2.641
						6	6.360	4.884
						12	10.895	9.107



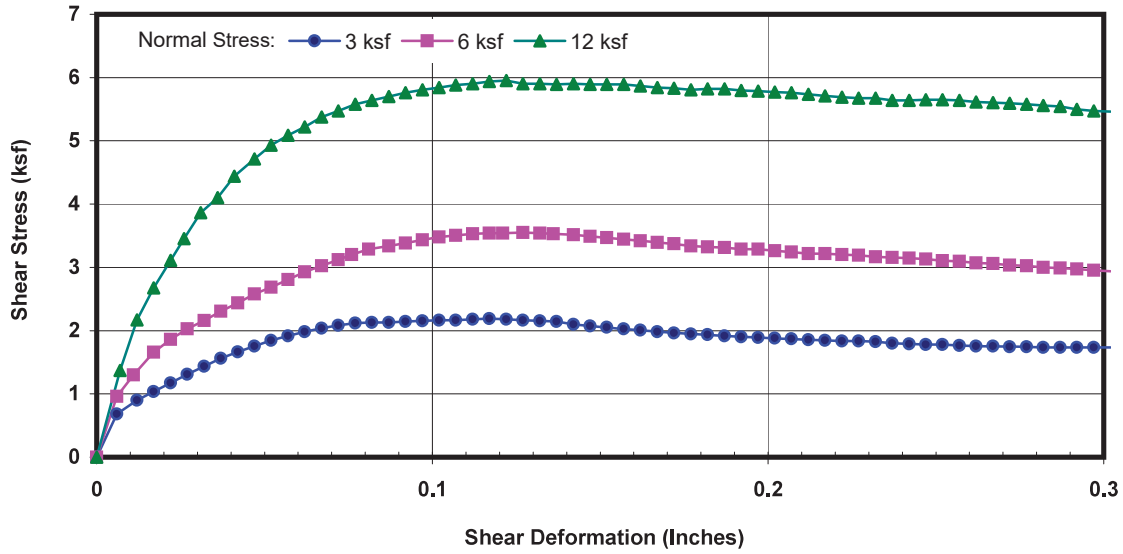


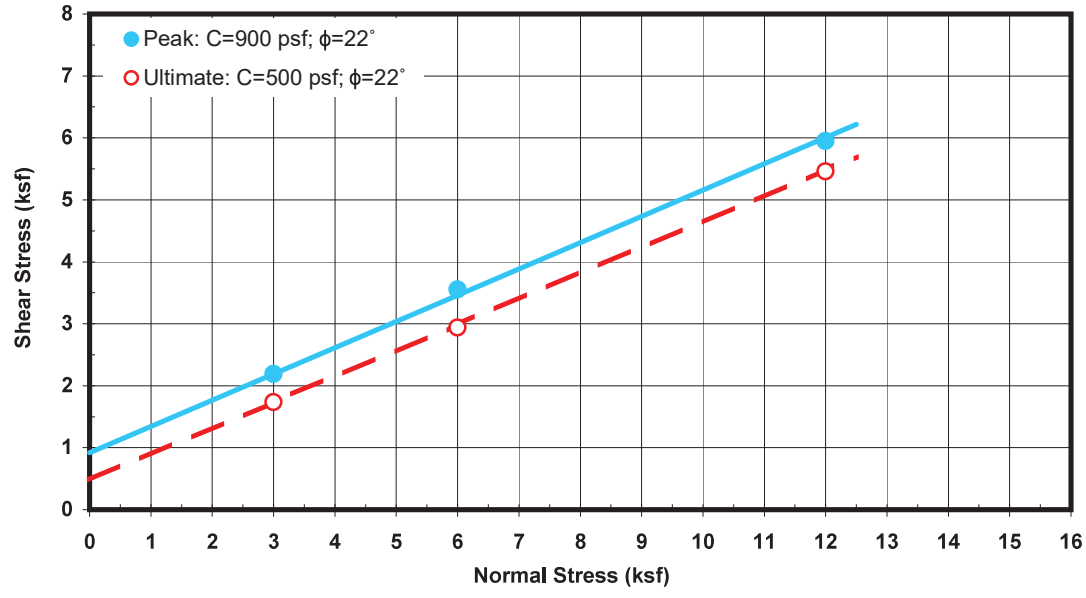
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	LAC+USC WC Hospital Demolition		**Tested By:**	ST	**Date:** 02/15/19
Project No.:	60597019		**Computed By:**	JP	**Date:** 02/20/19
Boring No.:	B-19-07		**Checked by:**	AP	**Date:** 02/20/19
Sample No.:	S-8	**Depth (ft):** 35			
Sample Type:	Mod. Cal.				
Soil Description:	Clay				
Test Condition:	Inundated	**Shear Type:** Regular			

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
126.2	102.4	23.3	24.0	97	100	3	2.189	1.733
						6	3.552	2.940
						12	5.952	5.460





AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

COMPACTION TEST

Client: AECOM
Project Name: LAC+USC WC Hospital Demolition
Project No. : 60597019
Boring No.: B-19-01
Sample No.: B-1
Visual Sample Description: Clay w/sand

AP Number: 19-0216
Tested By: AM Date: 02/18/19
Calculated By: JP Date: 02/19/19
Checked By: AP Date: 02/19/19
Depth(ft.): 0-5

Compaction Method: [X] ASTM D1557 [ ] ASTM D698
Preparation Method: [ ] Moist [X] Dry

METHOD: A
MOLD VOLUME (CU.FT): 0.0333

Wt. Comp. Soil + Mold (gm.)	3671	3754	3761	3715		
Wt. of Mold (gm.)	1837	1837	1837	1837		
Net Wt. of Soil (gm.)	1834	1917	1924	1878		
Container No.						
Wt. of Container (gm.)	148.09	147.09	142.64	140.49		
Wet Wt. of Soil + Cont. (gm.)	380.52	377.25	350.14	338.92		
Dry Wt. of Soil + Cont. (gm.)	363.87	356.23	326.66	312.94		
Moisture Content (%)	7.72	10.05	12.76	15.07		
Wet Density (pcf)	121.30	126.75	127.22	124.17		
Dry Density (pcf)	112.61	115.18	112.82	107.91		

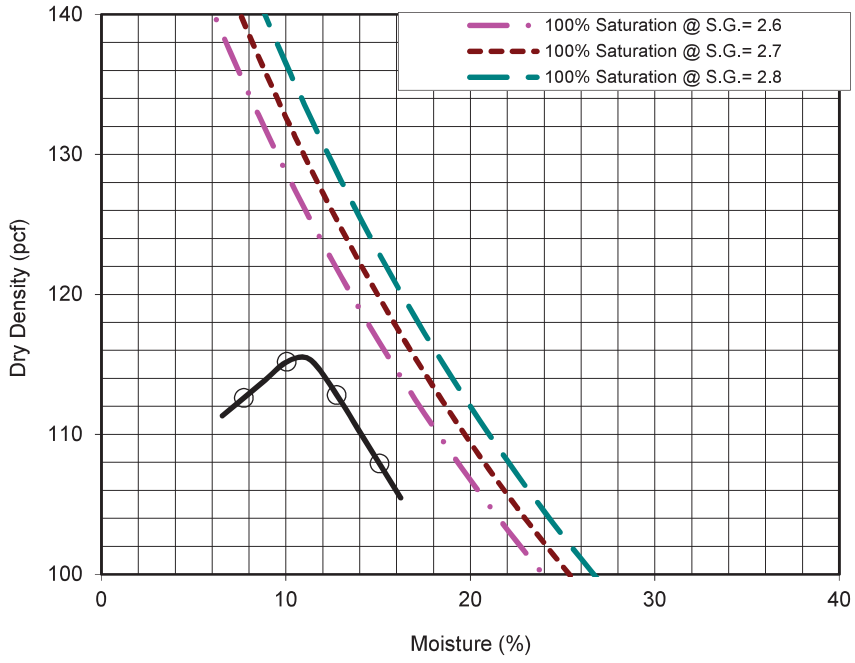
Maximum Dry Density (pcf): **115.4**
Maximum Dry Density w/ Rock Correction (pcf): **N/A**
Optimum Moisture Content (%): **10.8**
Optimum Moisture Content w/ Rock Correction (%): **N/A**

PROCEDURE USED

[X] *METHOD A: Percent of Oversize:* 1.4%
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD B: Percent of Oversize:* N/A
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD C: Percent of Oversize:* N/A
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)



AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

COMPACTION TEST

Client: AECOM
Project Name: LAC+USC WC Hospital Demolition
Project No. : 60597019
Boring No.: B-19-02
Sample No.: B-1
Visual Sample Description: Clay w/sand

AP Number: 19-0216
Tested By: AM Date: 02/18/19
Calculated By: JP Date: 02/19/19
Checked By: AP Date: 02/19/19
Depth(ft.): 0-5

Compaction Method: [X] ASTM D1557 [ ] ASTM D698
Preparation Method: [ ] Moist [X] Dry

METHOD: A
MOLD VOLUME (CU.FT): 0.0333

Wt. Comp. Soil + Mold (gm.)	3726	3751	3626	3717		
Wt. of Mold (gm.)	1837	1837	1837	1837		
Net Wt. of Soil (gm.)	1889	1914	1789	1880		
Container No.						
Wt. of Container (gm.)	140.13	150.17	148.86	155.12		
Wet Wt. of Soil + Cont. (gm.)	321.38	351.86	355.92	377.59		
Dry Wt. of Soil + Cont. (gm.)	303.87	328.31	341.28	347.63		
Moisture Content (%)	10.69	13.22	7.61	15.56		
Wet Density (pcf)	124.93	126.59	118.29	124.31		
Dry Density (pcf)	112.86	111.81	109.92	107.56		

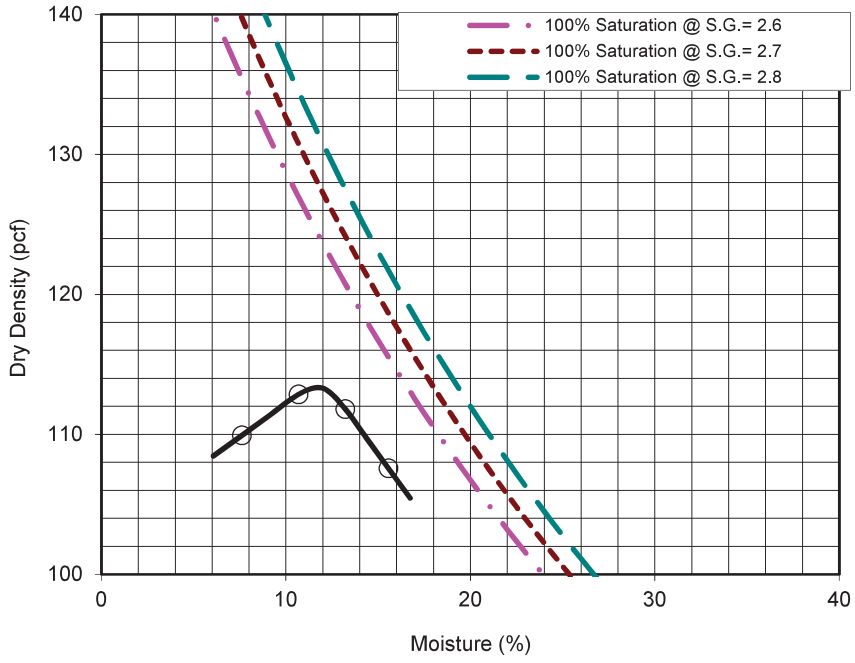
Maximum Dry Density (pcf): **113.3**
Maximum Dry Density w/ Rock Correction (pcf): **N/A**
Optimum Moisture Content (%): **11.9**
Optimum Moisture Content w/ Rock Correction (%): **N/A**

PROCEDURE USED

[X] *METHOD A: Percent of Oversize:* 0.3%
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD B: Percent of Oversize:* N/A
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD C: Percent of Oversize:* N/A
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)



AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

COMPACTION TEST

Client: AECOM
Project Name: LAC+USC WC Hospital Demolition
Project No. : 60597019
Boring No.: B-19-03
Sample No.: B-1
Visual Sample Description: Clay w/sand

AP Number: 19-0216
Tested By: JT Date: 02/18/19
Calculated By: JP Date: 02/19/19
Checked By: AP Date: 02/19/19
Depth(ft.): 0-5

Compaction Method: [X] ASTM D1557 [ ] ASTM D698
Preparation Method: [ ] Moist [X] Dry

METHOD: B
MOLD VOLUME (CU.FT): 0.0333

Wt. Comp. Soil + Mold (gm.)	3862	3821	3837	3715		
Wt. of Mold (gm.)	1836	1836	1836	1836		
Net Wt. of Soil (gm.)	2026	1985	2001	1879		
Container No.						
Wt. of Container (gm.)	147.93	139.42	142.00	134.84		
Wet Wt. of Soil + Cont. (gm.)	418.44	462.20	511.84	386.54		
Dry Wt. of Soil + Cont. (gm.)	384.18	414.24	472.24	366.90		
Moisture Content (%)	14.50	17.45	11.99	8.46		
Wet Density (pcf)	133.99	131.28	132.34	124.27		
Dry Density (pcf)	117.02	111.78	118.17	114.58		

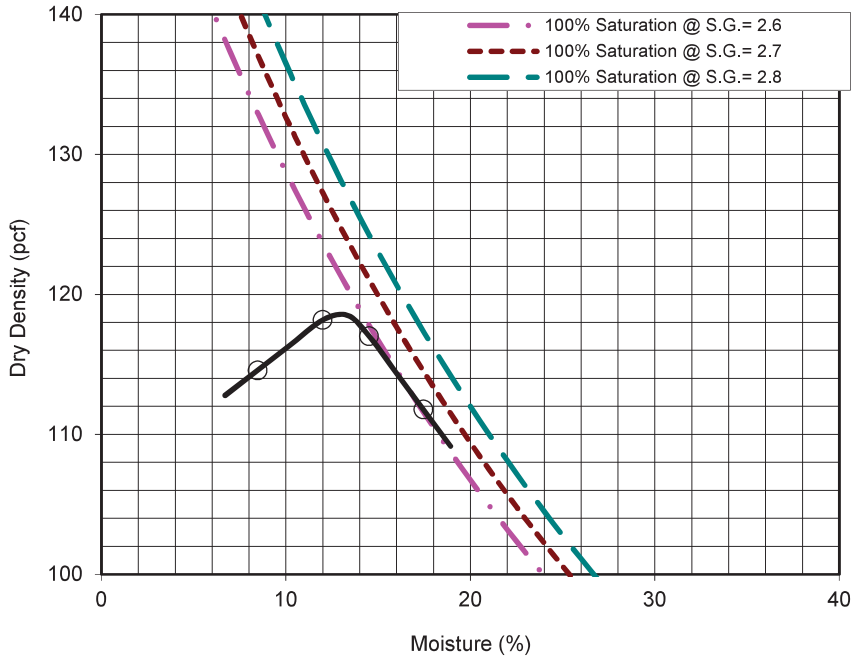
Maximum Dry Density (pcf): **118.5**
Maximum Dry Density w/ Rock Correction (pcf): **N/A**
Optimum Moisture Content (%): **13.0**
Optimum Moisture Content w/ Rock Correction (%): **N/A**

PROCEDURE USED

[ ] *METHOD A: Percent of Oversize:* N/A
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[X] *METHOD B: Percent of Oversize:* 3.9%
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD C: Percent of Oversize:* N/A
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)



AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

COMPACTION TEST

Client:	AECOM			AP Number:	19-0216
Project Name:	LAC+USC WC Hospital Demolition	Tested By:	AM	Date:	02/14/19
Project No. :	60597019	Calculated By:	JP	Date:	02/19/19
Boring No.:	B-19-04	Checked By:	AP	Date:	02/19/19
Sample No.:	B-1	Depth(ft.):	0-5		
Visual Sample Description:	Clay w/sand				

METHOD	A	Compaction Method	[X] ASTM D1557
MOLD VOLUME (CU.FT)	0.0333		[ ] ASTM D698
		Preparation Method	[ ] Moist
			[X] Dry

Wt. Comp. Soil + Mold (gm.)	3873	3792	3888	3782		
Wt. of Mold (gm.)	1837	1837	1837	1837		
Net Wt. of Soil (gm.)	2036	1955	2051	1945		
Container No.						
Wt. of Container (gm.)	148.85	135.84	143.29	147.37		
Wet Wt. of Soil + Cont. (gm.)	327.52	321.72	304.10	321.20		
Dry Wt. of Soil + Cont. (gm.)	303.76	294.06	285.86	304.73		
Moisture Content (%)	15.34	17.48	12.79	10.47		
Wet Density (pcf)	134.66	129.27	135.65	128.64		
Dry Density (pcf)	116.75	110.03	120.26	116.45		

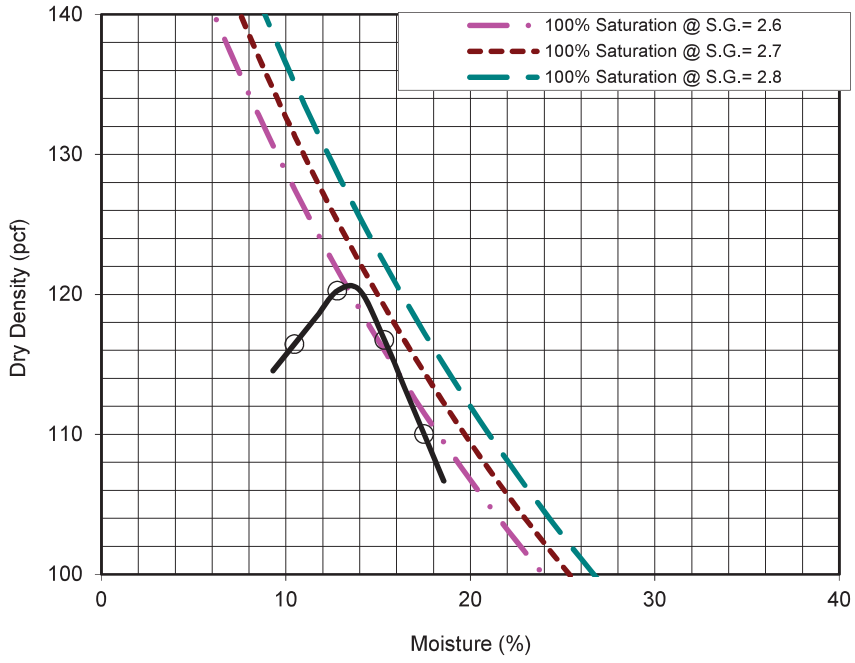
Maximum Dry Density (pcf)	120.4	Optimum Moisture Content (%)	13.5
Maximum Dry Density w/ Rock Correction (pcf)	N/A	Optimum Moisture Content w/ Rock Correction (%)	N/A

PROCEDURE USED

[X] *METHOD A: Percent of Oversize:* 0.6%
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD B: Percent of Oversize:* N/A
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD C: Percent of Oversize:* N/A
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)



AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

COMPACTION TEST

Client: AECOM
Project Name: LAC+USC WC Hospital Demolition
Project No. : 60597019
Boring No.: B-19-07
Sample No.: B-1
Visual Sample Description: Lean Clay w/sand

AP Number: 19-0216
Tested By: AM Date: 02/18/19
Calculated By: JP Date: 02/19/19
Checked By: AP Date: 02/19/19
Depth(ft.): 0-5

Compaction Method: [X] ASTM D1557 [ ] ASTM D698
Preparation Method: [ ] Moist [X] Dry

METHOD: A
MOLD VOLUME (CU.FT): 0.0333

Wt. Comp. Soil + Mold (gm.)	3837	3803	3697	3799		
Wt. of Mold (gm.)	1837	1837	1837	1837		
Net Wt. of Soil (gm.)	2000	1966	1860	1962		
Container No.						
Wt. of Container (gm.)	329.22	474.34	474.34	571.09		
Wet Wt. of Soil + Cont. (gm.)	472.49	607.98	608.23	698.64		
Dry Wt. of Soil + Cont. (gm.)	455.24	594.41	597.88	680.95		
Moisture Content (%)	13.69	11.30	8.38	16.10		
Wet Density (pcf)	132.24	130.03	123.02	129.76		
Dry Density (pcf)	116.32	116.82	113.51	111.77		

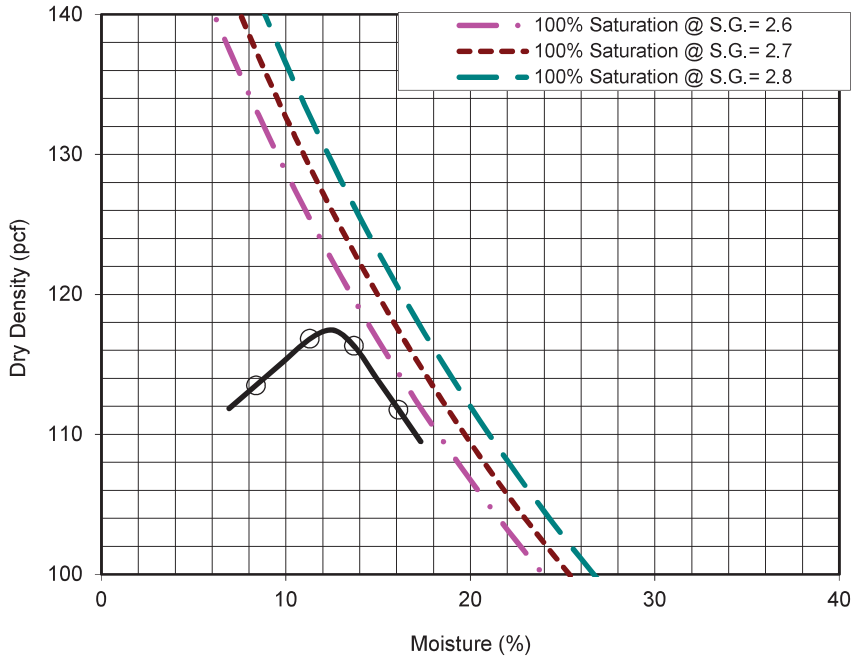
Maximum Dry Density (pcf): 117.4
Maximum Dry Density w/ Rock Correction (pcf): N/A
Optimum Moisture Content (%): 12.5
Optimum Moisture Content w/ Rock Correction (%): N/A

PROCEDURE USED

[X] *METHOD A: Percent of Oversize:* 0.9%
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD B: Percent of Oversize:* N/A
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD C: Percent of Oversize:* N/A
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)



AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

COMPACTION TEST

Client: AECOM
Project Name: LAC+USC WC Hospital Demolition
Project No. : 60597019
Boring No.: B-19-10
Sample No.: B-1
Visual Sample Description: Clay w/sand

AP Number: 19-0216
Tested By: AM Date: 02/18/19
Calculated By: JP Date: 02/19/19
Checked By: AP Date: 02/19/19
Depth(ft.): 0-5

Compaction Method: [X] ASTM D1557 [ ] ASTM D698
Preparation Method: [ ] Moist [X] Dry

METHOD: A
MOLD VOLUME (CU.FT): 0.0333

Wt. Comp. Soil + Mold (gm.)	3799	3815	3719	3783		
Wt. of Mold (gm.)	1837	1837	1837	1837		
Net Wt. of Soil (gm.)	1962	1978	1882	1946		
Container No.						
Wt. of Container (gm.)	151.12	137.76	146.85	140.97		
Wet Wt. of Soil + Cont. (gm.)	349.29	341.96	351.99	350.45		
Dry Wt. of Soil + Cont. (gm.)	332.11	319.93	337.96	324.65		
Moisture Content (%)	9.49	12.09	7.34	14.04		
Wet Density (pcf)	129.76	130.79	124.47	128.67		
Dry Density (pcf)	118.51	116.68	115.96	112.82		

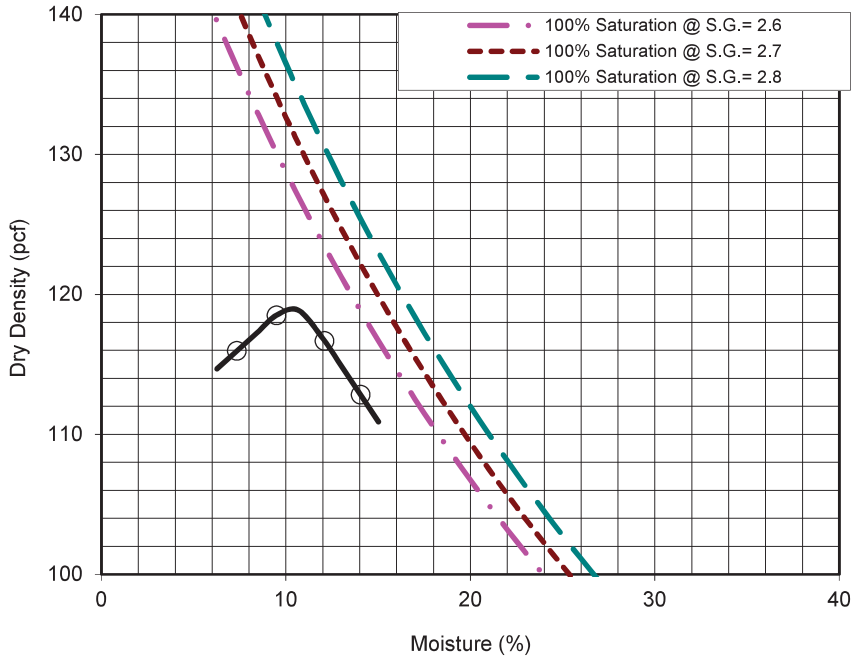
Maximum Dry Density (pcf): **118.9**
Maximum Dry Density w/ Rock Correction (pcf): **N/A**
Optimum Moisture Content (%): **10.5**
Optimum Moisture Content w/ Rock Correction (%): **N/A**

PROCEDURE USED

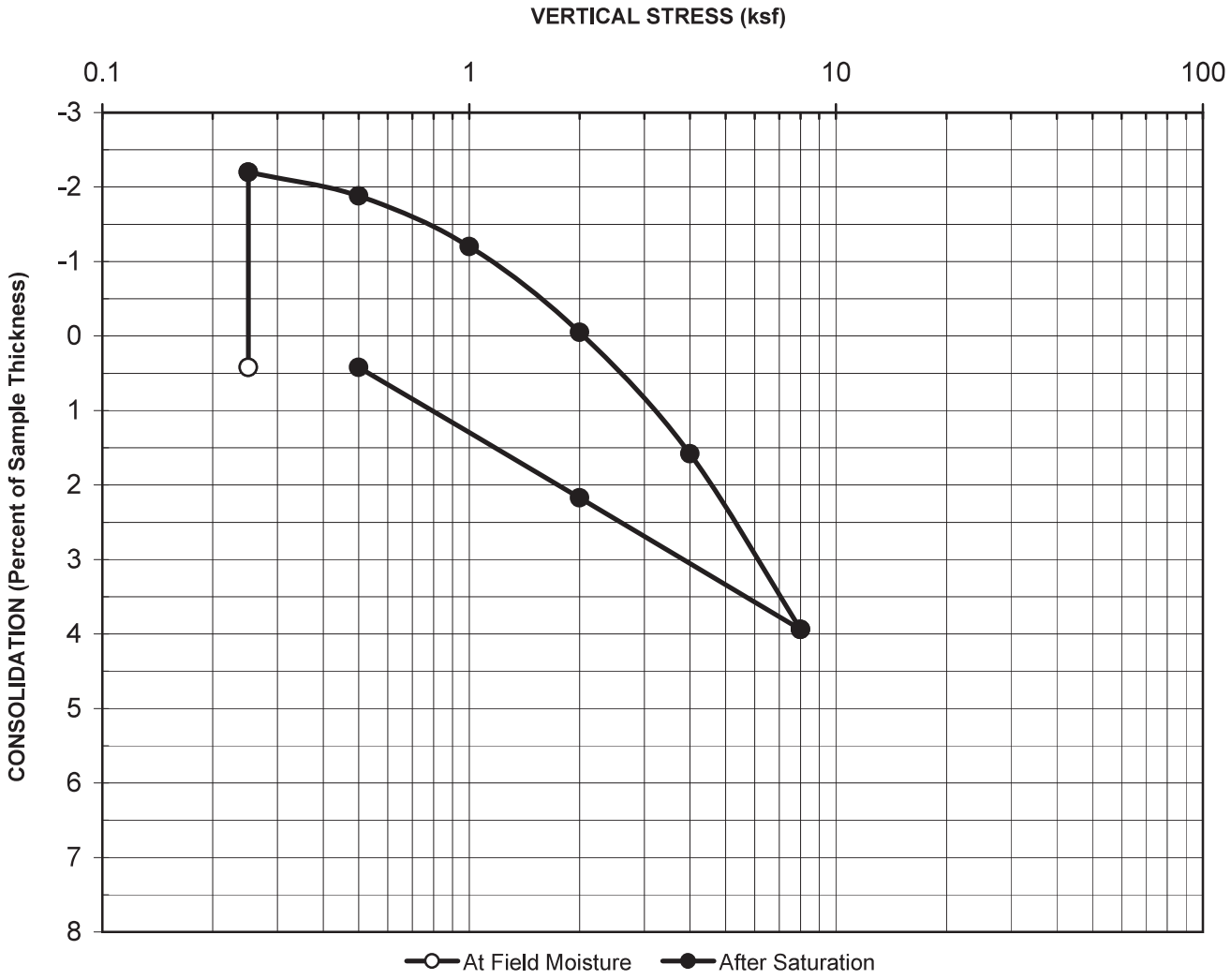
[X] *METHOD A: Percent of Oversize:* 1.4%
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD B: Percent of Oversize:* N/A
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

[ ] *METHOD C: Percent of Oversize:* N/A
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)



AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

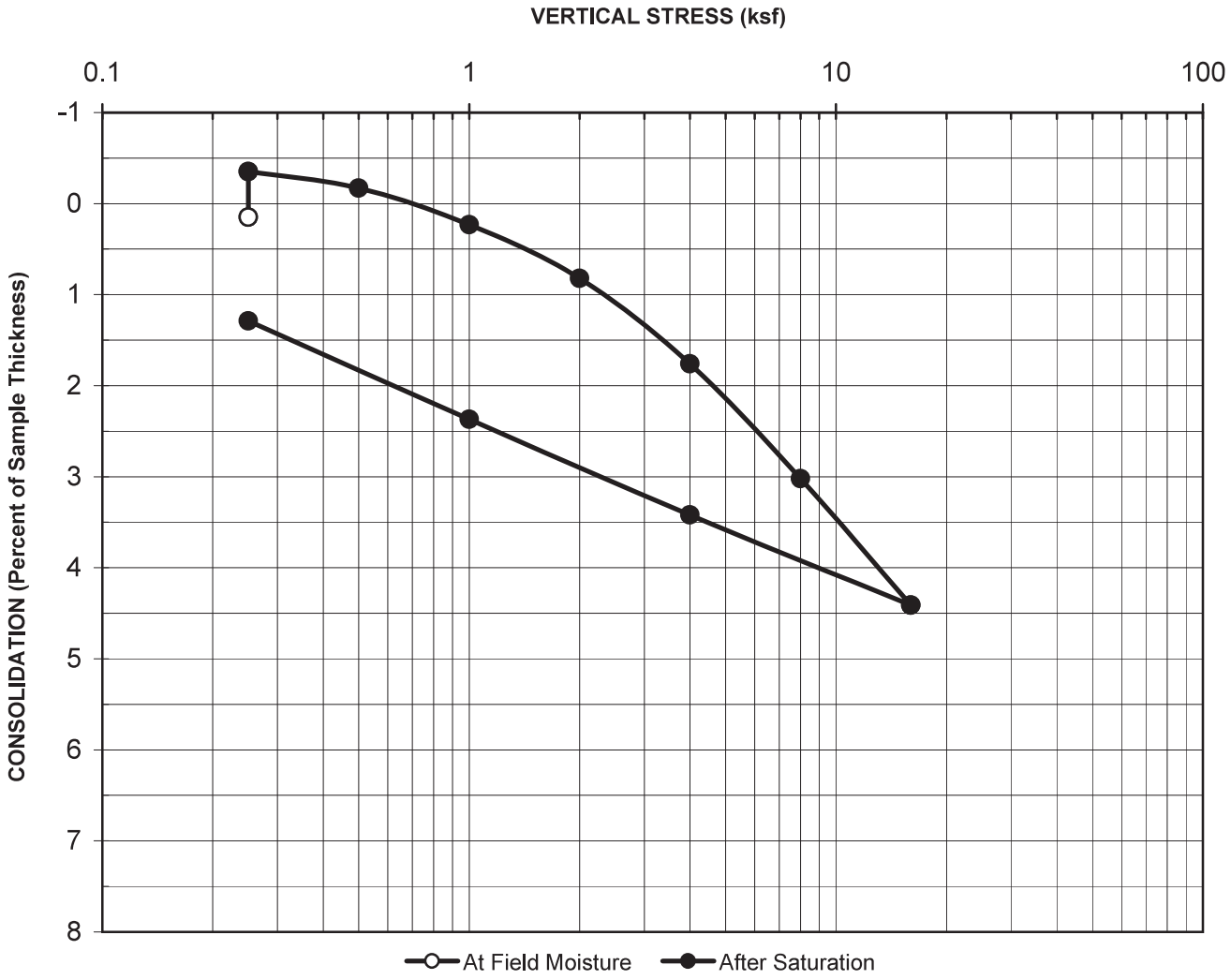


Boring No. :	B-19-05	Initial Dry Unit Weight (pcf):	100.4
Sample No.:	S-3	Initial Moisture Content (%):	26.0
Depth (feet):	15	Final Moisture Content (%):	26.6
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Lean Clay w/caliche	Initial Void Ratio:	0.68
Remarks:	Swell= 2.62% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name:	LAC+USC WC Hospital Demolition
Project No.:	60597019
Date:	2/12/2019
AP No:	19-0216 **Figure No:**

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com



Boring No. :	B-19-05	Initial Dry Unit Weight (pcf):	103.0
Sample No.:	S-11	Initial Moisture Content (%):	22.3
Depth (feet):	55	Final Moisture Content (%):	26.7
Sample Type:	Mod Cal	Assumed Specific Gravity:	2.7
Soil Description:	Fat Clay	Initial Void Ratio:	0.64
Remarks:	Swell= 0.50% upon inundation		

CONSOLIDATION CURVE
ASTM D 2435

Project Name: LAC+USC WC Hospital Demolition
Project No.: 60597019
Date: 2/12/2019
AP No: 19-0216 **Figure No:**

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

EXPANSION INDEX TEST RESULTS

ASTM D 4829

Client Name: AECOM
Project Name: LAC+USC WC Hospital Demolition
Project No.: 60597019

AP Job No.: 19-0216
Date: 02/18/19

Boring No.	Sample No.	Depth (ft)	Soil Description	Molded Dry Density (pcf)	Molded Moisture Content (%)	Init. Degree Saturation (%)	Measured Expansion Index	Corrected Expansion Index
B-19-04	S-1	5	Clay w/sand	106.8	10.5	49.1	102	101

ASTM EXPANSION CLASSIFICATION

Expansion Index	Classification
0-20	V. Low
21-50	Low
51-90	Medium
91-130	High
>130	V. High

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

EXPANSION INDEX TEST RESULTS

ASTM D 4829

Client Name: AECOM
Project Name: LAC+USC WC Hospital Demolition
Project No.: 60597019

AP Job No.: 19-0216
Date: 02/18/19

Boring No.	Sample No.	Depth (ft)	Soil Description	Molded Dry Density (pcf)	Molded Moisture Content (%)	Init. Degree Saturation (%)	Measured Expansion Index	Corrected Expansion Index
B-19-08	S-2	10	Clay w/sand	104.3	11.6	51.0	110	111

ASTM EXPANSION CLASSIFICATION

Expansion Index	Classification
0-20	V. Low
21-50	Low
51-90	Medium
91-130	High
>130	V. High

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

ASTM D 2419
SAND EQUIVALENT TEST

Client Name: AECOM
Project Name: LAC+USC WC Hospital Demolition
Project No.: 60597019

AP Job No.: 19-0216
Test Date: 02/18/19

Boring No.	Sample No.	Depth (feet)	Soil Description	Clay Reading	Sand Reading	Corrected Sand Reading	Sand Equivalent
B-19-02	S-3	15	Silty Clay	12.8	10.2	0.1	1

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

ASTM D 2419
SAND EQUIVALENT TEST

Client Name: AECOM
Project Name: LAC+USC WC Hospital Demolition
Project No.: 60597019

AP Job No.: 19-0216
Test Date: 02/18/19

Boring No.	Sample No.	Depth (feet)	Soil Description	Clay Reading	Sand Reading	Corrected Sand Reading	Sand Equivalent
B-19-05	S-4	20	Clay	12.9	10.2	0.1	1

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

CORROSION TEST RESULTS

Client Name: AECOM
Project Name: LAC+USC WC Hospital Demolition
Project No.: 60597019

AP Job No.: 19-0216
Date: 02/15/19

Boring No.	Sample No.	Depth (feet)	Soil Type	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
B-19-04	B-1	0-5	CL	2025	8.8	56	37

NOTES: Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

CORROSION TEST RESULTS

Client Name: AECOM
Project Name: LAC+USC WC Hospital Demolition
Project No.: 60597019

AP Job No.: 19-0216
Date: 02/15/19

Boring No.	Sample No.	Depth (feet)	Soil Type	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
B-19-09	S-2	10	CL	981	7.7	248	44

NOTES: Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

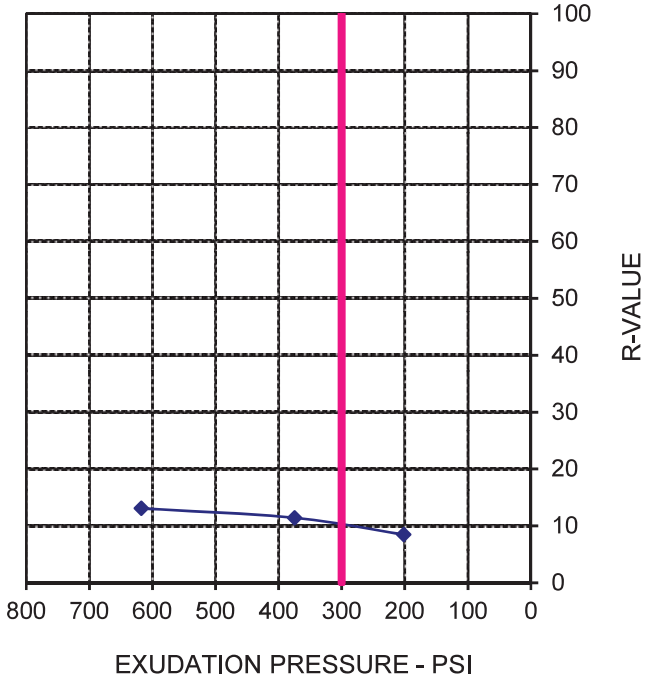
R-VALUE TEST DATA

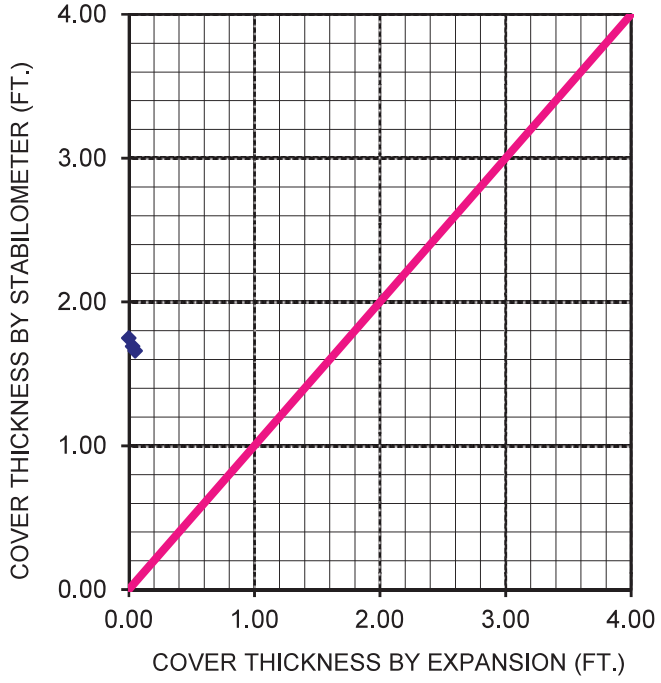
ASTM D2844

Project Name:	LAC+USC WC Hospital Demolition	Tested By:	ST	Date:	02/12/19
Project Number:	60597019	Computed By:	KM	Date:	02/18/19
Boring No.:	B-19-09	Checked By:	AP	Date:	02/20/19
Sample No.:	B-1	Depth (ft.):	0-5		
Location:	N/A				
Soil Description:	Clay				

Mold Number	G	H	I	
Water Added, g	64	48	32	
Compact Moisture(%)	22.0	20.1	18.3	
Compaction Gage Pressure, psi	50	50	100	
Exudation Pressure, psi	202	375	618	
Sample Height, Inches	2.5	2.5	2.4	
Gross Weight Mold, g	2859	2851	2842	
Tare Weight Mold, g	1827	1836	1818	
Net Sample Weight, g	1032	1015	1023	
Expansion, inchesx10^{-4}	0	9	15	
Stability 2,000 (160 psi)	61/136	58/132	56/130	
Turns Displacement	4.76	4.11	3.63	
R-Value Uncorrected	8	11	14	
R-Value Corrected	8	11	13	
Dry Density, pcf	102.5	102.4	109.2	
Traffic Index	8.0	8.0	8.0	
G.E. by Stability	1.75	1.69	1.66	
G.E. by Expansion	0.00	0.03	0.05	

R-VALUE	By Exudation:	10
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	10
Remarks	Gf = 1.34, and 0.4 % Retained on the ¾" *Not Applicable	





The liquefaction potential evaluation was generally based on the approaches recommended by R. B. Seed, et al. (2003) for a unified and consistent framework to evaluate liquefaction. The comprehensive efforts by Seed et al. (2003) summarized recent advances in the liquefaction engineering topics based on a recent database. This approach meets the requirements in the California Geological Survey (CGS; formerly California Division of Mines and Geology [CDMG]) Special Publication 117A (2008). The simplified liquefaction evaluation procedure involves the following basic steps:

1. An estimate is made of the cyclic stress ratio (CSR) caused by given earthquake ground motions at different depths using a simplified approach. The intensity of ground shaking, the duration of shaking, and the variations of induced shear stresses with depth are incorporated into the evaluation.
2. An estimate is then made of cyclic resistance ratio (CRR), indicative of the liquefaction resistance of the site subsurface soils. Recommendations were provided to estimate CRR from standard penetration test (SPT) blow counts.

The procedure can provide a deterministic index, namely a factor of safety (FS), comparing of the CRR and CSR to evaluate the potential zone of liquefaction in the soil deposit. The FS against liquefaction is defined as FS = CRR/CSR. Potential zones of liquefaction correspond to soil layers where FS is less than 1. In addition to the FS, the procedure can provide the probabilistic evaluation for liquefaction potential.

SIMPLIFIED PROCEDURES BASED ON SPT BLOWCOUNTS

The use of standard penetration test (SPT) blow counts in liquefaction potential evaluation is considered to be appropriate because many factors affecting the liquefaction potential of sandy soils affect the SPT blow count in a similar way. Among several SPT-based approaches, the procedure proposed by Seed, et al. (2003) was adopted in this analysis because it was developed more recently and based on a larger database, which consisted of modern field records, than previous approaches.

Initially, the subsurface soils that consisted of fine-grained materials (silts and clays) were evaluated to screen out the non-liquefiable fine-grained soils. The silts and clays (including the soils containing more than 20 percent fines with plasticity index [PI] higher than 12 percent or soils consisting of fines more than 35 percent fines with PI less than 12 percent) were categorized into three groups based on the following criteria:

- Zone A: soils with PI less than 12 percent and liquid limit (LL) less than 37 percent were considered potentially susceptible to cyclically induced liquefaction if the in-situ moisture content was more than 80 percent of the LL.
- Zone B: soils with PI less than 20 percent and LL less than 47 percent may be liquefiable if the in-situ moisture content was more than 85 percent of the LL.
- Zone C: soils outside zones A and B were generally not considered susceptible to classic cyclic liquefaction, but should be checked for potential loss of strength with remolding or monotonic accumulation of shear deformation.

The liquefaction potential of the soils classified as Zones A or B was then estimated based on the following procedures recommended for the sandy soils. The soils classified as Zone C were not included in this liquefaction analysis; however, other evaluations were performed on those Zone C clays/silts, which may be vulnerable to strength loss during shaking.

The CSR, generally based on the earthquake magnitude, shaking intensity (maximum ground acceleration), and overall soil conditions (average shear wave velocity at depths within approximately 50

feet below the ground surface), was then adjusted by a duration weighting factor (DWFM), which was correlated to the design earthquake magnitude, and an effective overburden stress factor (K_σ) to a fully normalized value at a magnitude of 7.5 and effective overburden stress of 1 atmosphere (atm; approximately 1 ton per square foot [tsf]).

Either the CRR or probability of liquefaction potential can be evaluated using the SPT blow count data of potentially liquefiable sandy soils (including the fine-grained material classified as Zones A and B). Normalized clean sand SPT blow counts, $N_{1,60cs}$, were obtained by modifying the field SPT blow counts. The modifications included normalizing to an overburden pressure of 1 atm and corrections for hammer energy, borehole diameter, rod length, the lining (or lack of lining) of SPT samplers, and fines content (percent passing No. 200 sieve). An expression was presented in the procedure for CRR for a given earthquake magnitude and probability, as a function of $(N_1)_{60cs}$ and effective overburden stress. For the deterministic approach, the threshold was set at a probability of 15 percent in this analysis. The FS can be obtained using the pairs of CRR and CSR. In addition to CRR, probabilities at a given earthquake magnitude can be calculated directly as a function of $(N_1)_{60cs}$, CSR, and effective overburden stress.

ANALYSES

Calculations for liquefaction potential evaluation was based on the design event with a peak ground acceleration (PGA) of 1.095g and corresponding moment magnitude of 6.8. The PGA was based on the full mapped maximum considered earthquake geometric mean (MCE_G) and the corresponding modal magnitude was the result of deaggregation using USGS online tool (USGS, 2008). The design groundwater level was assumed at elevation 300 feet MSL.

SEISMIC SETTLEMENT

Liquefaction-induced settlement on level ground at the existing ground surface was estimated based on procedures presented by Tokimatsu and Seed (1987) for the SPT method. The dry sand and liquefaction-induced settlement were based on design high groundwater (more conservative for liquefaction condition) and existing low groundwater level during field exploration (more conservative for dry sand settlement). The total seismic-induced settlement is up to 0.3 inch for either scenario. The dry sand and liquefaction induced settlement calculations are attached at the end of this appendix.

SHEET	1 OF 1
PROJECT	29405910.0
DATE	4/22/2019
COMPUTED BY	DN
CHECKED BY	NS

AECOM

PROJECT: LAC+USC WC Demolition

SUBJECT: LIQUEFACTION POTENTIAL EVALUATION: B-19-05

Reference: Seed, et.al., 2003, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework
FAT CLAYS (CH) and ELASTIC SILTS (MH) not considered in analyses

Soil Unit Weight (pcf)	=	120
Energy Ratio (%)	=	80
Design Probability in decimals	=	0.15
Peak Ground Acceleration (g)	=	1.095
Earthquake Magnitude	=	6.8

BORING	DEPTH	SPT-N	SOIL CLASSIF.	PERCENT FINES	DEPTH TO WATER DURING DRILLING (feet)	BOREHOLE DIAMETER (inches)	SAMPLE TYPE	LINER USED	DESIGN WATER LEVEL IN DEPTH (feet)	LIQUE-FIABLE FINE-GRAINED SOILS	AVERAGE SHEAR WAVE VELOCITY AT UPPER 40 FEET (feet/sec)	TOTAL PRESSURE	EFFECTIVE PRESSURE	DEPTH FACTOR	ENERGY FACTOR	BORING DIAMETER FACTOR	ROD FACTOR	SAMPLER FACTOR	NOR. PENETR.	MODIFIED PENETR.	FINES CORRECTION	PENETR. RESIST.	SHEAR MASS PARTICIPATION FACTOR	STD DEVIATION FOR r_d	EFFECTIVE PRESSURE DURING LIQUEFACTION	IN-SITU CYCLIC STRESS RATIO	SOIL TYPE	Dr			ADJUSTED CYCLIC STRESS RATIO	CYCLIC RESISTANCE RATIO	PROBABILITY OF LIQUEFACTION	FACTOR OF	VOLUM. STRAIN	LAYER THICKNESS	SETTLE LAYER	SETTLE TOTAL
B-19-05	10	3	Clay	85.0	46	8	SPT	No	5	No	500	1.20	1.20	1.29	1.33	1.15	0.80	1.10	4	5	1.5	8	0.90	0.05	0.89	0.866	COARSE	31	1.25	1.19	0.584	0.119	0	non-liquefiable		10		
B-19-05	20	9	Clay	85.0	46	8	SPT	No	5	No	500	2.40	2.40	0.91	1.33	1.15	0.95	1.10	8	13	1.3	17	0.73	0.09	1.46	0.852	COARSE	46	1.25	1.12	0.612	0.231	0	non-liquefiable		10		
B-19-05	30	37	Sand	15.0	46	8	SPT	No	5		500	3.60	3.60	0.75	1.33	1.15	0.95	1.20	28	48	1.1	52	0.54	0.13	2.04	0.682	COARSE	84	1.25	1.01	0.539	1.998	0	>1.3		10		
B-19-05	40	42	Sand	15.0	46	8	SPT	No	5		500	4.80	4.18	0.69	1.33	1.15	1.00	1.20	29	53	1.1	57	0.42	0.17	2.62	0.543	COARSE	86	1.25	0.92	0.474	2.837	0	>1.3		10		
B-19-05	50	45	Sand	15.0	46	8	SPT	No	5		500	6.00	4.75	0.65	1.33	1.15	1.00	1.20	29	54	1.1	58	0.35	0.17	3.19	0.475	COARSE	86	1.25	0.85	0.449	2.735	0	>1.3		10		
B-19-05	60	63	Clay	90.0	46	8	SPT	No	5		500	7.20	5.33	0.61	1.33	1.15	1.00	1.20	39	71	1.2	77	0.33	0.17	3.77	0.450	COARSE	99	1.25	0.79	0.454	67.917	0	>1.3		10		
B-19-05	70	100	Sand	15.0	46	8	SPT	No	5		500	8.40	5.90	0.58	1.33	1.15	1.00	1.20	58	107	1.1	113	0.32	0.17	4.34	0.438	COARSE	122	1.25	0.75	0.468	175.591	0	>1.3		10		
B-19-05	80	100	Sand	15.0	46	8	SPT	No	5		500	9.60	6.48	0.56	1.33	1.15	1.00	1.20	56	102	1.1	108	0.30	0.17	4.92	0.422	COARSE	119	1.25	0.71	0.474	115.121	0	>1.3		10		0.0

SHEET	1 OF 1
PROJECT	29405910.0
DATE	4/22/2019
COMPUTED BY	DN
CHECKED BY	NS

AECOM

PROJECT: LAC+USC WC Demolition

SUBJECT: LIQUEFACTION POTENTIAL EVALUATION: B-19-05

Reference: Seed, et.al., 2003, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework
FAT CLAYS (CH) and ELASTIC SILTS (MH) not considered in analyses

Soil Unit Weight (pcf)	=	120
Energy Ratio (%)	=	80
Design Probability in decimals	=	0.15
Peak Ground Acceleration (g)	=	1.095
Earthquake Magnitude	=	6.8

BORING	DEPTH	SPT-N	SOIL CLASSIF.	PERCENT FINES	DEPTH TO WATER DURING DRILLING (feet)	BOREHOLE DIAMETER (inches)	SAMPLE TYPE	LINER USED	DESIGN WATER LEVEL IN DEPTH (feet)	LIQUE-FIABLE FINE-GRAINED SOILS	AVERAGE SHEAR WAVE VELOCITY AT UPPER 40 FEET (feet/sec)	TOTAL PRESSURE	EFFECTIVE PRESSURE	DEPTH FACTOR	ENERGY FACTOR	BORING DIAMETER FACTOR	ROD FACTOR	SAMPLER FACTOR	NOR. PENETR.	MODIFIED PENETR.	FINES CORRECTION	PENETR. RESIST.	SHEAR MASS PARTICIPATION FACTOR	STD DEVIATION FOR r_d	EFFECTIVE PRESSURE DURING LIQUEFACTION	IN-SITU CYCLIC STRESS RATIO	SOIL TYPE	Dr			ADJUSTED CYCLIC STRESS RATIO	CYCLIC RESISTANCE RATIO	PROBABILITY OF LIQUEFACTION	FACTOR OF	VOLUM. STRAIN	LAYER THICKNESS	SETTLE LAYER	SETTLE TOTAL
B-19-05	10	3	Clay	85.0	30	8	SPT	No	5	No	500	1.20	1.20	1.29	1.33	1.15	0.80	1.10	4	5	1.5	8	0.90	0.05	0.89	0.866	COARSE	31	1.25	1.19	0.584	0.119	0	non-liquefiable		10		
B-19-05	20	9	Clay	85.0	30	8	SPT	No	5	No	500	2.40	2.40	0.91	1.33	1.15	0.95	1.10	8	13	1.3	17	0.73	0.09	1.46	0.852	COARSE	46	1.25	1.12	0.612	0.231	0	non-liquefiable		10		
B-19-05	30	37	Sand	15.0	30	8	SPT	No	5		500	3.60	3.60	0.75	1.33	1.15	0.95	1.20	28	48	1.1	52	0.54	0.13	2.04	0.682	COARSE	84	1.25	1.01	0.539	1.998	0	>1.3		10		
B-19-05	40	42	Sand	15.0	30	8	SPT	No	5		500	4.80	4.18	0.69	1.33	1.15	1.00	1.20	29	53	1.1	57	0.42	0.17	2.62	0.543	COARSE	86	1.25	0.92	0.474	2.837	0	>1.3		10		
B-19-05	50	45	Sand	15.0	30	8	SPT	No	5		500	6.00	4.75	0.65	1.33	1.15	1.00	1.20	29	54	1.1	58	0.35	0.17	3.19	0.475	COARSE	86	1.25	0.85	0.449	2.735	0	>1.3		10		
B-19-05	60	63	Clay	90.0	30	8	SPT	No	5		500	7.20	5.33	0.61	1.33	1.15	1.00	1.20	39	71	1.2	77	0.33	0.17	3.77	0.450	COARSE	99	1.25	0.79	0.454	67.917	0	>1.3		10		
B-19-05	70	100	Sand	15.0	30	8	SPT	No	5		500	8.40	5.90	0.58	1.33	1.15	1.00	1.20	58	107	1.1	113	0.32	0.17	4.34	0.438	COARSE	122	1.25	0.75	0.468	175.591	0	>1.3		10		
B-19-05	80	100	Sand	15.0	30	8	SPT	No	5		500	9.60	6.48	0.56	1.33	1.15	1.00	1.20	56	102	1.1	108	0.30	0.17	4.92	0.422	COARSE	119	1.25	0.71	0.474	115.121	0	>1.3		10		0.0

AECOM

PROJECT NO.	60595699
DATE	1/11/2016
COMPUTED BY	DN
CHECKED BY	NS

PROJECT: LAC+USC WC Demolition

SUBJECT: DRY SAND SETTLEMENT DUE TO EARTHQUAKE SHAKING

INPUT

CLAYS (CL,CH) and ELASTIC SILTS (MH) not considered in analyses

Cs for CAL (Sand)	=	0.55
Cs for CAL (Clay)	=	0.7
Atmospheric Pressure, Pa (ksf)	=	2.116
Energy Ratio (%)	=	80
Ground Surface Elevation (feet)	=	330
Depth of Groundwater (feet)	=	46
Peak Ground Acceleration (g)	=	1.095
Earthquake Magnitude M_w	=	6.8

BORING (No.)	SAMPLE ELEVATION (feet)	DEPTH (feet)	UNIT WEIGHT (pcf)	FIELD BLOWCOUNT (bl/ft)	SOIL CLASSIF.	PI	D_{50} (mm)	BOREHOLE DIAMETER (in)	SAMPLE TYPE	LINER USED	TOTAL PRESSURE (ksf)	EFFECTIVE PRESSURE (ksf)	DEPTH FACTOR Cn	ENERGY FACTOR Ce=ER/60	BORING DIAMETER FACTOR Cb	ROD FACTOR Cr	SAMPLER FACTOR Cs	MODIFIED PENETR. (N1)60	RELATIVE DENSITY Dr (%)	REDUCTION FACTOR Rd	ϕ (rad)	EFFECTIVE PRESSURE AT DESIGN WATER TABLE (ksf)	σ_m' (kPa)	Gmax (psf)	γ_{eff}*G_{sat}/G_{max} (%)	n(PI)	γ (%)	ε (%)	ε_{vn} (%)	LAYER (feet)	SETTLEMENT LAYER (in)	SETTLEMENT TOTAL (in)
B-19-05	320	10	120	3	Clay	30.0	0.075	8	SPT	NO	1.200	1.200	1.33	1.33	1.15	0.75	1.10	5.0		0.98	0.50	1.20	39	980114	0.09	5.81E-04	--	--	--	10		
B-19-05	310	20	120	9	Clay	30.0	0.075	8	SPT	NO	2.400	2.400	0.94	1.33	1.15	0.95	1.10	13.5		0.95	0.61	2.40	71	1839393	0.09	5.81E-04	--	--	--	10		
B-19-05	300	30	120	37	Sand	0.0	1.000	8	SPT	NO	3.600	3.600	0.77	1.33	1.15	0.95	1.20	49.6	91	0.93	0.70	3.60	99	3332701	0.07	0.00E+00	0.26	0.08	0.07	10	0.164	
B-19-05	290	40	120	42	Sand	0.0	1.000	8	SPT	NO	4.800	4.800	0.66	1.33	1.15	1.00	1.20	51.3	92	0.85	0.70	4.80	131	3892399	0.07	0.00E+00	0.22	0.07	0.06	10	0.137	
B-19-05	280	50	120	45	Sand	0.0	1.000	8	SPT	NO	6.000	5.376	0.63	1.33	1.15	1.00	1.20	51.9	93	0.77	0.70	5.38	147	4136291	0.08	0.00E+00	0.22	Below WT	Below WT	10		
B-19-05	270	60	120	63	Clay	55.0	0.075	8	SPT	NO	7.200	5.952	0.60	1.33	1.15	1.00	1.20	69.1		0.69	0.70	5.95	163	4786907	0.07	1.92E-03	--	--	--	10		
B-19-05	260	70	120	100	Sand	0.0	1.000	8	SPT	NO	8.400	6.528	0.57	1.33	1.15	1.00	1.20	104.8	100	0.60	0.70	6.53	179	5758559	0.06	0.00E+00	0.13	Below WT	Below WT	10		
B-19-05	250	80	120	100	Sand	0.0	1.000	8	SPT	NO	9.600	7.104	0.55	1.33	1.15	1.00	1.20	100.4	100	0.55	0.70	7.10	194	5923178	0.06	0.00E+00	0.12	Below WT	Below WT	10		**0.3**

AECOM

PROJECT NO.	60595699
DATE	1/11/2016
COMPUTED BY	DN
CHECKED BY	NS

PROJECT: LAC+USC WC Demolition

SUBJECT: DRY SAND SETTLEMENT DUE TO EARTHQUAKE SHAKING

INPUT

CLAYS (CL,CH) and ELASTIC SILTS (MH) not considered in analyses

Cs for CAL (Sand)	=	0.55
Cs for CAL (Clay)	=	0.7
Atmospheric Pressure, Pa (ksf)	=	2.116
Energy Ratio (%)	=	80
Ground Surface Elevation (feet)	=	330
Depth of Groundwater (feet)	=	30
Peak Ground Acceleration (g)	=	1.095
Earthquake Magnitude M_w	=	6.8

BORING (No.)	SAMPLE ELEVATION (feet)	DEPTH (feet)	UNIT WEIGHT (pcf)	FIELD BLOWCOUNT (bl/ft)	SOIL CLASSIF.	PI	D_{50} (mm)	BOREHOLE DIAMETER (in)	SAMPLE TYPE	LINER USED	TOTAL PRESSURE (ksf)	EFFECTIVE PRESSURE (ksf)	DEPTH FACTOR Cn	ENERGY FACTOR Ce=ER/60	BORING DIAMETER FACTOR Cb	ROD FACTOR Cr	SAMPLER FACTOR Cs	MODIFIED PENETR. (N1)60	RELATIVE DENSITY Dr (%)	REDUCTION FACTOR Rd	ϕ (rad)	EFFECTIVE PRESSURE AT DESIGN WATER TABLE (ksf)	σ_m' (kPa)	Gmax (psf)	γ_{eff}*G_{eff}/G_{max} (%)	n(PI)	γ (%)	ε (%)	ε_{1D} (%)	LAYER (feet)	SETTLEMENT LAYER (in)	SETTLEMENT TOTAL (in)
B-19-05	320	10	120	3	Clay	30.0	0.075	8	SPT	NO	1.200	1.200	1.33	1.33	1.15	0.75	1.10	5.0		0.98	0.50	1.20	39	980114	0.09	5.81E-04	--	--	--	10		
B-19-05	310	20	120	9	Clay	30.0	0.075	8	SPT	NO	2.400	2.400	0.94	1.33	1.15	0.95	1.10	13.5		0.95	0.61	2.40	71	1839393	0.09	5.81E-04	--	--	--	10		
B-19-05	300	30	120	37	Sand	0.0	1.000	8	SPT	NO	3.600	3.600	0.77	1.33	1.15	0.95	1.20	49.6	91	0.93	0.70	3.60	99	3332701	0.07	0.00E+00	0.26	0.08	0.07	10	0.164	
B-19-05	290	40	120	42	Sand	0.0	1.000	8	SPT	NO	4.800	4.800	0.66	1.33	1.15	1.00	1.20	51.3	92	0.85	0.70	4.80	131	3892399	0.07	0.00E+00	0.22	0.07	0.06	10	0.137	
B-19-05	280	50	120	45	Sand	0.0	1.000	8	SPT	NO	6.000	5.376	0.63	1.33	1.15	1.00	1.20	51.9	93	0.77	0.70	5.38	147	4136291	0.08	0.00E+00	0.22	Below WT	Below WT	10		
B-19-05	270	60	120	63	Clay	55.0	0.075	8	SPT	NO	7.200	5.952	0.60	1.33	1.15	1.00	1.20	69.1		0.69	0.70	5.95	163	4786907	0.07	1.92E-03	--	--	--	10		
B-19-05	260	70	120	100	Sand	0.0	1.000	8	SPT	NO	8.400	6.528	0.57	1.33	1.15	1.00	1.20	104.8	100	0.60	0.70	6.53	179	5758559	0.06	0.00E+00	0.13	Below WT	Below WT	10		
B-19-05	250	80	120	100	Sand	0.0	1.000	8	SPT	NO	9.600	7.104	0.55	1.33	1.15	1.00	1.20	100.4	100	0.55	0.70	7.10	194	5923178	0.06	0.00E+00	0.12	Below WT	Below WT	10		**0.3**

INTRODUCTION

This appendix summarizes the slope stability analyses for the temporary cut and fill slopes for the grading operation of the "Make-Ready" package. The subsurface conditions for these slope stability analyses were based on field investigation and laboratory testing results.

SLOPE STABILITY ANALYSIS PROCEDURE

Slope stability analyses were performed for static and pseudo-static for circular failure surfaces. The Slope/W 2007 (Geo-Slope International, 2007) computer program calculates the factor of safety (FS) against instability of a slope by the method of slices using a two-dimensional, limit-equilibrium method. The Spencer method was used in these analyses.

The input parameters for the Slope/W computer program include slope geometry, soil unit weight, and soil shear strength. The shear strength parameters used in the analyses are summarized in Tables F1 through F2. The analysis method and basic geometry (entry, exit points) for a failure surface are specified by the user. The soil geometry and shear strength parameters are also specified by the user. The computer program then generates failure surfaces and calculates the FS against sliding. It should be noted that surcharge of 240 psf was used in the analyses for the equipment/traffic loading during construction.

TABLE F1 - GENERALIZED SOIL PROFILE - AREA A - COARSE-GRAINED SOIL

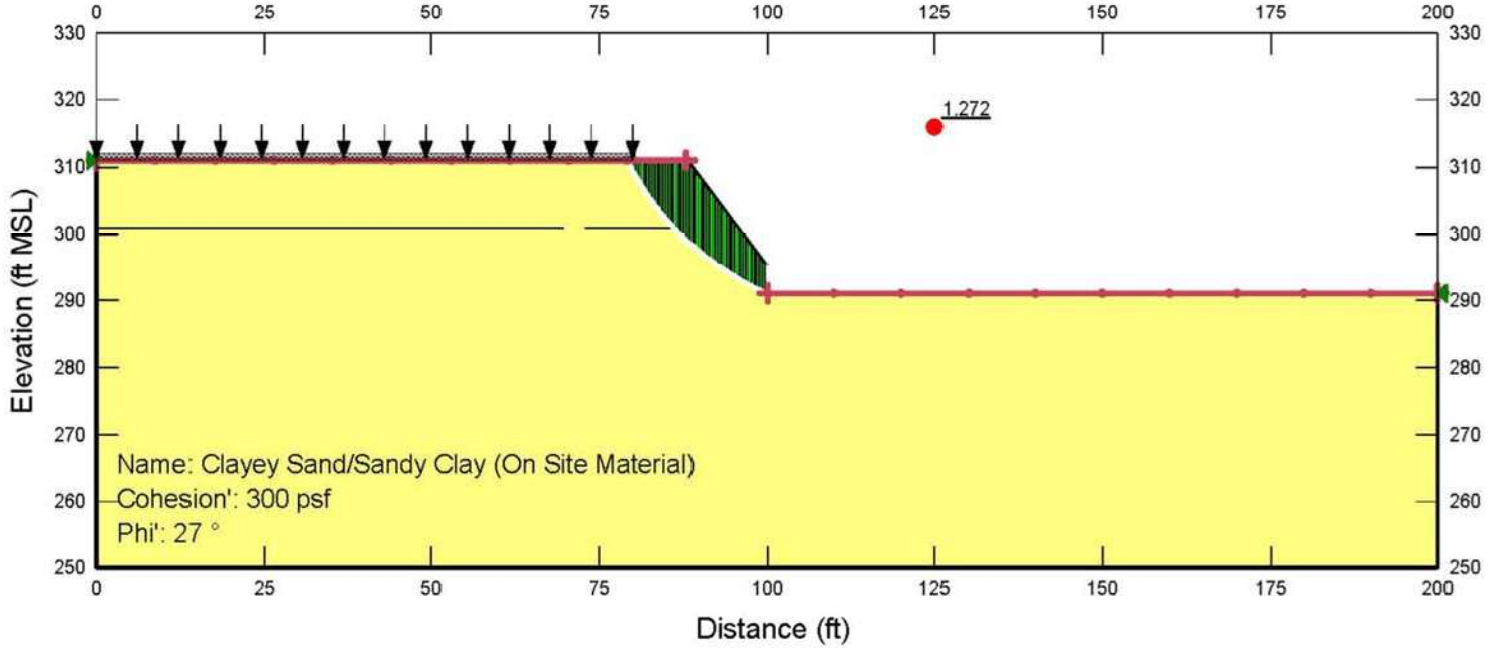
SOIL TYPE	ELEVATION[1] (feet MSL)	EFFECTIVE UNIT WEIGHT (lb/foot^3)	INTERNAL FRICTION ANGLE (degrees)	COHESION (lb/foot^2)	UNDRAINED SHEAR STRENGTH (lb/foot^2)
On Site	330 to 250	125	27	300	--
Aggregate Base	330 to 250	135	38	150	--

ANALYSIS RESULTS

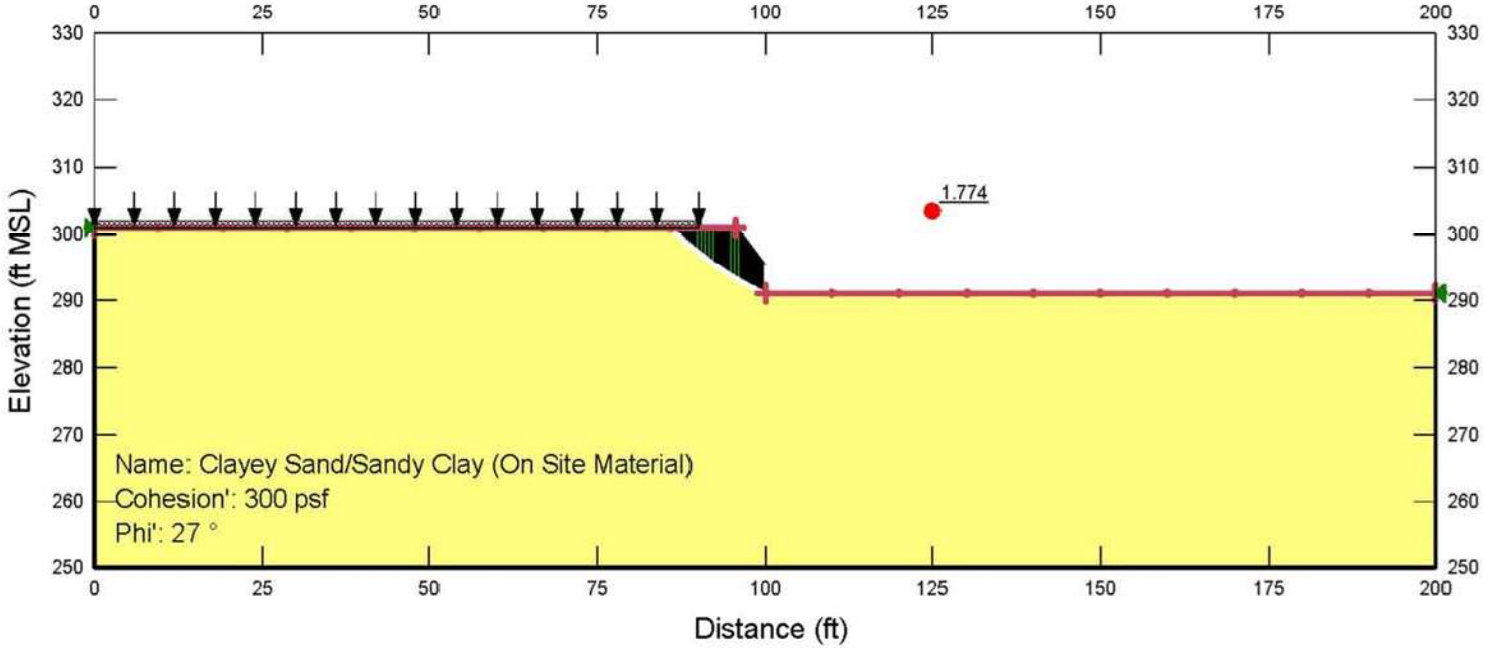
Summaries of the slope stability analysis results are provided in Table D6. Graphic outputs of the analyses performed are attached at the end of this appendix.

TABLE F2 - SUMMARY OF SLOPE STABILITY ANALYSES

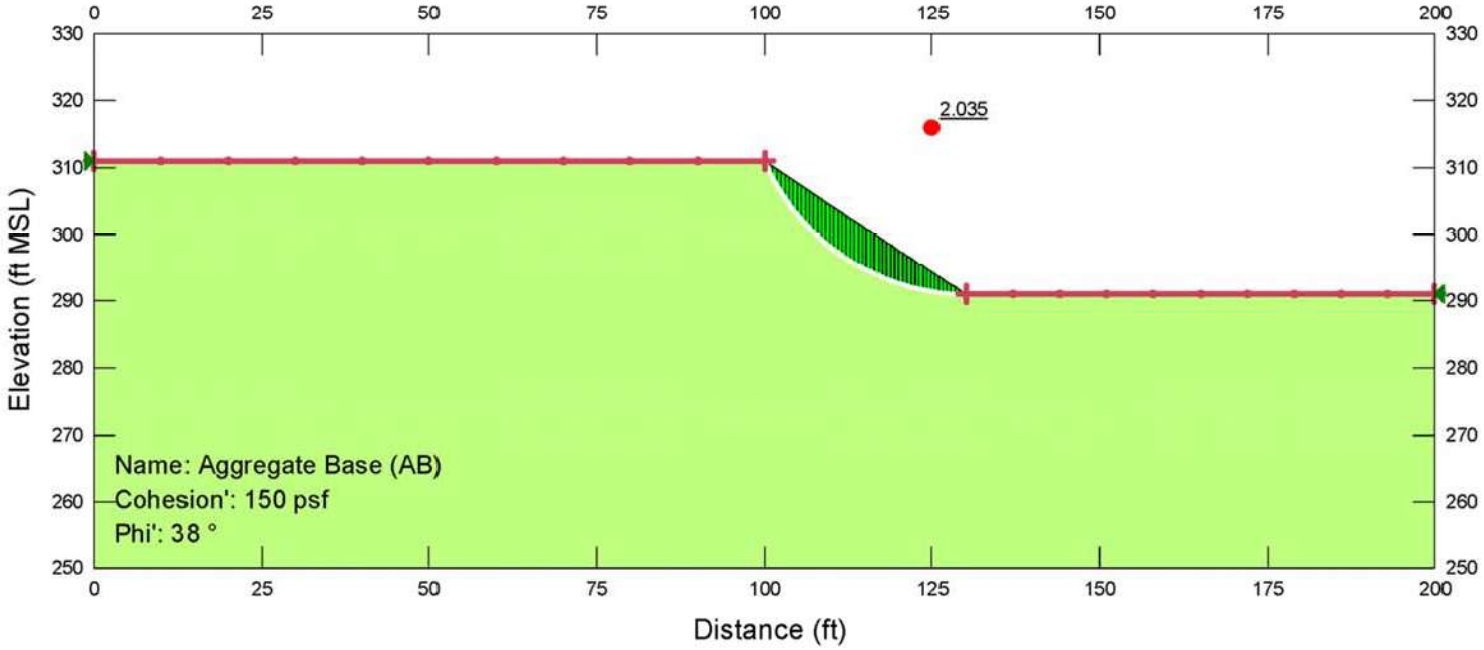
CASE	SLOPE GRADIENT	SLOPE HEIGHT (feet)	REQUIRED FACTOR OF SAFETY - TEMPORARY/ INTERIM CONDITIONS	FACTOR OF SAFETY
10-foot Excavation	5	1:1 (H:V)	1.25	1.94
20-foot Excavation	5	1:1 (H:V)	1.25	1.29
Interim Static	0	1.25:1 (H:V)	1.5	1.82
Interim Pseudostatic	0	1.25:1 (H:V)	1.1	1.08
Interim Static	0	1.5:1 (H:V)	1.5	2.03
Interim Pseudostatic	0	1.5:1 (H:V)	1.1	1.17



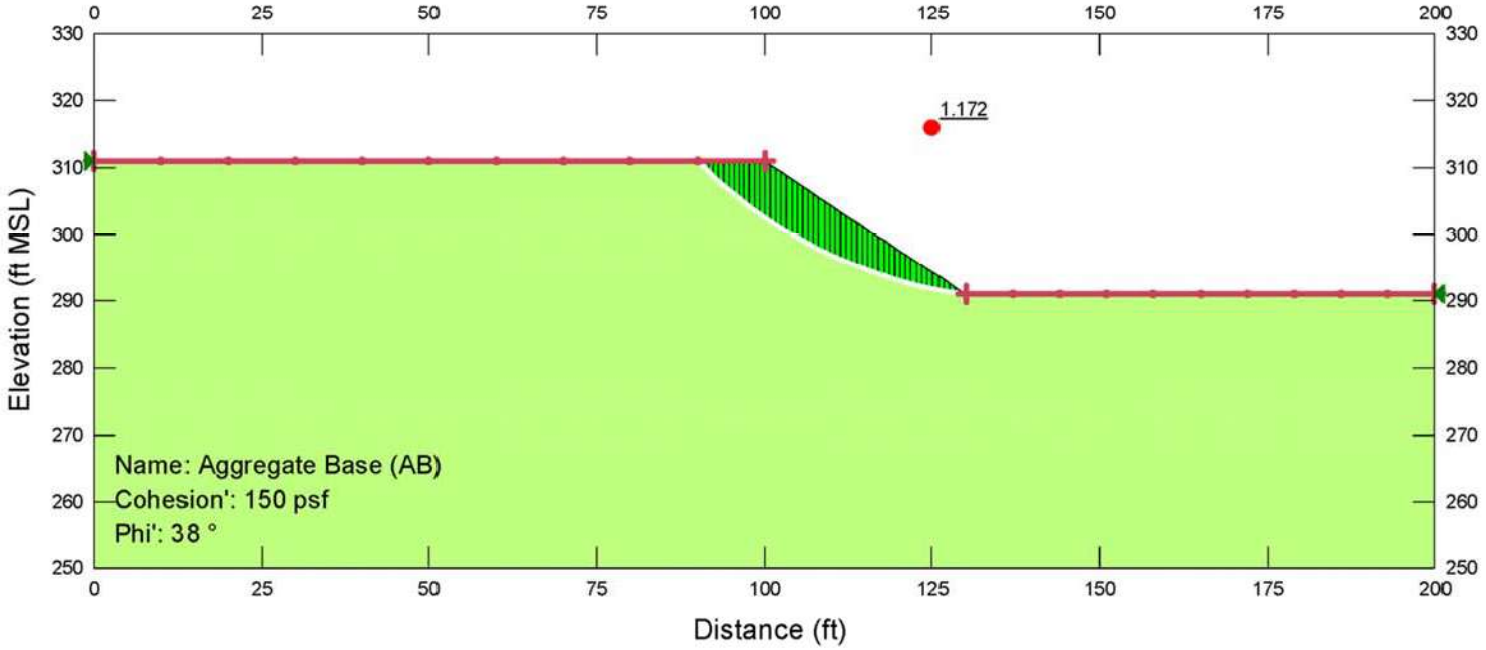
TEMPORARY SLOPE STABILITY: 20 FEET
Geotechnical Bulk Factor Report
Los Angeles County Public Works
Los Angeles, California



TEMPORARY SLOPE STABILITY: 10 FEET
Geotechnical Bulk Factor Report
Los Angeles County Public Works
Los Angeles, California

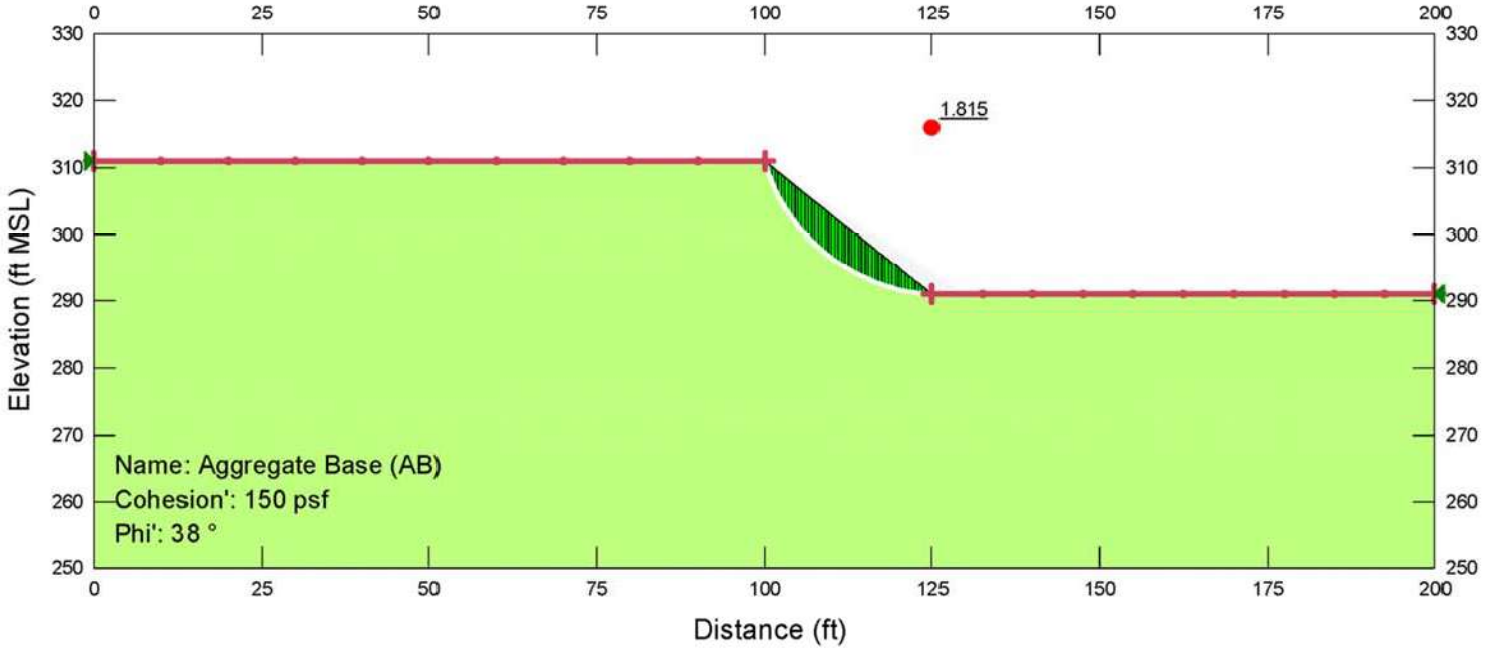


PERMANENT STATIC SLOPE STABILITY: 1.5H:1V
Geotechnical Bulk Factor Report
Los Angeles County Public Works
Los Angeles, California

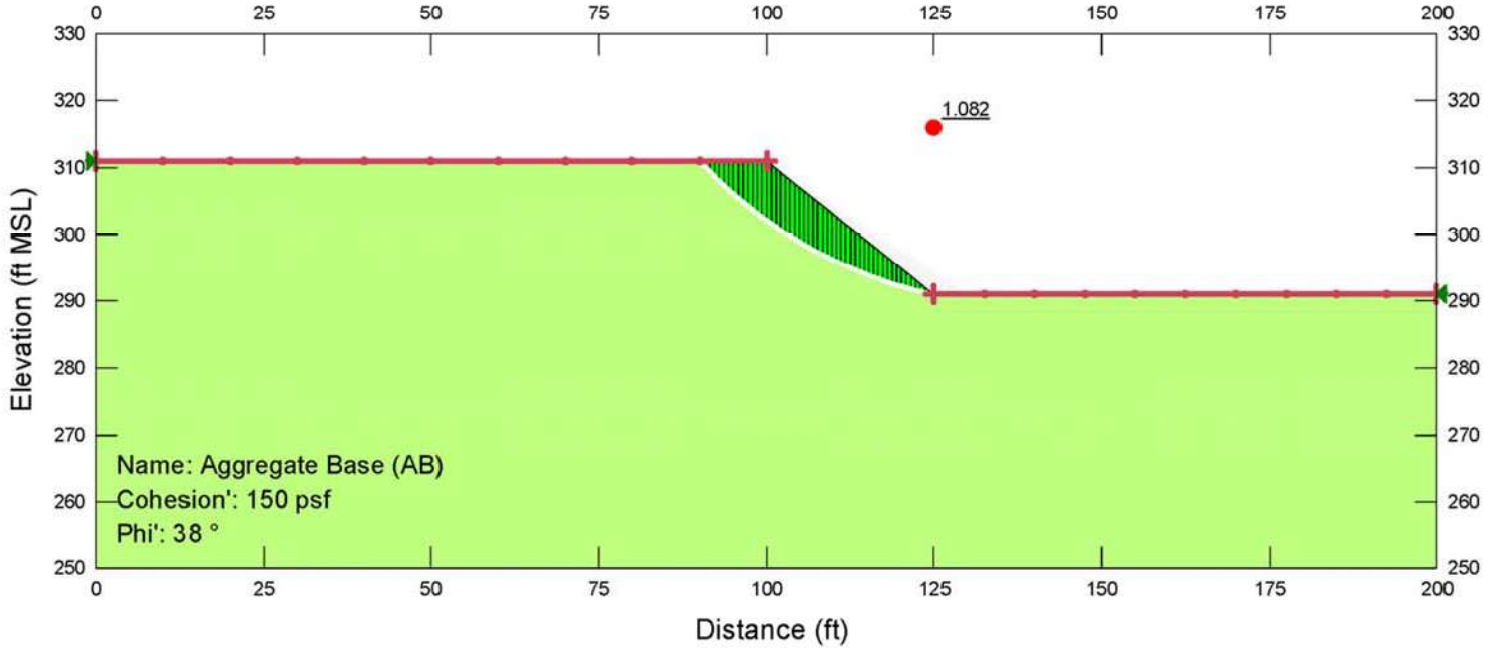


Horizontal Seismic Coefficient: 0.324

PERMANENT PSEUDOSTATIC SLOPE STABILITY: 1.5H:1V
Geotechnical Bulk Factor Report
Los Angeles County Public Works
Los Angeles, California



PERMANENT STATIC SLOPE STABILITY: 1.25H:1V
Geotechnical Bulk Factor Report
Los Angeles County Public Works
Los Angeles, California



Horizontal Seismic Coefficient: 0.324

PERMANENT PSEUDOSTATIC SLOPE STABILITY: 1.25H:1V
Geotechnical Bulk Factor Report
Los Angeles County Public Works
Los Angeles, California

APPENDIX F.5

REPORT OF GEOTECHNICAL INVESTIGATION FOR STRUCTURAL CONDITION ASSESSMENT AND SEISMIC EVALUATION, LAC+USC MEDICAL CENTER: OLD PHARMACY WAREHOUSE PROJECT

GEOTECHNICAL PROFESSIONALS INC.

REPORT OF GEOTECHNICAL INVESTIGATION FOR STRUCTURAL CONDITION ASSESSMENT AND SEISMIC EVALUATION LAC+USC MEDICAL CENTER - OLD PHARMACY WAREHOUSE PROJECT PROJECT ID 00001694, PROJECT P96184HR 1100 N. MISSION ROAD LOS ANGELES, CALIFORNIA

Prepared for:
County of Los Angeles Department of Public Works
Project Management Division I
900 S. Fremont Avenue, 5th Floor
Alhambra, California 91803-1331

Prepared by:
Geotechnical Professionals Inc.
5736 Corporate Avenue
Cypress, California 90630
(714) 220-2211

GPI Project No. 3080.22I November 7, 2022

GEOTECHNICAL PROFESSIONALS INC.

November 7, 2022

County of Los Angeles Department of Public Works
Project Management Division I
900 S. Fremont Avenue, 5th Floor
Alhambra, California 91803-1331

Attention: Mr. Aren Avakian, CCM
Capital Projects Management Associate

Subject: Report of Geotechnical Investigation for
Structural Condition Assessment and Seismic Evaluation
LAC+USC Medical Center - Old Pharmacy Warehouse Project
1100 N. Mission Road
Los Angeles, California
GPI Project No. 3080.22I
Project I.D. 00001694, Project P96184HR

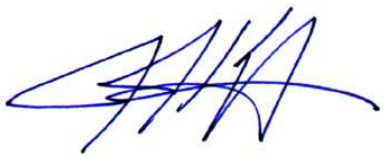
Dear Aren:

Transmitted herewith is an electronic copy of our geotechnical investigation report for the subject project. The report presents the results of our evaluation of the subsurface and seismic conditions at the site with respect to the planned building retrofit. We understand that County will distribute this report to the appropriate Design Team members and regulatory agencies. We can also provide wet-signed originals of this report to the County upon request.

We appreciate the opportunity of providing our services to the County and look forward to seeing the project through its successful completion. Please do not hesitate to call us if you have any questions on the contents of our report or need further geotechnical assistance.

Very truly yours,
Geotechnical Professionals Inc.





Dylan J. Boyle, G.E.
Senior Engineer

Justin J. Kempton, G.E.
Principal

Distribution: Addressee

5736 Corporate Avenue • Cypress, CA 90630 • (714) 220-2211 , FAX (714) 220-2122

TABLE OF CONTENTS

	PAGE
1.0 INTRODUCTION	1
1.1 GENERAL	1
1.2 PROJECT DESCRIPTION	1
1.3 PURPOSE OF INVESTIGATION	2
1.4 PRIOR INVESTIGATIONS BY OTHERS	2
1.5 AS-BUILT PLANS	3
2.0 SCOPE OF WORK	4
3.0 SITE CONDITIONS	5
3.1 SURFACE CONDITIONS	5
3.2 SUBSURFACE SOIL AND BEDROCK CONDITIONS	6
3.3 GROUNDWATER AND CAVING	7
3.4 GEOLOGIC - SEISMIC HAZARDS	8
4.0 CONCLUSIONS AND RECOMMENDATIONS	9
4.1 OVERVIEW	9
4.2 SEISMIC DESIGN	10
4.2.1 General	10
4.2.2 Site-Specific Ground Motion Analyses	12
4.2.3 Liquefaction, Lateral Spreading, and Seismic Settlement	14
4.3 EARTHWORK	15
4.3.1 Demolition and Clearing	16
4.3.2 Excavations	16
4.3.3 Subgrade Preparation	18
4.3.4 Material for Fill	18
4.3.5 Placement and Compaction of Fills	19
4.3.6 Shrinkage and Subsidence	20
4.3.7 Trench/Wall Backfill	20
4.3.8 Observation and Testing	20
4.4 FOUNDATIONS	21
4.4.1 General	21
4.4.2 Spread Footings	21
4.4.3 Deep Foundations	23
4.4.4 Foundation Concrete	27
4.4.5 Foundation Construction Observation	27
4.5 RETAINING STRUCTURES AND SHORING	27
4.5.1 Basement and Retaining Walls	28
4.5.2 Temporary Shoring	29
4.6 CONCRETE SLABS	30
4.7 EXTERIOR CONCRETE AND MASONRY FLATWORK	31
4.8 PAVED AREAS	31
4.9 CORROSION	33
4.10 SURFACE DRAINAGE AND INFILTRATION	33
4.11 GEOTECHNICAL OBSERVATION AND TESTING	33
5.0 LIMITATIONS	34
REFERENCES	

APPENDICES

A EXPLORATORY BORINGS
B LABORATORY TESTS
C GEOLOGIC-SEISMIC EVALUATION
D SEISMIC SHEAR-WAVE SURVEY

LIST OF FIGURES

FIGURE NO.

1	Site Location Map
2	Site Plan
3	Subsurface Cross Section
4	Site-Specific Probabilistic & Deterministic (MCE_R) Response Spectra @ 5% Damping
5	Site-Specific BSE-2N and BSE-1N Response Spectra
6	Site-Specific BSE-2E and BSE-1E Response Spectra
7	Site-Specific BSE-2N, BSE-1N, BSE-2E, and BSE-1E Response Spectra
8A-8F	Lateral Pile Analysis – 24-inch Diameter
9A-9F	Lateral Pile Analysis – 30-inch Diameter
10A-10F	Lateral Pile Analysis – 36-inch Diameter

TABLE NO.

1	ASCE 41-17 Site-Specific Seismic Response Spectra Worksheet – New Buildings
2	ASCE 41-17 Site-Specific Seismic Response Spectra Worksheet – Existing Buildings

APPENDIX A

A-1 to A-3	Logs of Borings

APPENDIX B

B-1	Atterberg Limits Test Results
B-2 to B-5	Direct Shear Test Results
B-6 to B-8	Consolidation Test Results
Table 1	HDR Laboratory Tests on Soil Samples

APPENDIX C

C-1	Regional Geologic Map
C-2	Geologic Cross Section
C-3	Historic Groundwater Map
C-4	Regional Fault Map
C-5	Regional Seismicity
C-6	Seismic Hazard Zones Map

1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for the proposed structural condition assessment and seismic evaluation of the Old Pharmacy Warehouse building at the LAC+USC Medical Center in Los Angeles, California. The site location is shown on the Site Location Map, Figure 1.

In accordance with our approved scope, a detailed geologic-seismic evaluation was performed for the project, including site-specific response spectra, as required by the 2019 California Building Code (CBC) and in accordance with both ASCE 41-17 and ASCE 7-16.

1.2 PROJECT DESCRIPTION

The Project Structural Engineer is performing a structural condition assessment and seismic evaluation of the existing Old Pharmacy Warehouse (OPW) building. The OPW consists of five stories, including a partial basement as well as roof top entrance structures. The partial basement is one-level below grade on the north side of the building and at-grade on the south side. The OPW has an approximate footprint of 8,100 square feet (roughly 110 feet by 73 feet). Based on our review of as-built records, we understand the OPW was constructed in 1916 and received major upgrades in 1953. We understand a minor portion of the OPW is currently occupied, with the remainder of the structure being used as a warehouse for storage.

The existing site conditions are shown on the Site Plan, Figure 2.

Based on our discussions with the Project Team, we understand the Project Structural Engineer is requesting geotechnical assessments and recommendations including, but not limited to, the following items:

- Site-specific seismic parameters in accordance with the requirements of the 2019 California Building Code (CBC), ASCE 7-16, and ASCE 41-17.
- Evaluation of the geotechnical parameters for support of the existing foundations, including bearing capacity and settlement.
- Recommended geotechnical parameters for new foundations, including shallow footings and pile foundations.
- Earthwork criteria for planned remediation of the existing structure.

GPI has not yet been provided with anticipated structural loads (dead plus live or seismic) associated with the planned retrofit. We anticipate the planned improvements will not significantly impact the existing grades at the subject site.

Our recommendations are based upon the above-assumed structural and finish grade information. We should be notified if the actual loads and/or grades differ or change during the project design to allow our office to either confirm or modify our recommendations. Also, when the project grading and foundation plans become available, we should be provided with a copy for review and comment.

1.3 PURPOSE OF INVESTIGATION

The primary purpose of this investigation and report is to provide an evaluation of the existing geotechnical and geologic conditions at the site as they relate to the assessment of the existing structure and the design and construction of the proposed retrofit.

1.4 PRIOR INVESTIGATIONS BY OTHERS

GPI was provided with the following prior geotechnical reports for nearby projects by others:

- AECOM, “Report of Geotechnical Investigation and Bulk Factor, Women’s and Children’s Hospital Demolition, LAC+USC Medical Center, 1240 North Mission Road, Los Angeles, California”, AECOM Project No. 60597019, dated October 21, 2019
- Geotechnologies, Inc., “Geotechnical Engineering Investigation, Proposed Exterior Elevator, LAC+USC Medical Center, 2051 Marengo Street, Los Angeles, California”, Geotechnologies File No. 21832, dated August 12, 2019
- RMA Group, “Geotechnical Investigation for Proposed L.A. Co. – USC Medical Center Child Care Project, N. State Street between Zonal Avenue and Marengo Street, Los Angeles, CA,” RMA Project No. 18-0056-01, dated March 30, 2018

The above-referenced report by AECOM covered the demolition of the previously existing Women’s and Children’s (WC) Hospital that was located approximately 450 feet northeast of the subject OPW building. The previously existing WC Hospital included a basement that was infilled as part of the demolition process. Following demolition, the WC Hospital site will be converted to a surface parking lot. Based on our review of historical aerial photographs, it appears as though the demolition and rough grading of the WC Hospital site occurred between September 2020 and August 2021.

The above-referenced report by Geotechnologies covered the design and construction of a proposed two-story exterior elevator structure at the LAC+USC Medical Center. The proposed elevator structure is located approximately 1,800 feet to the southeast of the subject OPW building. Based on our review of historic aerial photographs, it is unclear as to whether this elevator structure has since been constructed.

The above-referenced report by RMA Group covered a proposed child-care facility for the LAC+USC Medical Center to be located on the west side of North State Street and approximately 500 feet southeast of the subject OPW building. The proposed child-care facility included one-story buildings having a combined approximately footprint of 5,700

square feet as well as play yards covering an approximate 9,000 square-foot area. The childcare facility would be constructed over an existing utility tunnel extending through the site, with the utility tunnel also trending towards the subject OPW building. Based on our review of historic aerial photographs, it appears as though construction of the child-care facility is ongoing.

Subsurface information and laboratory testing from the above referenced reports of nearby sites was reviewed and referenced when planning our field investigation and developing our recommendations for the subject project.

1.5 AS-BUILT PLANS

GPI was provided with the as-built foundation plan and building sections for the “Service Building at County Hospital Grounds” dated May 8, 1916. The south building section from the 1916 as-built plans was incorporated into the Subsurface Cross Section, Figure 3.

GPI has also been provided with building alteration plans for the OPW from 1953 and 1954. These plans were referenced when developing our findings and recommendations included herein.

2.0 SCOPE OF WORK

Our scope of work included field explorations, laboratory testing, site-specific geologic and seismic evaluation, foundation analyses, and preparation of this report.

Our field investigation consisted of three exploratory hollow stem auger (HSA) borings and a seismic shear-wave study. The HSA borings were performed to depths of 61½ to 81½ feet below existing grades at the locations shown on Figure 2. Descriptions of the field procedures and logs of the borings are presented in Appendix A.

Our laboratory testing program included evaluations of in-place moisture content, Atterberg Limits, direct shear, consolidation, expansion index, fines content, maximum dry density and optimum moisture content, and corrosivity. Laboratory test procedures and results are presented in Appendix B.

Soil corrosivity testing was performed by HDR under subcontract to GPI. The corrosivity test results are presented in Appendix B.

An evaluation of geologic and seismic hazards is presented in Appendix C.

A seismic shear-wave study was performed by Terra Geosciences under subcontract to GPI. Two seismic shear-wave lines were conducted on September 17, 2021 to a depth of at least 100 feet below the existing ground surface and at the locations shown on Figure 2. Details and results of the seismic shear-wave survey are presented in Appendix D.

Engineering evaluations were performed to provide earthwork criteria, foundation and slab design parameters, and assessments of seismic hazards. The results of our evaluations are presented in the remainder of the report.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The LAC+USC Medical Center campus is roughly 78 acres in size and located northeast of the intersection of the I-5 and I-10 freeways within the northern portion of the Boyle Heights neighborhood of the City of Los Angeles. The overall medical center campus is bounded on the southwest by Marengo Street, on the northwest by N. Mission Road, on the northeast by Zonal Avenue, and on the southeast by N. Chicago Street. The subject OPW building is located within the northwestern portion of the medical center campus.

The OPW building is located approximately 340 feet southeast of N. Mission Road and approximately 280 feet northwest of Marengo Street. The existing OPW building consists of five above-grade stories plus a partial basement and roof-top entrance structures. The partial basement level is below grade on the northwest side of the OPW building and at-grade on the southeast side of the OPW building. The footprint of the OPW building is approximately 8,100 square feet (110 feet by 73 feet). Based on our review of the as-built plans previously referenced, the OPW building was constructed in 1916 and received a major upgrade in 1953. At this time, a minor portion of the building is occupied with the remainder of the building being used for storage.

We reviewed historical aerial photographs of the site dating back to 1948. While the overall medical center campus has experienced change since 1948, the subject OPW building appears to have remained relatively the same.

In general, the topography of the Medical Center complex slopes down towards the central portion of the campus with ground surface elevations generally ranging from roughly +325 feet along N. Mission Road, to a low of roughly +300 feet along Marengo Street, back up to roughly +350 feet along N. State Street, and generally continuing to increase in elevation towards the southeast. The low elevation areas across the campus appear to correspond to a previous drainage channel associated with Lincoln Park Lake (located to the northeast) which flows down to the southwest towards the intersection of the I-5 and I-10 freeways.

The OPW building spans a grade change at the site. On the north side of the subject building, ground surface elevations generally range from approximately +323 to +325 feet. On the south side of the subject building, ground surface elevations generally range from +305 to +306 feet. Because of the grade change, the bottom floor of the subject building is a partial basement that is one-level below grades on the north side of the building and at-grade on the south side of the building. To the east and west of the subject building, the elevation differentials are separated by retaining walls ranging in height from approximately 15 to 20 feet.

The pavement sections encountered at GPI's explorations consisted of 4 to 6 inches of asphalt concrete over 0 to 4 inches of aggregate base. In general, the asphalt on the north side of the OPW building was in poor to fair condition, while the asphalt on the

south side of the building was in very poor condition, exhibiting significant cracking, spalling, and full deterioration in select areas.

3.2 SUBSURFACE SOIL AND BEDROCK CONDITIONS

Our understanding of the subsurface conditions at the site are primarily based on our field investigation, as well as our review of prior nearby subsurface investigations by others (AECOM, 2019; Geotechnologies, 2019; RMA Group, 2018).

Our field investigation disclosed a subsurface profile consisting of undocumented fills overlying natural younger and older alluvial soils and, at depth, Puente Formation bedrock materials. Detailed descriptions of the conditions encountered are shown on the Logs of Borings in Appendix A.

The subject site is located in a Methane Zone as designated by the City of Los Angeles. We recommend that an engineering firm specializing in methane evaluation and mitigation be consulted if methane mitigation recommendations are required.

Undocumented Fills

We encountered undocumented fill soils extending to depths of approximately 10 feet on the north side of the building and approximately 15 to 18 feet below existing grades on the south side of the building. In general, the undocumented fills consisted of loose to medium dense silty sands and firm to stiff silty clays and sandy clays. The undocumented fills encountered on the south side of the OPW building also contained debris consisting of concrete, lumber, brick, and glass fragments. Based upon the age of the existing OPW building, we anticipate the undocumented fills are relatively old.

North of OPW Building

On the north side of the building (Boring B-1), we encountered roughly 5 feet of younger alluvial clays and sands overlying approximately 20 feet of older alluvial clays and sands. The younger alluvial materials were generally loose and stiff while the older alluvial soils were generally dense to very dense and very stiff to hard. The moisture contents of the alluvial soils generally ranged from roughly 2 to 25 percent (average of about 10 percent), with higher moisture contents measured in the finer grained soils.

We encountered Puente Formation siltstone and sandstone bedrock at a depth of approximately 35 feet in Boring B-1, which corresponds to an approximate elevation of +289 feet. In general, the bedrock materials were found to be weathered near the contact with the alluvial soils, becoming less weathered with depth. The bedrock materials were generally moist to wet, with moisture contents averaging about 15 percent. Sampler blowcounts indicate the bedrock material is dense to very dense and hard, based on soil consistency terminology instead of rock terminology. Although not tested, the siltstone bedrock materials are expected to exhibit a moderate to high potential for expansion when processed and recompacted and would be expected to shrink and swell with changes in moisture content.

South of OPW Building

On the southwest side of the building (Boring B-2), we encountered roughly 10 feet of younger alluvial sandy clays underlying the undocumented fills. We did not encounter natural alluvial soils (younger or older) between the undocumented fills and the underlying bedrock on the southeast side of the building (Boring B-3). Where encountered, the younger alluvial materials were generally firm to stiff, with moisture contents ranging from 21 to 29 percent (average of 25 percent).

We encountered Puente Formation siltstone and sandstone bedrock at depths of approximately 25 feet in Boring B-2 and 18 feet in Boring B-3, corresponding to approximate elevations of +280 and +287 feet, respectively. As with the bedrock materials on the north side of the building, the sandstone and siltstone was found to be weathered near the contact with the alluvial soils, becoming less weathered with depth. The bedrock materials were generally very moist to wet, with moisture contents averaging about 17 percent. Sampler blowcounts indicate the bedrock material is dense to very dense and hard, based on soil consistency terminology instead of rock terminology.

Expansive Soils/Bedrock

Laboratory testing of the sandy and silty clays and clayey silts (undocumented fill and alluvial soils) indicated a low to moderate potential for expansion with changes in moisture content (Expansion Indices [EIs] of 11 to 23 and Plasticity Indices [PIs] of 15 to 17). The siltstone and claystone bedrock materials are also anticipated to have low to moderate potential for expansion when processed and recompacted.

3.3 GROUNDWATER AND CAVING

Groundwater was encountered at a depth of 54 feet below existing grades in our Boring B-1, located on the north side of the OPW building (roughly corresponding to Elevation +271 feet). In our explorations located on the south side of the OPW building (Borings B-2 and B-3), we encountered groundwater and depths of 25 to 35 feet below existing grades (roughly corresponding to Elevations +280 and +270 feet).

Historical data published by the State indicates that the historical shallowest groundwater depth in the site vicinity to be approximately 20 feet below prevailing site grades.

Prior nearby explorations by AECOM (2019), located roughly 500 feet east/northeast of the subject site, encountered groundwater between depths of 32 and 38 feet below site grades (roughly corresponding to Elevations +287 to +296 feet). Prior nearby explorations by RMA Group (2018), located roughly 500 feet to the southeast of the subject site, did not encounter groundwater within 45 feet of existing grades (deeper than Elevation +311 feet). Prior nearby explorations by Geotechnologies, Inc. (2019), located roughly 1,700 feet to the southeast did not encounter groundwater within the 40-foot depth explored (deeper than Elevation +325 feet).

Caving was not encountered in our relatively small diameter borings and would not be expected in the fine-grained subsurface soils. Minor caving should be anticipated in the upper, sandy alluvial soils.

3.4 GEOLOGIC - SEISMIC HAZARDS

A detailed evaluation of the geologic conditions at the site, including seismic hazards, is presented in Appendix C. Ground motion and seismic settlement is addressed in a following section of this report.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 OVERVIEW

Based on the results of our investigation, it is our opinion that from a geotechnical viewpoint it is feasible to update and retrofit the site as proposed. Structural improvements and seismic retrofit to the existing OPW building can be supported on either new or enlarged shallow foundations or new deep foundations provided the geotechnical constraints discussed below are mitigated. The most significant geotechnical issues that will affect the design and construction of the proposed structures are as follows:

- The thickness of the existing undocumented (artificial) fill soils encountered in our recent borings is variable. Undocumented fills were encountered up to depths of 10 feet below existing grades on the north side of the OPW building and up to depths of 18 feet below grades on the south side of the OPW building.

- For new and enlarged shallow foundations within the existing OPW building pad, the extent of remedial grading will depend on the subsurface conditions encountered. For planning purposes, these foundations should be underlain by at least 1-foot of new properly compacted crushed aggregate base (CAB) materials. Deeper removals may be required depending on the actual subgrade conditions exposed in the foundation excavations. Additional details are provided in the “Earthwork” section of this report.

- To provide uniform support of minor at-grade structures and new hardscape outside the OPW building pad, we recommend removal and recompaction of a portion of the upper fill soils.

- Deep foundations consisting of cast-in-drilled-hole (CIDH) piles may also be used to support increased seismic demands on the OPW building. The CIDH piles should extend through the undocumented fills and alluvial soils and into the underlying bedrock materials. Additional details are provided in the “Foundations” section of this report.

- We anticipate the proposed improvements to the OPW building will not result in an increase in the building’s static loads. As such, we anticipate new static settlements associated with the proposed improvements to be negligible, provided the new foundations (new or enlarged shallow foundations or deep foundations) are sized and constructed in accordance with our recommendations.

- The moisture contents of the upper fill soils are generally moist to wet and above the optimum moisture content of approximately 10 percent. These materials will likely require moisture conditioning (drying) prior to reuse as

properly compacted fill. Where over-optimum clayey fill soils are exposed during grading, drying or stabilization may be required to achieve the required degree of compaction to support additional fill soils or pavements. Earthwork subcontractors should review the moisture conditions when planning the work.

- The upper clayey and silty fill and alluvial soils exhibited a low to medium potential for expansion (EIs of 11 to 23 and PIs of 15 to 17) and would be expected to shrink and swell with significant changes in moisture content. As such, these soils are not considered to be suitable for use as retaining wall backfill or direct support of building floor slabs and hardscape. In addition, the on-site clayey soils will require moisture conditioning when being placed as compacted fill and should not be allowed to dry-out prior to covering.

- The OPW building is located on the edge, and outside of, an area designated by the State of California as having a potential for soil liquefaction. Based on our analyses, we computed negligible total and differential (across a span of 40 feet) seismic-induced liquefaction settlements (less than ¼ inch).

- Corrosivity testing performed by HDR on samples provided from our borings indicates negligible levels of soluble sulfate and chloride content with respect to concrete. The soils are also considered to be corrosive to buried ferrous metals.

- The site is located in a Methane Zone as designated by the City of Los Angeles. We recommend that an engineering firm specializing in methane evaluation and mitigation be consulted if methane mitigation recommendations are required.

Our recommendations related to the geotechnical aspects of the development of the site are presented in the subsequent sections of this report.

4.2 SEISMIC DESIGN

4.2.1 General

Details of our geologic and seismic evaluation for the site are presented in Appendix C.

As stated previously, GPI retained Terra Geosciences to perform a seismic shear-wave survey to a depth of at least 100 feet below the existing ground surface (see Appendix D). Terra performed two seismic lines to measure shear-wave velocity at the site: one located on the north side of the building and one located on the south side of the building. The locations of the seismic lines are shown on the Site Plan, Figure 2. Analysis of the data obtained during the seismic shear-wave survey indicate that the “weighted average” shear-wave velocity within the upper 100 feet of the subject site, Vs30, is 1,253.5 feet per second at SW-1 and 1,247.1 feet per second at SW-2.The

average Vs30 between both lines was approximately 1,250 feet per second. In accordance with ASCE 7-16 and ASCE 41-17, these shear-wave velocities correspond to a **Site Class C** (Very Dense Soil and Soft Rock).

We assume the seismic design of the proposed development will be in accordance with the California Building Code (CBC), 2019 edition, as well as ASCE 7-16 and ASCE 41-17. For the 2019 CBC/ASCE 7-16/ASCE 41-17, a Site Class C may be used. The seismic code values can be obtained directly from the tables in the building code using the above Site Class value and pertinent internet website (https://seismicmaps.org/). As mapped, the ASCE 7-16 seismic code values are as follows:

S_S = 1.989g	$S_{MS} = F_a * S_S$ = 2.387g	$S_{DS} = 2/3 * S_{MS}$ = 1.591g
S_1 = 0.712g	$S_{M1} = F_V * S_1$ = 0.997g	$S_{D1} = 2/3 * S_{M1}$ = 0.665g

As requested, site-specific seismic values were determined in accordance with ASCE 41-17, which provides methodologies for developing seismic response spectra for the evaluation and retrofit of existing buildings as well as construction of new buildings. In accordance with ASCE 41-17, we developed site-specific seismic response spectra for the following predetermined hazard levels:

- BSE-2N: Basic Safety Earthquake-2 for New Buildings. Equivalent to the Risk-Targeted Maximum Considered Earthquake (MCE_R) per Chapter 21 (§ 21.2) of ASCE 7-16 (1-percent probability of collapse within 50 years).

- BSE-1N: Basic Safety Earthquake-1 for New Buildings. Equivalent to the Risk-Targeted Design Response Spectrum per Chapter 21 (§ 21.3) ASCE 7-16 (two-thirds of BSE-2N).

- BSE-2E: Basic Safety Earthquake-2 for Existing Buildings. Taken as the seismic hazard with a 5-percent probability of exceedance in 50 years, but not greater than BSE-2N.

- BSE-1E: Basic Safety Earthquake-1 for Existing Buildings. Taken as the seismic hazard with a 20-percent probability of exceedance in 50 years, but not greater than BSE-1N.

In accordance with the requirements of the 2019 CBC, ASCE 7-16, and ASCE 41-17, the design spectral accelerations for the site should be determined based on a site-specific ground motion study. The details of our site-specific ground motion study are provided below, and the associated seismic parameters are presented on Table 1, ASCE 41-17 Site-Specific Seismic Response Spectra Worksheet – New Buildings, and Table 2, ASCE 41-17 Site-Specific Seismic Response Spectra Worksheet – Existing Buildings. The Project Structural Engineer should determine the seismic design method.

4.2.2 Site-Specific Ground Motion Analyses

The methodologies used to obtain site-specific ground motion analyses for new buildings and existing buildings are detailed in the following sections.

BSE-2N and BSE-1N Seismic Hazard Values (New Buildings)

Site-specific response spectra for the BSE-2N and BSE-1N hazard levels were generated in accordance with the 2019 California Building Code (CBC) (Section 1613A), Section 2.4 of ASCE 41-17, and Section 21.2 of ASCE 7-16, as well as ASCE 7-16 Supplements 1 to 3 (2018, 2021). Creation of a site-specific response spectrum requires analyzing site-specific deterministic and probabilistic seismic response spectra in order to create the Risk-Targeted Maximum Considered Earthquake (MCE_R) and Design Response Spectra.

We calculated the deterministic and probabilistic site-response spectra using web-based tools that estimate uniform hazard spectra using faults as earthquake sources. The web tools include geographic and seismic information of known active faults in California based on the 2014 USGS fault model. For both our deterministic and probabilistic analyses, we used four 2014 NGA West 2 attenuation relationships to determine the geometric-mean horizontal component of ground motion: Abrahamson-Silva-Kamai (2014), Boore-Stewart-Seyhan-Atkinson (2014), Campbell-Bozorgnia (2014), and Chiou-Youngs (2014).

For our evaluations, we used a shear wave velocity, V_{S30}, of 381 meters per second (m/s), or about 1,250 feet per second (fps), for the selected attenuation relationships. This value corresponds to a CBC Site Class C (Very Dense Soil and Soft Rock) and was determined directly from shear-wave velocity measurements obtained during our investigation at the project site (See Appendix D).

Probabilistic Spectra (2 percent probability of exceedance in 50 years)

The probabilistic (MCE_R) ground motion spectra (per the Method 1 requirements of Section 21.2.1.1, ASCE 7-16) were calculated using the USGS Unified Hazard Tool website. Using inputs of the site coordinates, Site Class, and time horizon (return period), the web tool outputs the Uniform Hazard Response Spectrum (UHRS) for predetermined Site Classes and shear wave velocities. For our analysis, we utilized the *Dynamic: Conterminous U.S. 2014 (update, v4.2.0)* edition of the web tool. We calculated response spectra for 360 m/s (C/D Boundary) and 537 m/s (Site Class C) and then linearly interpolated between those spectra based on our estimated site-specific shear wave velocity of 381 m/s.

The MCE_R corresponds to an earthquake ground motion having a 2 percent probability of exceedance within a 50-year period, or an average return period of 2,475 years. The final probabilistic response spectrum was based on the geometric mean horizontal component, scaled by factors to convert the geometric-mean response to the maximum-rotated response, of the spectral response values at 5% damping for the four above noted attenuation relationships. The maximum rotated component (MRC) response

factors used were based on the period dependent factors developed by Huang, Whittaker, and Luco (2008) and presented in Section 21.2 of ASCE 7-16. The weighted average, maximum-rotated site-specific probabilistic response based on the site-specific shear wave velocity is shown on Figure 4 (see attached).

Deterministic Spectra

Site-specific deterministic MCE response spectra were generated per the requirements of Section 21.2.2 of ASCE 7-16. The response spectrum was generated for nearby active faults, which were determined based on a combination of proximity, previous screening using the EZ-Frisk program, and the table of deaggregation contributors developed with the USGS Unified Hazard Tool. We analyzed the deterministic spectrum using the Puente Hills Fault (Los Angeles segment), the Elysian Park (Upper) Fault, and the Raymond Fault. Based on the above resources, the controlling deterministic response spectrum was found to be the Puente Hills Fault (Los Angeles segment).

Spectral acceleration ordinates were calculated utilizing the Pacific Earthquake Engineering Research Center (PEER) ground motion database and the PEER NGA-West2 Spectrum model. We utilized the four previously noted attenuation relationships (equally weighted) and determined the required input fault parameters from USGS web resources (see References). Per the requirements of ASCE 7-16, we utilized an epsilon value of 1.0 for our analysis, which corresponds to the 84th percentile of the geometric-mean component (S_a + one standard deviation) of the spectral acceleration at 5% damping. As with the probabilistic spectrum, the geometric-mean values were scaled by period-dependent factors per Huang et al (2008) to obtain the maximum-rotated response. The site-specific 84th percentile deterministic response spectrum is also shown in Figure 4.

BSE-2N (MCE_R) and BSE-1N (Design) Response Spectra

The above-described analytical steps are presented in the attached Table 1, ASCE 41-17 Site-Specific Seismic Response Spectra Worksheet – New Buildings.

The BSE-2N response spectrum was generated per the requirements of Section 21.2 of ASCE 7-16 by comparing the spectral response accelerations from the probabilistic MCE_R (Section 21.2.1) and the deterministic MCE_R (Section 21.2.2), with the resulting MCE_R response spectrum being the lesser of the spectral accelerations at each period. The ordinates for the BSE-2N response spectrum are presented in Table 1 (Column 11).

The BSE-1N response spectrum was generated by taking 2/3 of the BSE-2N response spectrum but confirming that the values are not less than 80 percent of the spectral acceleration determined per Sections 11.4.6 and 21.3 of ASCE 7-16. The ordinates for the BSE-1N response spectrum are presented in Table 1 (Column 12).

The BSE-2N and BSE-1N response spectra, as well as the general horizontal response spectra (per Section 2.4.1.7.1 of ASCE 41-17) for each respective hazard level, are shown in Figure 5 and tabulated in Table 1.

BSE-2E and BSE-1E Seismic Hazard Values (Existing Buildings)

The BSE-2E response spectrum corresponds to the probabilistic ground motions resulting in a 5 percent probability of exceedance within a 50-year period (average return period of 975 years) and was determined per the requirements of Section 2.4.1.3 of ASCE 41-17. The values for the BSE-2E spectrum need not exceed those for BSE-2N. The BSE-1E response spectrum corresponds to the probabilistic ground motions resulting in a 20 percent probability of exceedance within a 50-year period (average return period of 224 years) and was determined per the requirements of Section 2.4.1.4 of ASCE 41-17. The values for the BSE-1E spectrum need not exceed those for BSE-1N.

As with the MCE_R spectrum, the BSE-2E and BSE-1E spectrum were calculated using the USGS Unified Hazard Tool website [*Dynamic: Conterminous U.S. 2014 (update, v4.2.0)* edition]. We calculated response spectra for shear wave velocities of 360 m/s (C/D Boundary) and 537 m/s (Site Class C) and then linearly interpolated between those spectra based on our estimated site-specific shear wave velocity of 381 m/s.

The final BSE-2E and BSE-1E response spectra were based on the geometric mean horizontal component, scaled by period-dependent factors per Huang et al (2008) to convert the geometric-mean response to the maximum-rotated response. The ordinates for the BSE-2N response spectrum are presented in Table 2 (Column 11). The ordinates for the BSE-1E spectrum are presented in Table 2 (Column 12). For the BSE-2E and BSE-1E spectra, we confirmed that the values did not exceed the BSE-2N and BSE-1N spectra, respectively.

The BSE-2E and BSE-1E response spectra, as well as the general horizontal response spectra (per Section 2.4.1.7.1 of ASCE 41-17) for each respective hazard level, are shown on Figure 6 and tabulated in Table 2, ASCE 41-17 Site-Specific Seismic Response Spectra Worksheet – Existing Buildings. Figure 7 presents the site-specific response spectra for all four BSE hazard levels (BSE-2N, BSE-1N, BSE-2E, and BSE-1E).

4.2.3 Liquefaction, Lateral Spreading, and Seismic Settlement

Liquefaction is a phenomenon in which saturated cohesionless soils undergo a temporary loss of strength during severe ground shaking and acquire a degree of mobility sufficient to permit ground deformation. In extreme cases, the soil particles can become suspended in groundwater, resulting in the soil deposit becoming mobile and fluid-like. Liquefaction is generally considered to occur primarily in loose to medium dense deposits of saturated sandy soils. Thus, three conditions are required for liquefaction to occur: (1) a sandy soil of loose to medium density; (2) saturated conditions; and (3) rapid, large strain, cyclic loading, normally provided by earthquake motions.

The subject OPW building lies on the edge and outside of an area designated by the State of California as having a potential for soil liquefaction (CGS, 2017). The limits of the adjacent liquefaction zone appear to correspond to a historical drainage channel associated with Lincoln Park Lake located northeast of the site.

Groundwater was encountered at 55 feet in our northernmost boring and between depths of 25 and 35 feet in our southernmost borings. These depths correspond to approximate Elevations +270 to +280 feet. Historical groundwater levels at the site, as mapped by the State of California (CDMG, 1998), are estimated to be approximately 20 feet below the prevailing ground surface at the site. A groundwater depth of 20 feet was used in our evaluation, which is required even though the recently encountered groundwater depths are deeper. This groundwater depth, measured from the ground surface on the south side of the subject building, corresponds approximately to Elevation +285 feet.

In accordance with the 2019 CBC, ASEC 7-16, and Special Publication 117A (CGS, 2008), the ground motion used for this evaluation should be based on the Peak Ground Acceleration (PGA_M) adjusted for Site Class effects. This value is computed using the mapped Maximum Considered Geometric Mean (MCE_G) peak ground acceleration for a Site Class B and a site coefficient, F_{PGA}. As such, we considered a ground acceleration of 0.97g for a magnitude 7.0 earthquake for our analyses, which corresponds to the PGA_M obtained using the methods described above.

The potential for liquefaction was evaluated using the methods presented by the NCEER (Youd, 2001), with modifications provided in Special Publication 117A. The criterion for liquefaction susceptibility of the fine-grained soils was based on methods presented in Bray and Sancio (2006).

The materials encountered below the historical high groundwater depth of 20 feet consisted of relatively thin layers of alluvial clays overlying Puente Formation sandstone, claystone, and siltstone bedrock.

Based on our analyses, we computed negligible total and differential (across a span of 40 feet) seismic-induced liquefaction settlements (less than ¼ inch).

Seismic ground subsidence (not related to liquefaction induced settlements) occurs when strong earthquake shaking results in densification of loose to medium dense sandy soils above groundwater. The subsurface soils in the upper 20 feet, above the historical groundwater depth, consisted predominantly of firm to very stiff clays and silts and medium dense to very dense sands and clayey sands. As such, the estimated magnitude of total and differential (across a span of 40 feet) seismic ground subsidence is expected to be less than ¼-inch.

4.3 EARTHWORK

The earthwork at the project site for potential seismic retrofit and upgrades is anticipated to consist of demolition and clearing of portions of the existing concrete slab, excavations for enlarged portions of select shallow foundations, remedial grading for

new shallow foundations, excavations for new deep foundations, subgrade preparation, and the placement and compaction of fill.

4.3.1 Demolition and Clearing

Prior to construction of new shallow footings or enlarging existing shallow foundations, we anticipate portions of the existing slab and other improvements will be removed. Buried obstructions within the footprint of the proposed improvements, such as abandoned utilities, should also be removed. Deleterious material generated during the clearing operation should be removed from the site. At the conclusion of clearing operations, a representative of GPI should observe and accept the site prior to further grading.

If approved by the County, inert demolition debris, such as concrete, asphalt, and brick may be crushed for reuse in engineered fills outside the planned building areas in accordance with the criteria presented in the "Materials for Fill" section of this report. It is our experience that such material will be required to be exported from the site.

If cesspools or septic systems are encountered during grading, they should be removed in their entirety. The resulting excavation should be backfilled as recommended in the "Subgrade Preparation" and "Placement and Compaction of Fill" sections of this report. As an alternative, cesspools can be backfilled with lean sand-cement slurry. At the conclusion of the clearing operations, a representative of the personnel from GPI should observe and accept the site prior to further grading.

4.3.2 Excavations

Excavations at this site will include removals of soils disturbed during demolition, excavations for new and enlarged foundations, and may include trenching for new utility lines. The exposed subgrade at the bottom of these foundations should be prepared as detailed in the following section.

For pile supported structures, including minor structures such as light poles, remedial grading is not required unless new floor slabs are to be supported at grade.

Depth of Removals

For building retrofit where new foundations are required within the building pad, or where existing footings will be enlarged to carry additional loads, the extent of remedial grading will depend on the subsurface conditions encountered. For planning purposes, these foundations should be underlain by at least 1-foot of new properly compacted crushed aggregate base (CAB) materials. Deeper removals may be required depending on the actual conditions exposed in the foundation excavations.

Although not anticipated, at-grade improvements including minor structures or new pavements will require remedial grading. For planning purposes, removals below minor at-grade structures (e.g., free-standing walls, trash enclosures, etc.) should extend to depths of at least 2 feet below existing grade or 1 foot below the base of shallow

foundations, whichever is deeper. For new pavements, overexcavation is not required but the exposed subgrade soils below the new pavement section should be prepared as detailed in the following section.

Localized deeper removals may be required where deeper unsuitable fills are encountered or where loose subgrade soils at the bottom of removals are observed. The actual depths of removals should be determined in the field during grading by GPI. Existing grades refer to the grades at our exploration locations. The soils exposed at the base of the overexcavation should be processed in-place as described in the "Subgrade Preparation" section of this report. When structural loads are determined for the building retrofit, we should be provided with the loads to confirm the suitability of the planned overexcavation depths.

Lateral Extent of Removals

The Project Surveyor should accurately stake the corners of the areas to be overexcavated in the field. Where space is available, the base of the excavations for at-grade improvements should extend laterally a minimum distance equal to the depth of overexcavation/compaction below the bottom of new at-grade foundations (i.e., a 1:1 projection below the bottom outside edge of footings), whichever is greater. For the enlarged shallow foundations or new foundations within the existing building pad, lateral excavations beyond the edge of those foundations are not considered necessary.

Temporary Excavations

In general, the upper fill soils are not considered to be susceptible to caving in shallow excavations. Temporary construction excavations may be made vertically without shoring to a depth of 4 feet below the adjacent grade. Interior shoring, such as trench shoring, may be required for footing excavations that require vertical cuts deeper than 4 feet as access will be required to the base of the excavation for cleaning of loose soils as well as observation and probing. Deeper cuts, where the ground surface above the cut is level, should be properly shored or sloped back as follows:

- Cuts up to 8 feet should be sloped back to at least 1:1 (h:v) or flatter
- Cuts up to 16 feet should be sloped back to at least 1½:1 (h:v) or flatter

Where the ground surface above the cut is ascending, the sloping portion will need to be considered in the stability of the excavation and we can evaluate specific conditions when plans are further developed. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing.

In areas where temporary slopes are not feasible, temporary shoring or "ABC" slot cuts should be utilized. Unsurcharged slots adjacent to level ground should be no wider than 8 feet and no deeper than 8 feet and should be backfilled immediately. If desired, we can evaluate the potential for deeper slot cuts as necessary to achieve the required excavations. The slots should be backfilled immediately to finished grade prior to excavation of the adjacent slots. If the slots are performed adjacent to an existing building that has perimeter pad footings in addition to a continuous footing, the slots

should be aligned so that not more than one-half of the pad footing is exposed at a time. We should review the plans for excavation adjacent to existing buildings when they are developed.

Surcharge loads should not be permitted within a horizontal distance equal to the height of cut from the top of the excavation or 5 feet from the top of the slopes, whichever is greater, unless the cut is properly shored. Excavations that extend below an imaginary plane descending at an inclination of 1:1 from a point 1-foot above the base of an existing foundation should be properly shored to maintain support of adjacent elements. Excavations and shoring systems should meet the minimum requirements given in Article 6 of the State of California Occupational Safety and Health Standards.

In general, excavation can be accomplished by conventional soil excavation equipment such as backhoes, loaders, scrapers, or dozers. However, rubber tire earthwork equipment should not be used where the exposed subgrade soils have moisture contents well above optimum because of an increased potential for “pumping” and subgrade disturbance.

4.3.3 Subgrade Preparation

Prior to placing fills, the subgrade soils at the bottom of overexcavations should be scarified to a depth of 8 inches, moisture-conditioned, and compacted to at least 90 percent of the maximum dry density (95 percent in sandy soils) determined in accordance with ASTM D1557. This recommendation also pertains to the subgrade areas of asphalt pavement and hardscape. Scarification may be omitted, upon review by a GPI representative, if very moist to wet soils are exposed.

As noted previously, over-optimum soils were encountered within the approximate depths of the existing shallow foundations. Where wet soils are encountered or if the exposed soils become wet from seasonal rains, subgrade stabilization may be required to support compaction equipment. For planning purposes, the stabilization may require the placement of 12 inches of crushed aggregate base (CAB) over a geogrid, such as Tensar BX1200. A thicker section of CAB could be used if the geogrid is omitted. The final thickness of the CAB and use of geogrid will be subject to the actual subgrade conditions requiring stabilization.

4.3.4 Material for Fill

Soils and bedrock materials available from on-site excavations, less debris or organic matter, will be suitable for re-use in compacted fills, with the exception of the on-site clays/silts or reconstituted siltstone/claystone bedrock materials that are not suitable for placement as retaining wall backfill or within 12 inches of the finished subgrade for building floor slabs and hardscape. Soils placed behind retaining walls or within 12 inches of the finished subgrade should be predominately granular (containing no more than 40 percent fines - portion passing No. 200 sieve) and non-expansive (E.I. of 20 or less). Such materials do not appear to be readily on-site in sufficient quantities. Re-use of the bedrock materials as compacted fill will require extensive processing prior to placement.

In addition to the above, fills placed within the depth of planned foundations should not contain particles greater than 3 inches in diameter.

Imported fill material should be predominately granular and non-expansive as defined above. Import or on-site materials used in compacted fills should not contain particles larger than 6 inches in diameter. GPI should be provided with a sample (at least 50 pounds) and notified of the location of soils proposed for import at least 72 hours in advance of importing. Each proposed import source should be sampled, tested and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by GPI may be rejected if not suitable.

If open-graded gravel is planned as fill, it should be placed in 1- to 2-foot lifts and densified. Where placed adjacent to soil, the open-graded gravel should be separated from the soil with a suitable, non-woven geotextile, such as Mirafi 140N.

If the County approves of inert demolition debris, such as concrete, asphalt, and brick being crushed for reuse in engineered fills outside the planned building areas, the materials should meet the following processing requirements:

- If the inert debris is crushed to a well-graded mixture with maximum particle sizes of 1½ inches, the crushed material may be used directly in the fill without further blending.

- Inert debris up to a maximum size of 3 inches may also be used in fills, provided it is thoroughly blended with enough soils to form a well-grade mixture (typically a 4:1 soil to debris ratio).

4.3.5 Placement and Compaction of Fills

Fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to densities equal to at least 90 percent of the maximum dry density (95 percent for sandy soils), determined in accordance with ASTM D1557. The optimum lift thickness will depend on the compaction equipment used and can best be determined in the field. The following uncompacted lift thickness can be used as preliminary guidelines.

Plate compactors	4-6 inches
Small vibratory or static rollers (5-ton±) or track equipment	6-8 inches

The maximum lift thickness should not be greater than 8 inches and each lift should be thoroughly compacted and accepted prior to subsequent lifts.

Fills should be placed at moisture contents of 0 to 2 percent over the optimum moisture content for granular soils and silts, and 1 to 3 percent over optimum for clays. The moisture content of the soils encountered in the explorations was variable but, in general, over optimum. As such, moisture conditioning (drying) is anticipated prior to replacing the soils as properly compacted fill. The on-site soils should not be allowed to dry out prior to covering or additional processing will be required.

Where the existing ground surface is sloped at 5:1 (horizontal:vertical) or steeper, properly compacted fill should be placed in benches. Compacted fill slopes should be overfilled and trimmed back to achieve proper compaction at the slope face.

4.3.6 Shrinkage and Subsidence

Shrinkage is the loss of soil volume caused by compaction of fills to a higher density than before grading. Subsidence is the settlement of in-place subgrade soils caused by loads generated by large earthmoving equipment. For earthwork volume estimating purposes, an average shrinkage value of 10 to 15 percent may be assumed for the surficial soils. Variable shrinkage values may be realized in the areas of deep undocumented fills. Subsidence is anticipated to be negligible considering large earthmoving equipment will unlikely be used for this project. These values are estimates only and exclude losses due to removal of vegetation or debris. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be verified during grading.

4.3.7 Trench/Wall Backfill

Utility trench and wall backfill, consisting of the on-site materials or imported sand, should be mechanically compacted in lifts. The on-site clayey soils should not be used for retaining wall or wall-below-grade backfill. Lift thickness should not exceed those values given in the "Placement and Compaction of Fill" section of this report. Moisture conditioning of the on-site soils will be required prior to re-use as backfill. Jetting or flooding of backfill materials should not be permitted. A representative of GPI should observe and test trench and wall backfill as it is placed.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, sand-cement slurry may be substituted for compacted backfill. The slurry should contain 1½ sacks of cement per cubic yard. Within building areas, the slurry should contain 2 sacks of cement per cubic yard. When set, such a mix typically has the consistency of compacted soil. Under foundations, concrete equal in strength to the foundation concrete should be used if fill is required.

4.3.8 Observation and Testing

A representative of GPI should observe excavations, subgrade preparation, and fill placement activities. Sufficient in-place field density tests should be performed during fill placement and in-place compaction to evaluate the overall compaction of the soils. Soils that do not meet minimum compaction requirements should be reworked and tested prior to placement of additional fill.

4.4 FOUNDATIONS

4.4.1 Foundation Type

We understand that the proposed structural renovations may include enlarging existing shallow foundations, new shallow foundations, and/or new deep foundations consisting of conventional piles and/or tension piles. We understand that the potential renovations will not result in an increase in the gravity (dead plus live) loads on the building. However, the new foundations will be designed to accommodate updates in the seismic demand in accordance with the current seismic design criteria.

It is our opinion that it is acceptable to enlarge existing shallow foundations or construct new foundations in order to accommodate the increase seismic loads. New shallow foundations and enlarged portions of the existing shallow foundations may bear on the properly compacted subgrade soils following the recommendations presented in the "Earthwork" section previously presented. Remedial grading is not anticipated for new deep foundations.

4.4.2 Spread Footings

Bearing Capacities

Based on the shear strength and elastic settlement characteristics of the existing site soils, a static allowable net bearing pressure of up to 4,500 pounds per square foot (psf) may be used for expanded isolated and continuous shallow foundations and new isolated and continuous shallow foundations. The maximum allowable net bearing pressure provided above is governed principally by settlement considerations of the underlying soils.

The following minimum footing widths and embedments are recommended for the corresponding allowable bearing pressure for new or enlarged continuous and isolated column footings. Increases per foot and width are incorporated into the table below.

NEW/ENLARGED SHALLOW FOUNDATIONS

STATIC ALLOWABLE BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (feet)	MINIMUM FOOTING* EMBEDMENT (feet)
4,500	8.0	3.0
4,000	6.0	2.5
3,000	4.0	2.0
2,000	2.0	1.5
1,500	1.5	1.5

* Depth to bottom of footing below lowest adjacent finish grade.

The actual bearing pressure used may be less, such that economics and structural loads will determine the minimum width for footings as discussed below. These bearing pressures are for dead load-plus-live loads and may be increased by one-third (⅓) for short-term, transient, wind and seismic loading. The maximum edge pressures induced

by eccentric loading or overturning moments should not be allowed to exceed these recommended values. We utilized a factor of safety of 3 to determine the allowable bearing capacities of shallow foundations based on shear strength.

A minimum footing width and depth of 1.5 feet (18 inches) should be used for new and/or enlarged foundations within the existing building pad even if the actual bearing pressure is less than 1,500 psf.

Anticipated Settlement

As stated previously, we anticipate structural retrofits or renovations may only result in a nominal increase to the existing gravity loads on the structure. As such, new static total and differential settlements below new or enlarged isolated pad or continuous footings is expected to be negligible (less than ¼ inch). We anticipate total seismic settlements will be less than ½ inch, with differential seismic settlements across a span of 40 feet to be less than ¼ inch.

The above settlement estimates are based on the assumption that the recommended earthwork will be performed and that the footings will be sized in accordance with our recommendations.

Lateral Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of footings and underlying soils and by passive soil pressures acting against the embedded sides of the footings. For frictional resistance, a coefficient of friction of 0.35 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 300 pounds per cubic foot may be used, provided the footings are poured tight against the compacted fill. A one-third increase in the above allowable lateral bearing pressure (but not the frictional resistance) may be taken for short-term wind and seismic loads. The passive pressure provided also assumes a level ground surface extending to a horizontal distance from the wall or footing face at least twice the depth of embedment. These values may be used in combination without reduction.

Uplift Resistance

The soils above the footings may be used for resistance of uplift loads. We recommend using a unit weight of 120 pounds per cubic foot for the compacted fill soils in place above the footing. The volume of soil can be taken as the volume directly above the footing as well as the wedge of soil extending outward at a projection of 24 degrees with the vertical. The soils within the upper 1 foot of the ground surface should be omitted from the computed soil volume to allow for surficial disturbance.

Subgrade Modulus

For the elastic design of a mat foundation or combined footings acting as a mat foundation within the existing building limits, a modulus of subgrade reaction (k-value) of 200 pounds per cubic inch (pounds per square inch per inch of deflection) may be used. This value is for a 1-foot by 1-foot square loaded area with short-term loads and should be adjusted for the area of the mat foundation using appropriate elastic theory. Using generally accepted methods and our site-specific consolidation test results, we recommend using a value of 50 pci for the adjusted k-value in design.

Spread Footings Adjacent to Underground Utilities/Trenches

Spread foundations should be deepened below an imaginary plane ascending at an inclination of 1:1 from an underground utility within 10 feet of the footing to avoid surcharging the adjacent utility. New underground utilities/trenches should not be installed/excavated within a 1:1 plane descending from the edge of existing spread foundations unless the excavation and spread foundation are properly shored. Underground utilities that traverse below and/or through new spread footings should be placed in a sleeve that can accommodate settlement of foundation without imposing stress on the underground utility line and transfer the foundation stress to suitable bearing materials below the utility. Installation of new utilities below existing foundations is not recommended.

4.4.3 Deep Foundations

We understand the deep foundations are also being considered to support new seismic demands on the existing building. As such, we are providing the following recommendations for 24- to 36-inch diameter cast-in-drilled-hole (CIDH) pile foundations. For design, we assumed the bottom of pile cap (top of pile) elevation will be comparable to bottom of existing spread foundations. The design parameters should be updated, if necessary, based on the actual top of pile elevation.

Axial Pile Capacity

Drilled pile foundations will develop their frictional capacity with relatively small deflection (about ¼-inch). The allowable capacities will depend on the pile diameter and the total depth of the pile below the pile cap or ground surface. The downward vertical capacities shown below do not include capacity for end bearing, as it is difficult to adequately clean the base of small diameter drilled shafts. The values presented are for static loads and can be increased by one-third for short-term wind and seismic loads. The undocumented fill soils were neglected in developing the axial capacity. The allowable uplift capacity for a given pile may be taken as one-half of the compressive capacity provided below.

Based upon the results of our subsurface explorations and laboratory testing, the drilled pile design parameters shown in the following tables may be used for axial pile design:

DEPTH BELOW PILE CAP* (feet)	ALLOWABLE COMPRESSIVE CAPACITY (kips)*		
	24-inch Diameter	30-inch Diameter	36-inch Diameter
20**	17	20	26
22	23	28	34
24	28	35	42
26	34	44	53
28	41	54	65
30	47	65	78
32	54	76	91
34	61	86	103
36	69	95	114
38	77	105	126
40	85	115	138
45	107	143	171
50	130	172	207

* Assumes bottom of pile cap (top of pile) at 8 feet below existing basement level.
** Minimum recommended pile length.

We utilized a factor of safety of 3 to determine the allowable capacities of the CIDH pile. Since the drilled piles will be designed to derive resistance from friction only, rigorous cleaning of loose material from the bottom of the excavation prior to placement of steel and concrete is not considered essential. Effort should be made to clean the bottom with the drill rig-mounted equipment.

If axial loading of the structures, in compression or tension, is greater than provided above or other diameter CIDH piles are being considered, we can provide additional allowable capacities as requested.

Lateral Pile Capacity

Based on the requests of the County, we developed lateral pile capacities for both free-head and fixed-head conditions under allowable lateral deflections of ¼-inch, ½-inch, and ¾-inch. In developing the allowable lateral capacities, we utilized the design parameters presented in the following table, which can be used in lateral load analysis software such as LPILE or COM624.

Recommended Soil Parameters for Lateral Pile Analyses

Depth Below Top of Pile (feet)*	Soil Type	L-Pile Model	Estimated Effective Unit Weight (pcf)	Internal Angle of Friction (Degrees)	Static Soil Modulus, k (pci)	Undrained Shear Strength, c_u (psf)	Strain $È_{50}$
0 – 9	Undocumented Clay/Silt Fills	Soft Clay (Matlock)	120	---	---	500	0.015
9 – 12	Stiff to Very Stiff Clays	Soft Clay (Matlock)	123	---	---	500	0.015
12 - 27	Puente Formation Bedrock	Sand (Reese)	125	31	150	---	---
>27	Puente Formation Bedrock	Sand (Reese)	62.6	31	150	---	---

* Assumes bottom of pile cap (top of pile) at 8 feet below existing basement level.

The unit weight, cohesion, internal friction angle, strain, and lateral subgrade modulus are based upon our exploration and laboratory data as well as our review of nearby exploration and laboratory data by others (see References).

We developed allowable lateral pile capacities for 24-inch, 30-inch, and 36-inch CIDH piles extending to a depth of approximately 45 feet. For the purposes of these analyses, we assumed an axial pile load of about 100 kips.

The analyses were performed using LPILE (v. 2018). The lateral capacities for the requested allowable lateral deflections at the top of the pile (free and fixed head conditions) are presented below:

PILE SIZE/TYPE	LATERAL CAPACITY (KIPS)		
	¼-inch Deflection	½-inch Deflection	¾-inch Deflection
24” Diameter/Free Head	13.5	20.3	26.1
24” Diameter/Fixed Head	31.0	49.3	65.5
30” Diameter/Free Head	20.6	31.7	41.1
30” Diameter/Fixed Head	49.3	79.3	106.5
36” Diameter/Free Head	30.4	47.1	61.3
36” Diameter/Fixed Head	74.8	119.7	158.7

The shear, moment, and deflection curves for the 24-inch, 30-inch, and 36-inch CIDH piles are presented in Figures 8, 9, and 10, respectively. Additional details based on these lateral pile curves are provided below:

- Depth to maximum moment
    - Free Head Piles: Approximately 10 to 15 feet, increasing in depth with pile diameter and lateral deflection
    - Fixed Head Piles: Pile top (0 feet)

- Depth to negligible moment (approximately 5% of maximum moment)
    - 24-inch Free Head Piles: Approximately 22 to 24 feet
    - 24-inch Fixed Head Piles: Approximately 24 to 26 feet
    - 30-inch Free Head Piles: Approximately 24 to 27 feet
    - 30-inch Fixed Head Piles: Approximately 28 to 30 feet
    - 36-inch Free Head Piles: Approximately 27 to 30 feet
    - 36-inch Fixed Head Piles: Approximately 32 to 35 feet

We recommend new pile foundations extend to minimum depths corresponding to the above depths to negligible moment. If the surface surrounding the pile is not paved, we recommend the upper foot of soil be ignored in developing lateral resistance.

Pile Spacing

To avoid group effects on axial capacity, piles should be spaced at least 2½ diameters, center to center. To avoid group effect reduction of the lateral capacity of trailing piles, the piles should be spaced at least 8 diameters, center to center (in the direction of loading). If piles are spaced less than 8 diameters but greater than 2½ diameters, the lead pile can be designed for the full lateral resistance and the lateral resistance of trailing piles should be reduced by 50 percent.

Slope Setback

Standard setback requirements for foundation adjacent to descending slopes are provided in Section 1808.7 of 2019 California Building Code (CBC). Figures 1808.7.1 of the 2019 CBC provides a schematic detailing these typical setback requirements. Alternatively, the Code notes that these minimum setbacks are subject to modification with the approval of a building official (Section 1808.7.5).

From a geotechnical standpoint, GPI does not have a minimum setback from the top of slope for pile foundations and structural slab. Although not anticipated, we recommend that the allowable lateral capacities for piles located within 8 pile diameters from the top of slope be reduced by 50 percent. The Project Architect and Civil Engineer should also evaluate the required building offset from the top of the slope.

Pile Constructability

Based on the density and moisture content of the soils encountered, the caving potential of the soils is considered to be low. If caving soil conditions are encountered, casing, slurry, or other methods should be utilized to maintain the pile excavation. In addition, caving soils would increase the required volume of concrete. Based on our explorations, we do not anticipate encountering groundwater during the installation of the piles. However, some seasonal groundwater seepage is possible if pile excavations are left open for extended periods.

In general, the subsurface soils encountered below approximate depths of 20 feet on the south side of the building (approximate Elevation +285 feet) consisted of weathered bedrock materials. Due to the hard nature of the bedrock, large diameter piles extending below depths of 20 feet may encounter difficulty during drilling. The drilled pile contractor should evaluate the potential drilling conditions when planning the installation methods.

Pile excavations should be filled with concrete on the same day they are drilled. Concrete mix designs should include provisions to minimize shrinkage, which can lead to lower frictional resistance of the pile shaft and reduced allowable capacity. The concrete should be placed with special equipment so that it is not allowed to fall freely more than 5 feet or strike the walls of the excavations. Drilling for piles should not be performed within 5 feet of recently excavated or recently poured piles until the concrete has been allowed to set for at least 6 hours. The piles should be poured in a manner that will not result in concrete flowing into adjacent drilled pile excavations and prevent segregation of aggregate. Drilled pile construction should be performed in accordance with the latest edition of ACI 336.1, “Standard Specifications for the Construction of Drilled Piles.”

4.4.4 Foundation Concrete

Laboratory testing by HDR during this investigation indicates that the on-site soils have soluble sulfate contents ranging from 263 to 325 mg/kg (0.0263 and 0.0325 percent by weight). In accordance with the 2019 CBC, foundation concrete should conform to the requirements outlines in ACI 318, Section 19.3 for negligible levels of soluble sulfate exposure for soil (Category S0).

Testing by HDR during this investigation also indicated chloride contents of 4.5 to 8.4 mg/kg. These chloride values are also considered to be low (Category C1).

4.4.5 Foundation Construction Observation

Prior to placement of steel and concrete, a representative of GPI should observe and approve foundation excavations. Footing excavations should not be allowed to dry out and crack and should be moistened immediately prior to concrete placement.

CIDH pile excavations should be observed by a representative of GPI to confirm and document the depth, diameter, and embedment in suitable materials. Downhole inspection of drilled pile excavations (by lowering an inspector into the excavation) is not needed if the piles are designed as frictional elements.

4.5 RETAINING STRUCTURES AND SHORING

As noted previously, the existing OPW building includes a single-level, partial basement extending up to an approximate depth of 19 feet below the northern surface grades. The following recommendations are provided for cantilevered site walls or building walls below grade up to 20 feet in height. We recommend that walls be properly drained and backfilled with granular soils (less than 40 percent passing the No. 200 sieve). The

on-site clays are not suitable for use as retaining wall backfill where conventional backfill is used.

If desired, the low to medium expansive on-site soils (Expansion Index of 50 or less) may be used as retaining wall backfill with higher design active pressure. The backfill option selected (select fill required or on-site medium expansive soils allowed) should be specified clearly on the structural plans so that the Contractor will know if non-expansive, granular import soils are required. Highly expansive clays (Expansion Index greater than 50) should not be used as wall backfill.

4.5.1 Basement and Retaining Walls

Active earth pressures can be used for designing walls that can yield at least ½-inch laterally per 10 feet of wall height under the imposed loads. For level, drained backfill, derived from non-expansive granular soils (EI $\leq$ 20), a lateral pressure of an equivalent fluid weighing of 40 pounds per cubic foot (pcf) may be used. At-rest pressures should be used for restrained walls that remain rigid enough to be essentially non-yielding. For select, non-expansive, level, drained backfill, a lateral pressure of an equivalent fluid weighing 60 pounds per cubic foot (pcf) can be used for the at-rest pressure. If the wall backfill is not drained, the combined earth and water pressures could be much higher.

For the on-site soils with a medium expansion index (EI of 50 or less), a lateral pressure of an equivalent fluid weighing 80 pounds per cubic foot may be used for both active and at-rest conditions.

A seismic lateral pressure should be used for the design of retaining walls supporting 6 feet or more of backfill. We recommend a seismic lateral pressure of 49 pounds per cubic foot be added to the active earth pressure recommended above (equivalent fluid pressure). If at-rest pressure is used to design the retaining wall, the total lateral pressure used (at-rest plus seismic) should not exceed the total active plus seismic pressure (89 pounds per cubic foot).

If the wall backfill is not drained, the combined earth and water pressures could be much higher. For undrained backfill conditions or for the portion of the wall backfill below bottom of the wall drain system, we recommend the walls be designed to resist an additional hydrostatic pressure equivalent to a fluid with a density of 48 pounds per cubic foot. This pressure should be added to the lateral earth pressures provide above

Walls subject to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge pressure for unrestrained and restrained walls, respectively. In addition to the recommended earth pressure, the upper 10 feet of the walls adjacent to the streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge behind the shoring due to normal street traffic. If traffic is kept at least 10 feet from the walls, the traffic surcharge may be neglected.

The recommended pressures are based on the assumption that the supported earth will be fully drained, preventing the build-up of hydrostatic pressures. For traditional backfilled retaining walls, a drain consisting of perforated pipe and gravel wrapped in filter fabric should be used. One cubic foot of rock should be used for each lineal foot of pipe. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top. We prefer pipe and gravel drains to weep holes to avoid potential for constant flow of surface water in front of the wall. For retaining walls constructed adjacent to temporary shoring, a composite geotextile drain may be used with a manifold-type collection drain at the base. In addition, “rock-pockets” can be installed at the base of the shoring with a collection pipe extending from the “rock-pocket” to the collection system. A representative of GPI should observe and approve wall drains prior to placement of wall backfill.

The Structural Engineer should specify the use of select, granular wall backfill on the plans for walls that are to be conventionally backfilled. Retaining wall foundations should be designed in accordance with criteria presented in Section 4.4. Earthwork associated with site walls should be performed in accordance with the recommendations presented in Section 4.3.

4.5.2 Temporary Shoring

Where there is not sufficient space for sloped embankments, such as along the property limits or adjacent to existing structures, shoring will be required. Due to the limited space available within the existing structure, the available temporary shoring options are limited. Interior shoring, such as trench shoring/internal bracing, may be a suitable option for excavations that require vertical cuts deeper than 4 feet. The presence of groundwater should be considered when evaluating the alternatives.

For cantilever shoring with level backfill, the magnitude of active pressure is equivalent to the pressures imposed by a fluid weighing 35 pounds per cubic foot (pcf). For sloping backfill with a 1:1 inclination, the active pressure would be 53 pcf. The active pressure should be applied to the full embedment depth of the cantilever shoring. It should be noted that the provided lateral earth pressures assume a fully drained condition and do not include hydrostatic pressures.

In addition to the recommended earth pressure, the shoring should be designed for surcharge loads due to the adjacent structures and construction traffic surcharge loads. The upper 10 feet of the shoring adjacent to streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge behind the shoring due to normal street traffic. If traffic is kept at least 10 feet from the shoring, the traffic surcharge may be neglected. Existing adjacent structures will impart a surcharge load on shoring. The location and depth of the adjacent building footings, as well as the loading, will need to be determined to estimate the surcharge pressure on the shoring.

The shoring contractor should evaluate the potential drilling conditions when planning the installation methods.

It is difficult to accurately predict the amount of deflection of the shored embankment. It should be realized, however, that some deflection will occur. Adjacent to city right-of-way, the shoring should be designed to limited deflection to 1-inch. We recommend limiting the lateral deflection of shoring adjacent to structures to ½-inch. If greater deflection occurs during construction, additional bracing may be necessary. In areas where less deflection is desired, such as adjacent to existing settlement sensitive improvements, the shoring should be designed for higher lateral earth pressures.

4.6 CONCRETE SLABS

The subject building is located in a Methane Hazard Zone as designated by the City of Los Angeles. We recommend that an engineering firm specializing in methane evaluation and mitigation be consulted if methane mitigation recommendations are required.

Concrete slabs should be supported on at least 12 inches of non-expansive (EI less than 20), compacted soils as discussed in the “Material for Fill” and “Placement and Compaction of Fill” sections. Suitable soils do not appear to be readily available on-site in sufficient quantities. Based on our testing, the on-site clayey soils are not suitable for direct support of slabs and hardscape. The subgrade soils should not be allowed to dry out prior to concrete placement or additional processing and moisture conditioning will be required.

A moisture vapor retarder should be placed under slabs that are to be covered with moisture-sensitive floor coverings (parquet, vinyl, tile, etc.). Currently, common practice is to use a 10 or 15 mil polyolefin product such as Stego Wrap for this purpose. Whether to place the concrete slab directly on the vapor barrier or place a clean sand layer between the slab and vapor barrier is a decision for the Project Architect, as it is not a geotechnical issue. If covered by sand, the sand layer should be about 2 inches thick and contain less than 5 percent by weight passing the No. 200 sieve. Based on our explorations and laboratory testing, the soils at the site are not suitable for this purpose. The function of the sand layer is to protect the vapor retarder during construction and to aid in the uniform curing of the concrete. This layer should be nominally compacted using light equipment. The sand placed over the vapor retarder should only be slightly moist. If the sand gets wet (for example as a result of rainfall or excessive moistening) it must be allowed to dry prior to placing concrete. Care should be taken to avoid infiltration of water into the sand layer after placement of the concrete slab, such as at slab cut-outs and other exposures. A sand layer is not required beneath the vapor retarder, but we take no exception if one is provided.

It should be noted that the material used as a vapor retarder is only one of several factors affecting the prevention of moisture accumulation under floor coverings. Other factors include maintaining a low water-cement ratio for the concrete used for the floor slab and effective sealing of joints and edges (particularly at pipe penetrations). The manufacturer of the floor coverings should be consulted for establishing acceptable criteria for the condition of the floor surface prior to placing moisture-sensitive floor coverings.

For the elastic design of slabs (within the existing building limits) supporting sustained concentrated loads, a modulus of subgrade reaction (k) of 200 pounds per cubic inch (pounds per square inch per inch of deflection) may be used. This value is for a 1-foot by 1-foot square loaded area and should be adjusted by the structural designer for the area of the proposed building slab using appropriate elastic theory. For concrete slabs located outside of the existing building limits, a modulus of subgrade reaction (k) of 100 pounds per cubic inch may be used.

4.7 EXTERIOR CONCRETE AND MASONRY FLATWORK

Exterior concrete and masonry flatwork, hardscape, and sidewalks should be supported on a zone of non-expansive compacted fill. The use of clayey soils within 12 inches of concrete hardscape and sidewalk subgrade is not recommended. Prior to placement of concrete, the subgrade should be prepared as recommended in “Subgrade Preparation” section. The moisture content of subgrade soils should be maintained above the optimum moisture content and confirmed by a representative of GPI prior to covering. Subgrade soils allowed to dry out will require moisture conditioning, including the potential for additional processing.

We recommend the Project Civil Engineer design the concrete hardscape and sidewalks, including determination of thickness and reinforcing. For exterior flatwork, hardscape, and sidewalks, we recommend a minimum slab thickness of 4 inches with minimum slab reinforcement of No. 3 rebar placed at 16 inches on-center, in both directions. Control joints to direct shrinkage cracking in exterior slabs and sidewalks should be provided at maximum spacing of 8 and 6 feet on center in two directions, respectively. Where flatwork is planned at building entries, we recommend it be structurally connected to the perimeter building foundation to reduce the potential differential settlement at this joint. These recommendations should be considered as minimums based on the geotechnical site conditions, and the Project Civil Engineer should confirm if more stringent recommendations are needed for other purposes.

Concrete hardscape, stairs, flatwork, and sidewalks should be underlain by at least 4 inches of crushed aggregate base (CAB). Base course materials should be compacted to at least 95 percent of the maximum dry density (ASTM D1557). Aggregate base should conform to the requirements of Section 200-2 of the Standard Specifications for Public Works Construction (Green Book) for CAB materials.

4.8 PAVED AREAS

For new pavements, an assumed R-value of 15 was used for the on-site soils. The California Division of Highways Design Method was used for design of the recommended preliminary pavement sections. These recommendations are based on the assumption that the pavement subgrades will consist of existing near surface soils. The following pavement sections are recommended:

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		ASPHALT/PORTLAND CONCRETE	AGGREGATE BASE COURSE
Asphalt Concrete			
Pedestrian (no vehicles)	---	2.0	3.0
Automobile Parking	4.0	3.0	5.0
Automobile Drives	5.0	3.0	8.0
Truck/Bus Drives	6.0	3.5	11.0
Heavy Truck/Bus Drives	7.0	4.0	13.0
Portland Cement Concrete			
Pedestrian (no vehicles)	---	4.0	4.0
Automobile Parking	4.0	6.0	4.0
Automobile Drives	5.0	6.5	4.0
Truck/Bus Drives	6.0	7.0	4.0
Heavy Truck/Bus Drives	7.0	7.5	4.0

For pavement design of fire lanes, assuming an anticipated usage of one round trip per month by a 68,000-pound fire truck, a pavement section corresponding to a traffic index (TI) of 5 is considered to be suitable. If the fire lane will also accommodate additional traffic consisting of trucks and buses, the pavement sections should correspond to either a TI of 6 (truck/bus drives) or 7 (heavy truck/bus drives).

The concrete used for paving should have a modulus of rupture of at least 550 psi (equivalent to an approximate compressive strength of 3,700 psi at the time the pavement is subjected to traffic). If the site is base paved prior to the start of building construction, the above pavement sections should be re-evaluated based on the anticipated construction traffic loads.

The pavement subgrade underlying the aggregate base should be properly prepared and compacted in accordance with the recommendations outlined under "Subgrade Preparation".

The aggregate base should be compacted to at least 95 percent of the maximum dry density (ASTM D1557). Aggregate base should conform to the requirements of Section 200-2 of the Standard Specifications for Public Works Construction (Green Book) for CAB materials.

The above recommendations are based on the assumption that the aggregate base and compacted subgrade will be properly drained. The design of paved areas should incorporate measures to prevent moisture build-up within the base course, which can otherwise lead to premature pavement failure. For example, curbing adjacent to landscaped areas should be deep enough to act as a barrier to infiltration of irrigation water into the adjacent base course.

4.9 CORROSION

Laboratory testing of representative samples for chemical content (sulfate, pH, resistivity) was performed by HDR under subcontract to GPI during our current investigation. Based on the current testing, the on-site soils were found to be corrosive to buried ferrous metals.

The results of the current testing are presented in Table 1 of Appendix B. We recommend that a corrosion-engineering firm, such as HDR, be consulted if corrosion protection recommendations are required.

4.10 SURFACE DRAINAGE AND INFILTRATION

Positive surface gradients should be provided adjacent to structures so as to direct surface water run-off and roof drainage away from foundations and slabs toward suitable discharge facilities. Long-term ponding of surface water should not be allowed on pavements, turf, or adjacent to buildings.

Field infiltration testing was not included in our scope. The potential for water to infiltrate into a soil is based on the gradation and in-place density of a soil. Based on our explorations, the subsurface consisted predominantly of clayey and silty fill soils overlying natural clayey and silty soils and bedrock materials. These materials are likely not suitable for infiltration as the infiltration rates will be negligible. If desired, GPI can perform supplemental field testing to determine the infiltration characteristics of the subsurface soils.

4.11 GEOTECHNICAL OBSERVATION AND TESTING

We recommend that a representative of GPI observe earthwork and shoring installation during construction to confirm that the recommendations provided in our report are applicable during construction. The earthwork activities include grading, compaction of fills, subgrade preparation, pavement construction and foundation excavations. If conditions are different than expected, we should be afforded the opportunity to provide an alternate recommendation based on the actual conditions encountered.

5.0 LIMITATIONS

This report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for use by the County of Los Angeles and their consultants in designing the proposed development. The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on project other than the currently proposed development as it may not contain sufficient or appropriate information for such uses.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations. While we cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data obtained in the field and laboratory are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

Furthermore, our recommendations were developed with the assumption that a proper level of field observation and construction review will be provided by GPI during grading, excavation, and foundation construction. If construction phase services are performed by others, they must accept full responsibility for geotechnical aspects of the project, including this report.

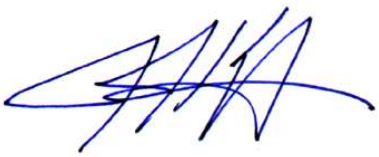
Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable geotechnical engineers practicing in this area. No other representation, either expressed or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals, Inc.





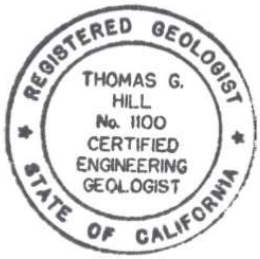
Dylan J. Boyle, G.E.
Senior Engineer





Justin J. Kempton, G.E.
Principal





Thomas Hill, C.E.G.
Consulting Geologist

REFERENCES

AECOM, "Report of Geotechnical Investigation and Bulk Factor, Women's and Children's Hospital Demolition, LAC+USC Medical Center, 1240 North Mission Road, Los Angeles, California", AECOM Project No. 60597019, dated October 21, 2019

American Society of Civil Engineers (ASCE) (2017), "Minimum Design Loads and Associated Criteria for Buildings and Other Structures," ASCE/SEI 7-16

American Society of Civil Engineers (2018), "Minimum Design Loads and Associated Criteria for Buildings and Other Structures, Supplement 1," ASCE/SEI 7-16, effective December 12, 2018

American Society of Civil Engineers (2021), "Minimum Design Loads and Associated Criteria for Buildings and Other Structures, Supplement 3," ASCE/SEI 7-16, effective November 5, 2021

American Society of Civil Engineers (2021), "Minimum Design Loads and Associated Criteria for Buildings and Other Structures, Supplement 2," ASCE/SEI 7-16, effective October 14, 2021

American Society of Civil Engineers (2017), "Seismic Evaluation and Retrofit of Existing Buildings," ASCE/SEI 41-17.

California Department of Conservation, Division of Mines and Geology (1997), "Special Publications 117: Guidelines for Evaluating and Mitigating Seismic Hazards in California."

California Geological Survey (2017), Earthquake Zones of Required Investigation, Los Angeles Quadrangle, released June 15, 2017

California Department of Conservation, Division of Mines and Geology (1998), Seismic Hazard Zone Report for the Los Angeles 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 029

California Geological Survey (2008), "Guidelines for Evaluating and Mitigating Seismic Hazards in California," Special Publication 117A

California Office of Statewide Health Planning and Development (OSHPD), Seismic Design Maps Website, https://seismicmaps.org/

City of Los Angeles, 1996, Safety Element of the Los Angeles City General Plan, November 26, 1996.

County of Los Angeles, 1990, Safety Element of the Los Angeles County General Plan, December 6, 1990.

Geotechnologies, Inc., "Geotechnical Engineering Investigation, Proposed Exterior Elevator, LAC+USC Medical Center, 2051 Marengo Street, Los Angeles, California", Geotechnologies File No. 21832, dated August 12, 2019

Huang, Y. N., Whittaker, A. S., and Luco, N. (2008). "Maximum spectral demands in the near-fault region." *Earthquake Spectra,* 24(1), 319-341.

Los Angeles Department of Building and Safety (LADBS), *Memorandum to Geology and Soils Engineering Firms Practicing in the City of Los Angeles*, dated July 16, 2014.

NETR Online, Historical Aerials, www.historicalaerials.com

Pacific Earthquake Engineering Research Center (PEER) Ground Motion Database, NGA-West2 Shallow Crustal Earthquakes in Active Tectonic Regimes, Target Spectrum (used for deterministic site-specific seismic analysis), accessed October 2022, https://ngawest2.berkeley.edu/spectras/new?sourceDb_flag=1

RMA Group, "Geotechnical Investigation for Proposed L.A. Co. – USC Medical Center Child Care Project, N. State Street between Zonal Avenue and Marengo Street, Los Angeles, CA," RMA Project No. 18-0056-01, dated March 30, 2018

Robertson, P.K., "*Performance based earthquake design using the CPT,"* Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009.

United States Geological Survey (2014), 2008 National Seismic Hazard Maps, Source Parameters, https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm

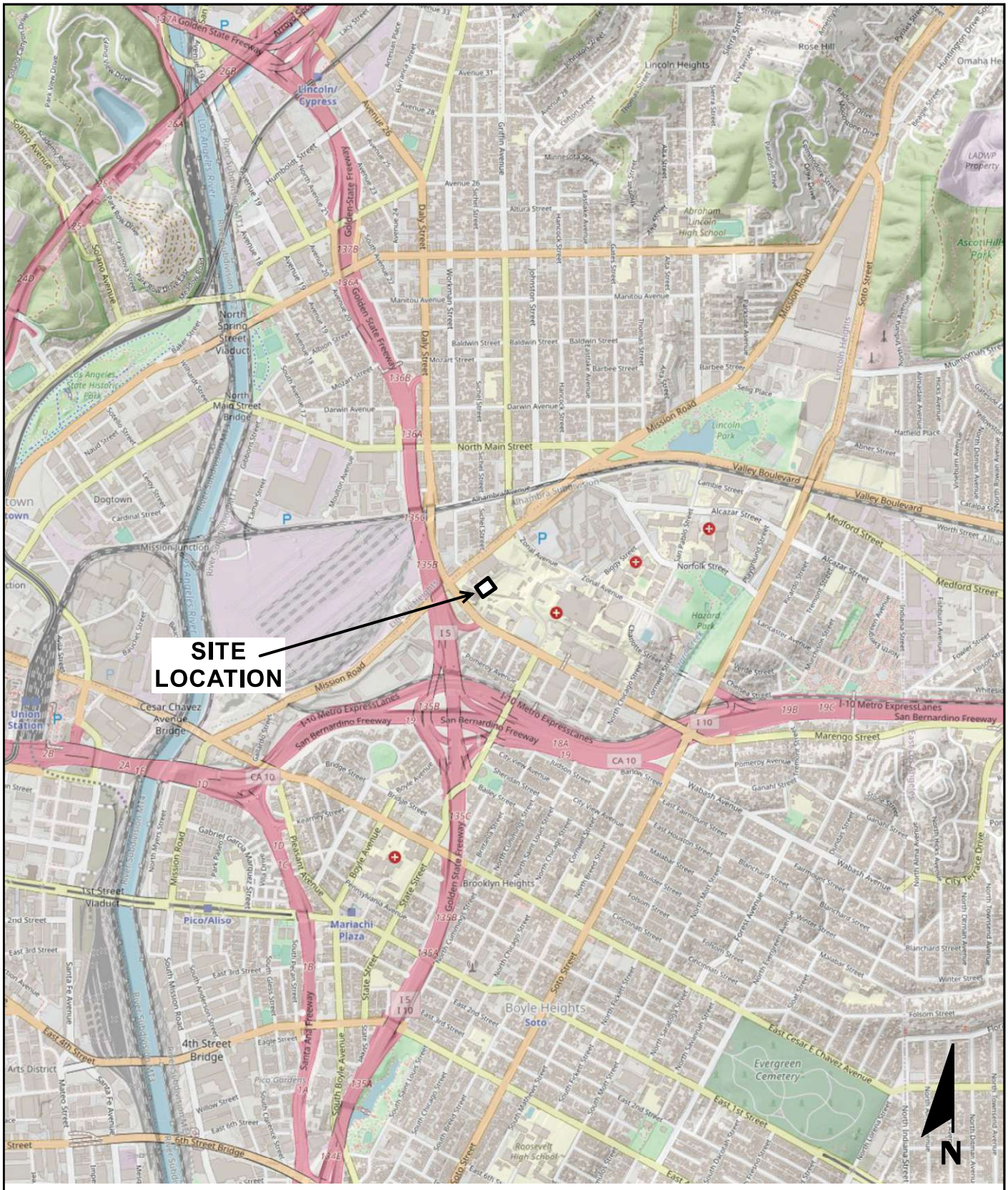
United States Geologic Survey (USGS), M 6.7 Scenario Earthquake – Elysian Park (Upper), accessed April 2020, https://earthquake.usgs.gov/scenarios/eventpage/bssc2014elysianparkupperellb_m6p65_se/executive

United States Geologic Survey (USGS), M 7.0 Scenario Earthquake – Puente Hills (LA), accessed April 2020, https://earthquake.usgs.gov/scenarios/eventpage/bssc2014puentehillslaellbgeo_m6p95_se/executive

United States Geologic Survey (USGS), M 6.7 Scenario Earthquake – Raymond, accessed October 2022, https://earthquake.usgs.gov/scenarios/eventpage/bssc2014raymondellbgeol_m6p71_se/executive

United States Geologic Survey (USGS), Unified Hazard Tool website, accessed October 2022, https://earthquake.usgs.gov/hazards/interactive/

United States Geologic Survey (USGS), USGS Earthquake Scenario Map website (Building Seismic Safety Council, BSSC 2014), accessed October 2022, https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=14d2f75c7c4f4619936dac0d14e1e468



BASE MAP REPRODUCED FROM OPENSTREETMAP © CALTOPO

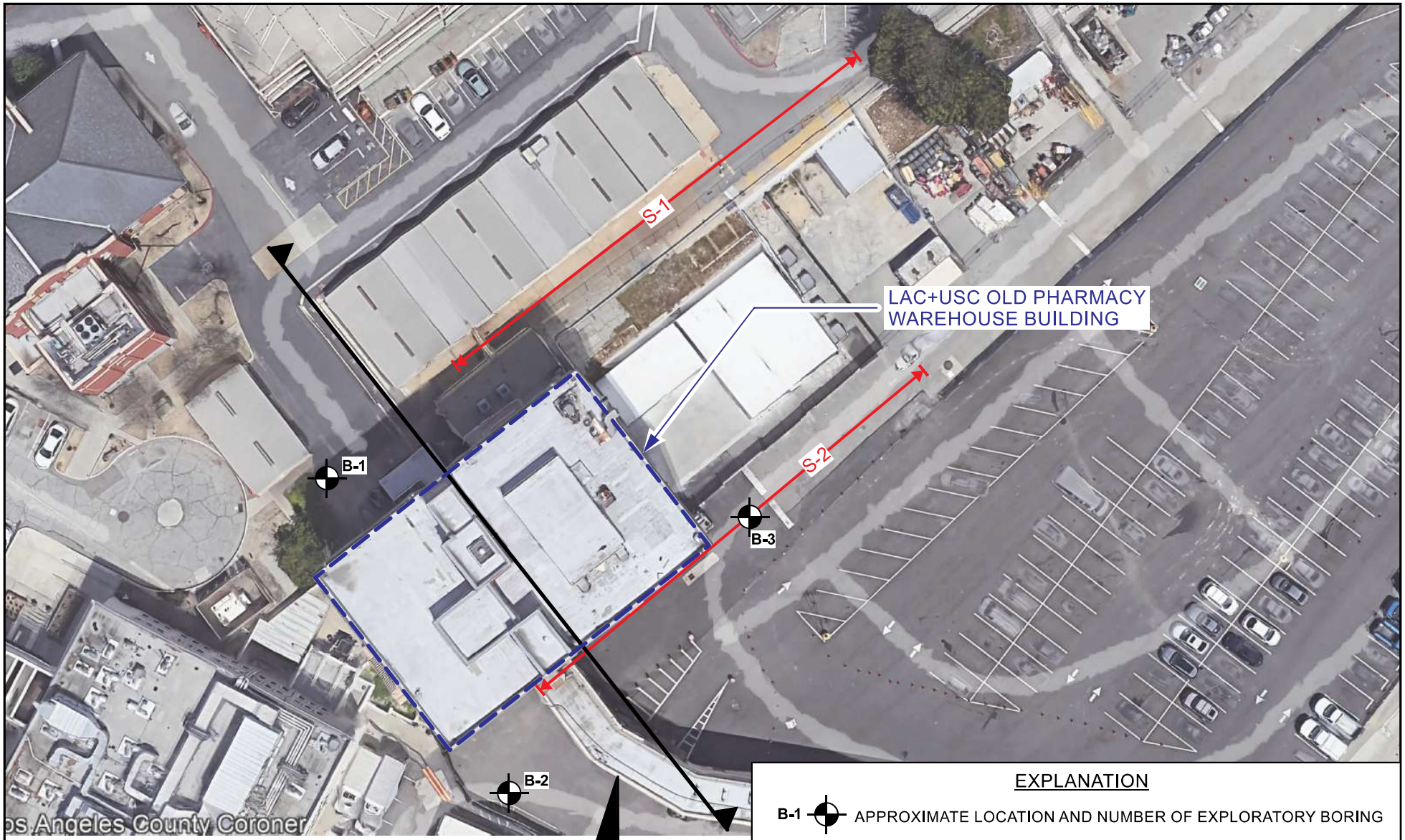
GPI GEOTECHNICAL PROFESSIONALS, INC.

LAC-USC OLD PHARMACY BUILDING

GPI PROJECT NO.: 3080.22I | SCALE: 1" = 2000'

SITE LOCATION MAP

FIGURE 1



BASE MAP REPRODUCED FROM GOOGLE EARTH © 2022

EXPLANATION

B-1 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

S-1 APPROXIMATE LOCATION AND NUMBER OF SEISMIC SURVEY LINE

APPROXIMATE LOCATION OF SUBSURFACE CROSS SECTION

GPI GEOTECHNICAL PROFESSIONALS, INC.

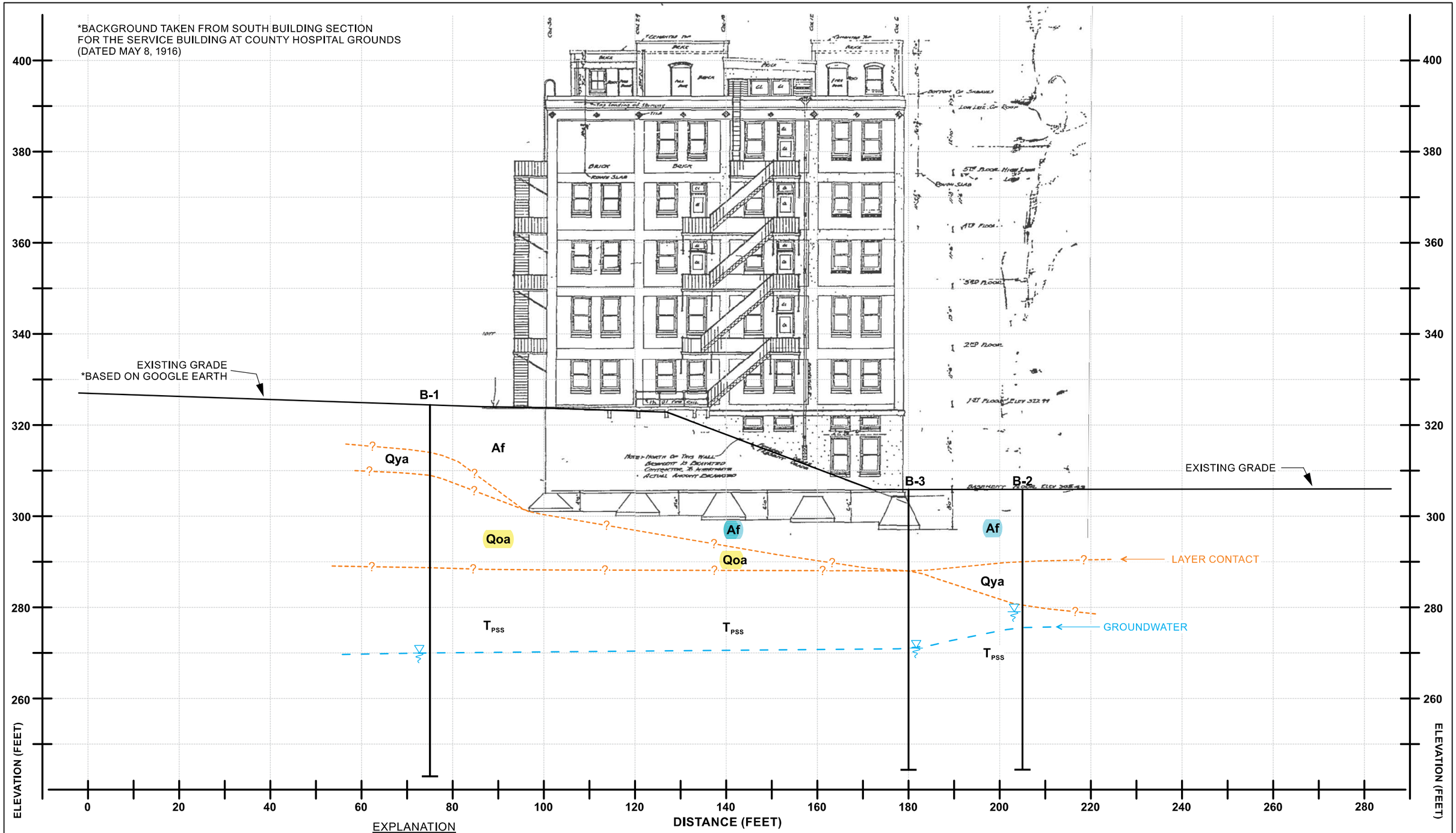
LAC+USC OLD PHARMACY WAREHOUSE

GPI PROJECT NO.: 3080.22I | SCALE: 1" = 50'

SITE PLAN

FIGURE 2

*BACKGROUND TAKEN FROM SOUTH BUILDING SECTION
FOR THE SERVICE BUILDING AT COUNTY HOSPITAL GROUNDS
(DATED MAY 8, 1916)



EXPLANATION

Af	ARTIFICIAL FILL SOILS
Qya	YOUNGER ALLUVIUM
Qoa	OLDER ALLUVIUM
T_{PSS}	BEDROCK: PUENTE FORMATION (SANDSTONES, CLAYSTONES, SILTSTONES)

Note: This section is based upon information obtained at borings obtained during our geotechnical investigation. This section is based upon limited geotechnical data and localized variations should be anticipated. This section is intended for descriptive purposes only.

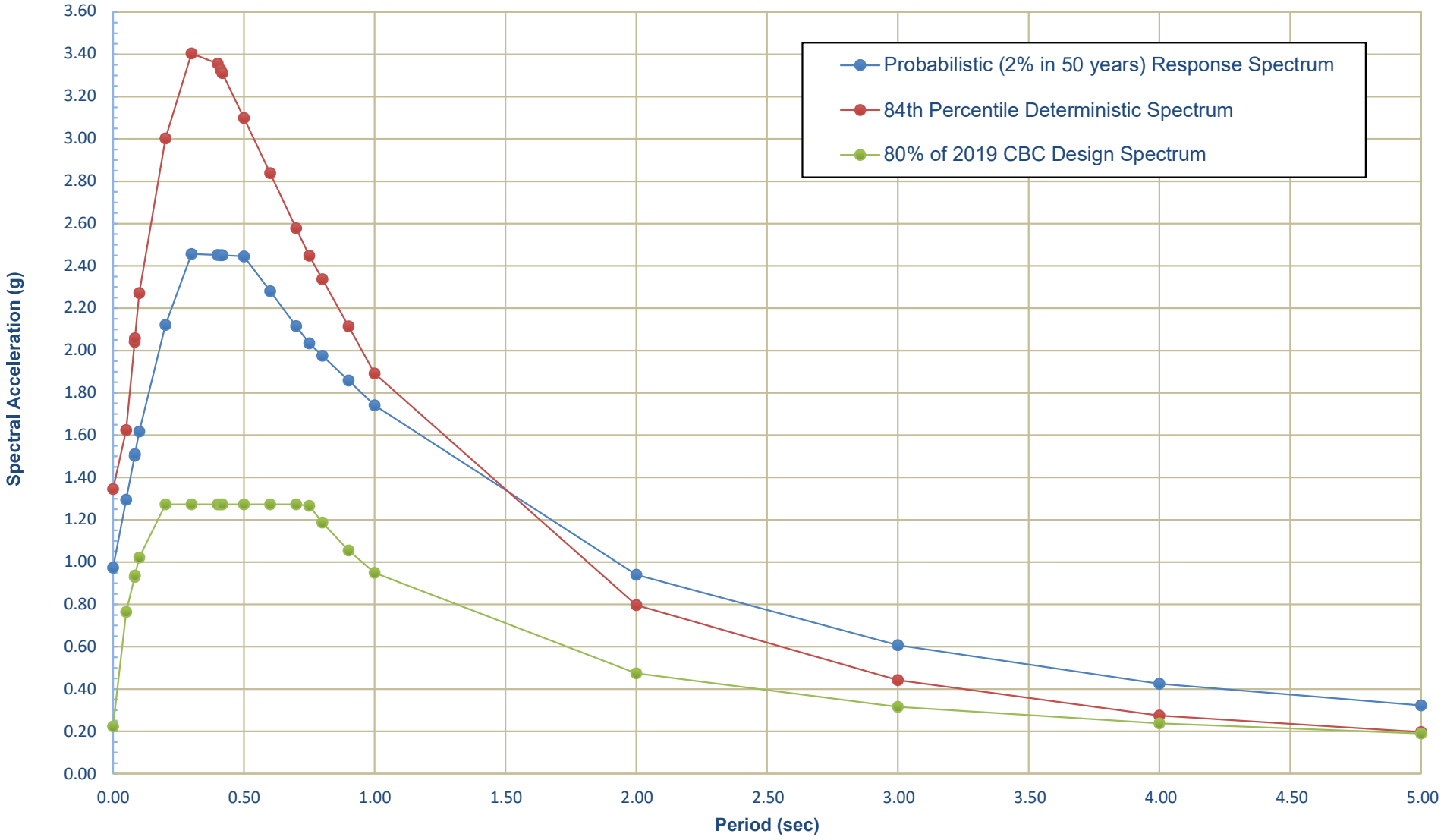
GPI GEOTECHNICAL PROFESSIONALS, INC.

LAC+USC OLD PHARMACY WAREHOUSE

GPI PROJECT NO.: 3080.022I | SCALE: 1" = 20'

SUBSURFACE CROSS SECTION

FIGURE 3

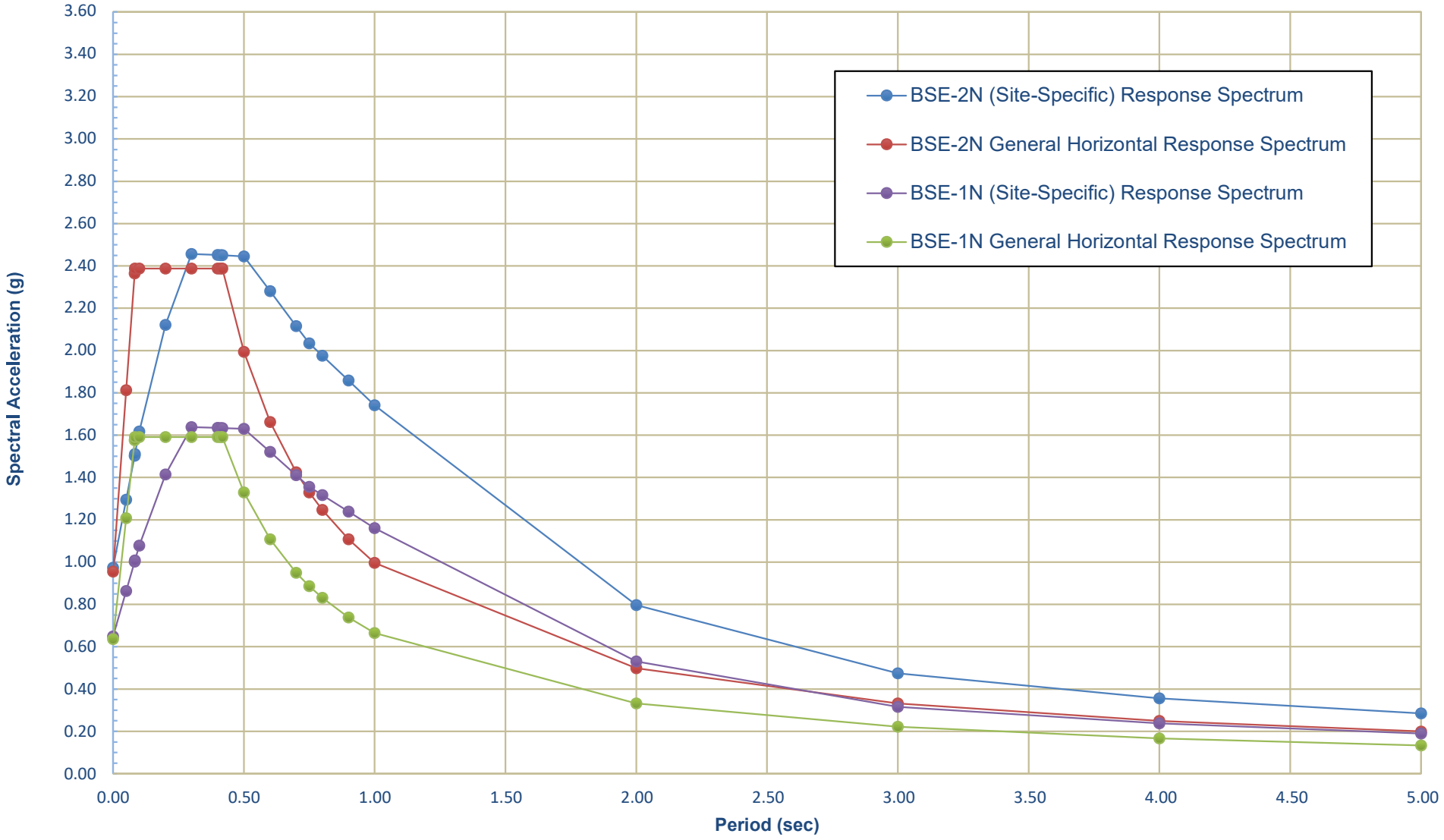


Site-Specific Probabilisitic & Deterministic (MCE_R) Response Spectra @ 5% Damping
per Chapter 21 (ASCE 7-16), Chapter 2 (ASCE 41-17), and 2019 CBC
LAC+USC Old Pharmacy Building
Los Angeles, California

GEOTECHNICAL PROFESSIONALS, INC.

Project No. 3080.22I

Figure 4

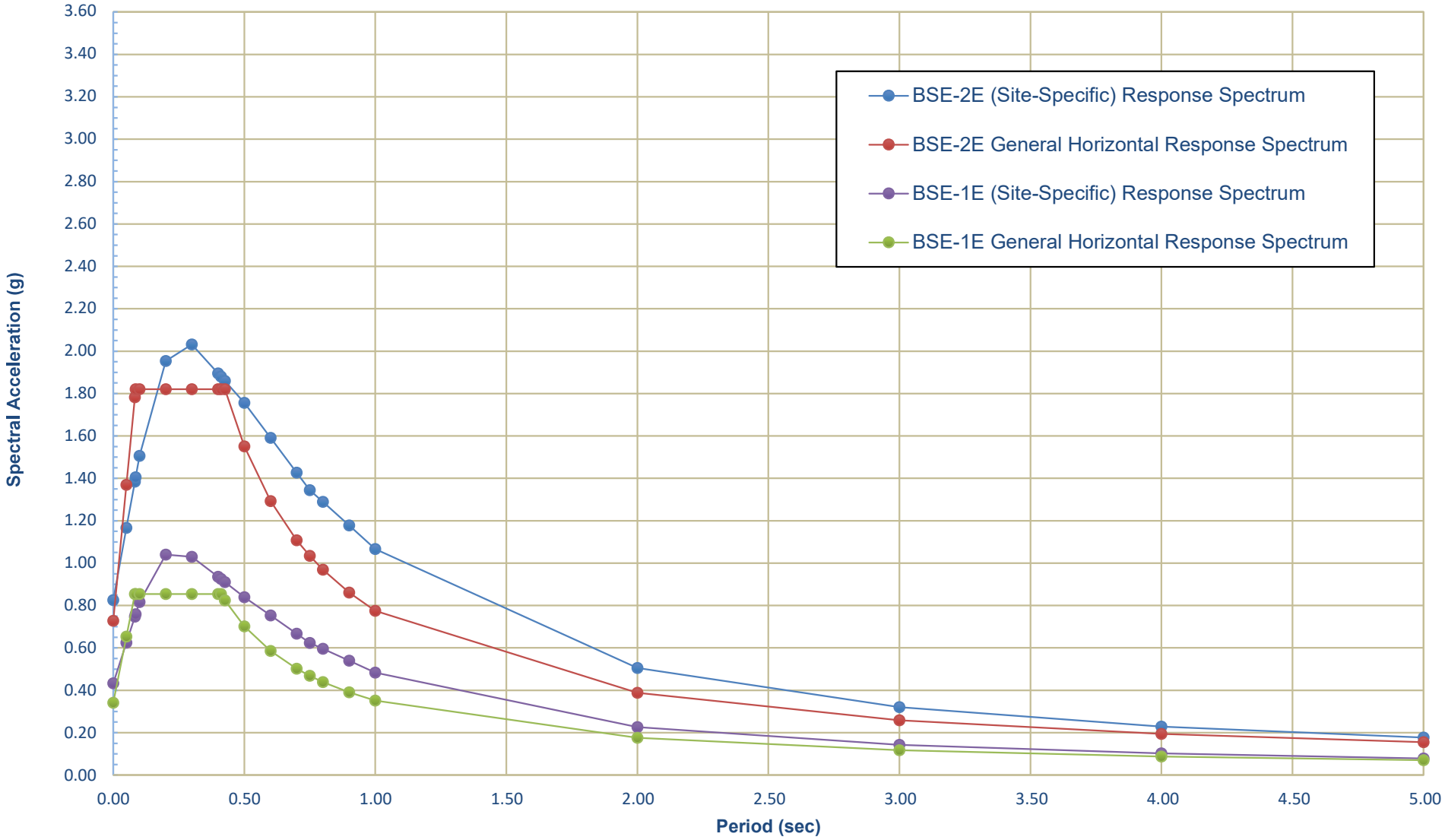


Site-Specific BSE-2N and BSE-1N Response Spectra
per Chapter 21 (ASCE 7-16), Chapter 2 (ASCE 41-17), and 2019 CBC
LAC+USC Old Pharmacy Building
Los Angeles, California

GEOTECHNICAL PROFESSIONALS, INC.

Project No. 3080.22I

Figure 5

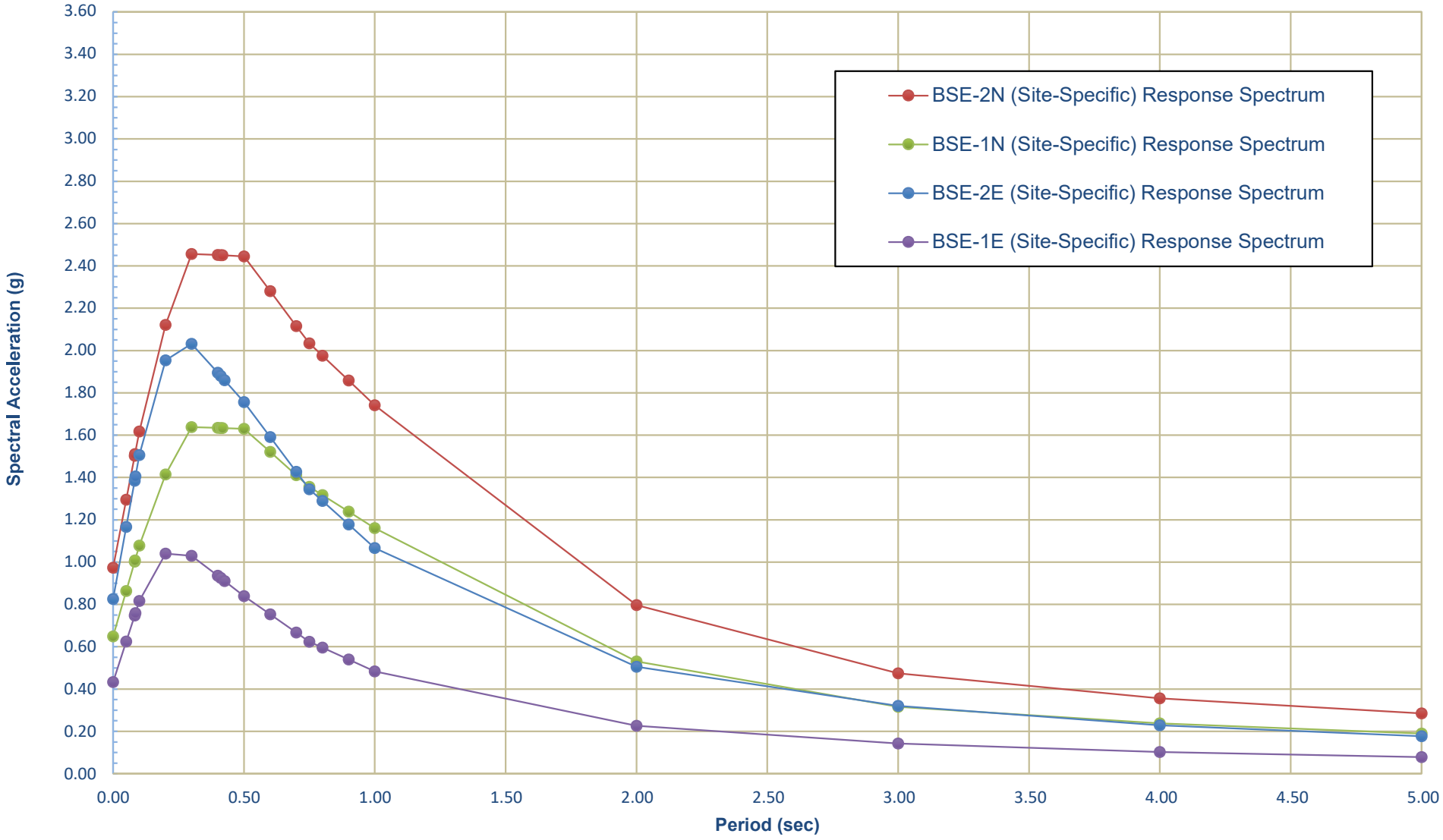


Site-Specific BSE-2E and BSE-1E Response Spectra
per Chapter 21 (ASCE 7-16), Chapter 2 (ASCE 41-17), and 2019 CBC
LAC+USC Old Pharmacy Building
Los Angeles, California

GEOTECHNICAL PROFESSIONALS, INC.

Project No. 3080.22I

Figure 6

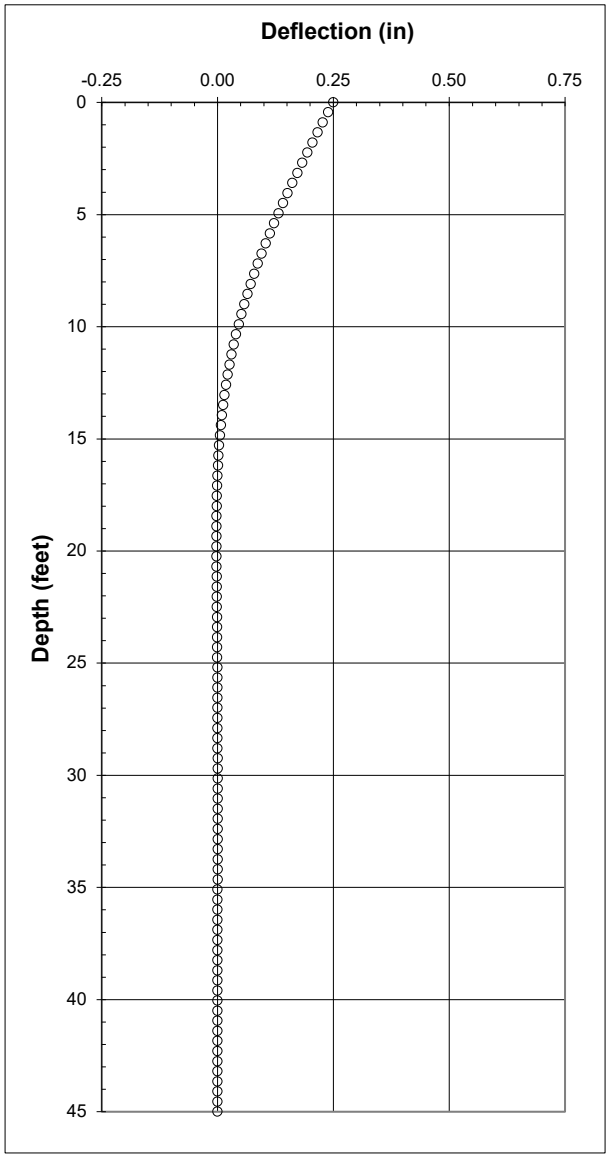


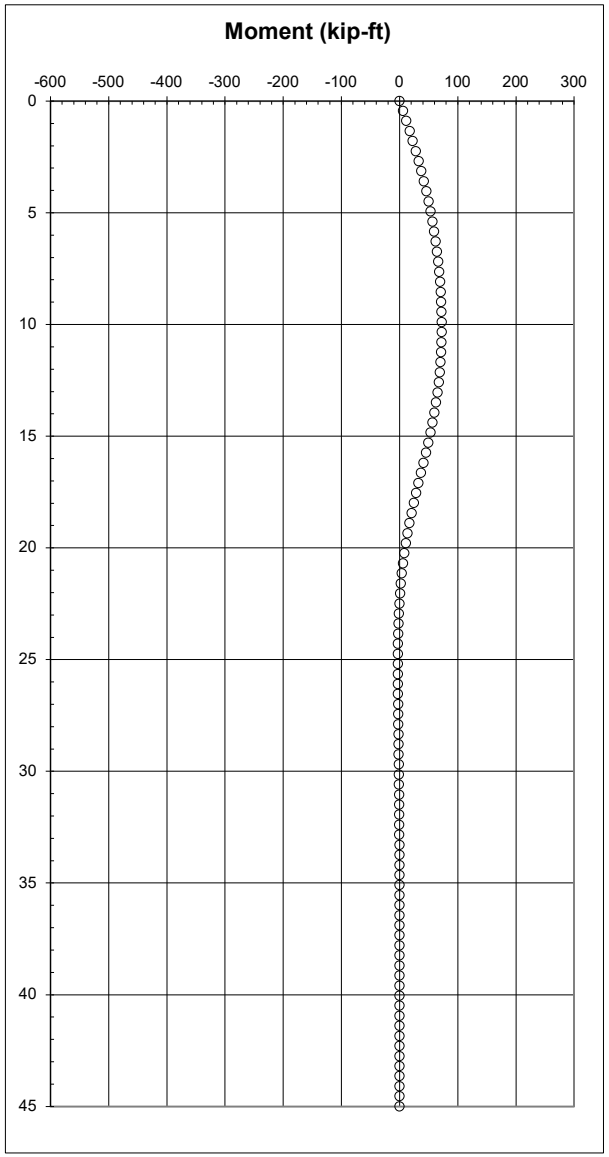
Site-Specific BSE-2N, BSE-1N, BSE-2E, and BSE-1E Response Spectra
per Chapter 21 (ASCE 7-16), Chapter 2 (ASCE 41-17), and 2019 CBC
LAC+USC Old Pharmacy Building
Los Angeles, California

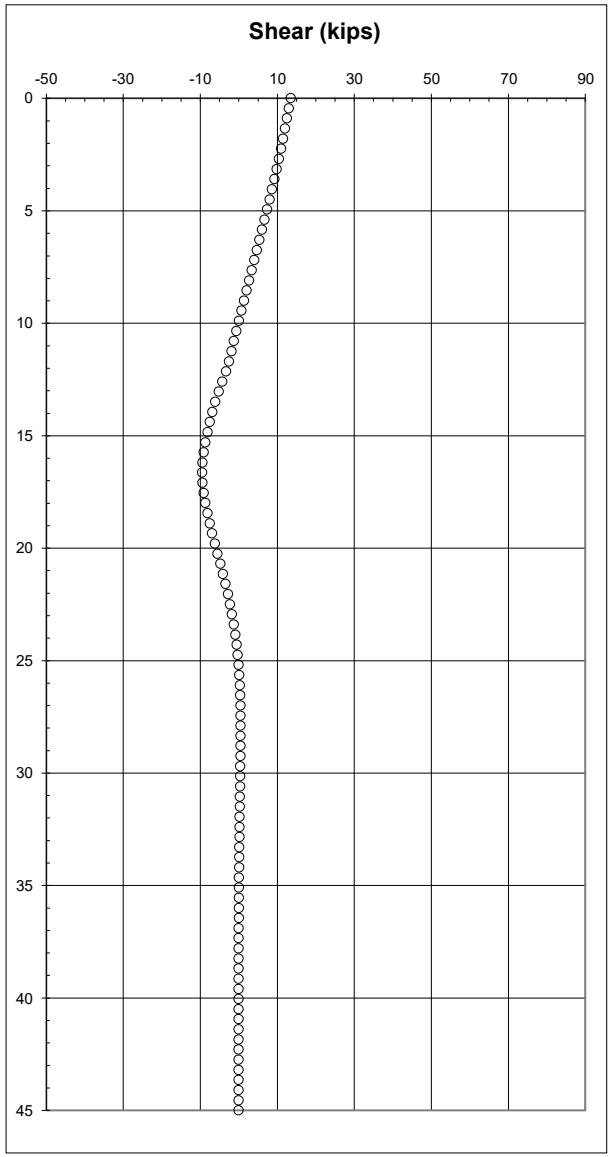
GEOTECHNICAL PROFESSIONALS, INC.

Project No. 3080.22I

Figure 7



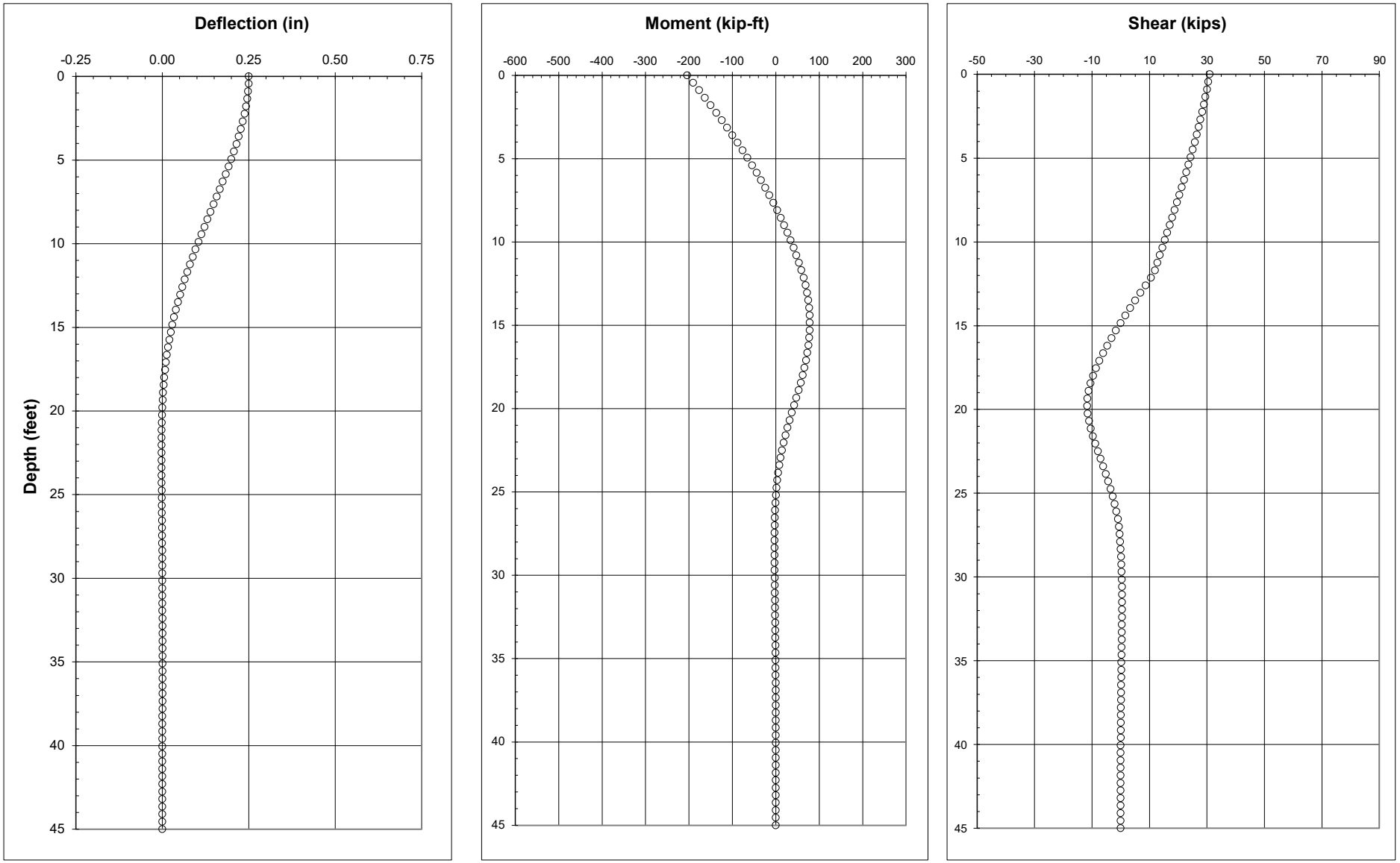




FREE HEAD LATERAL PILE ANALYSIS
24" CIDH PILE - 1/4 INCH DEFLECTION; 13.5 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

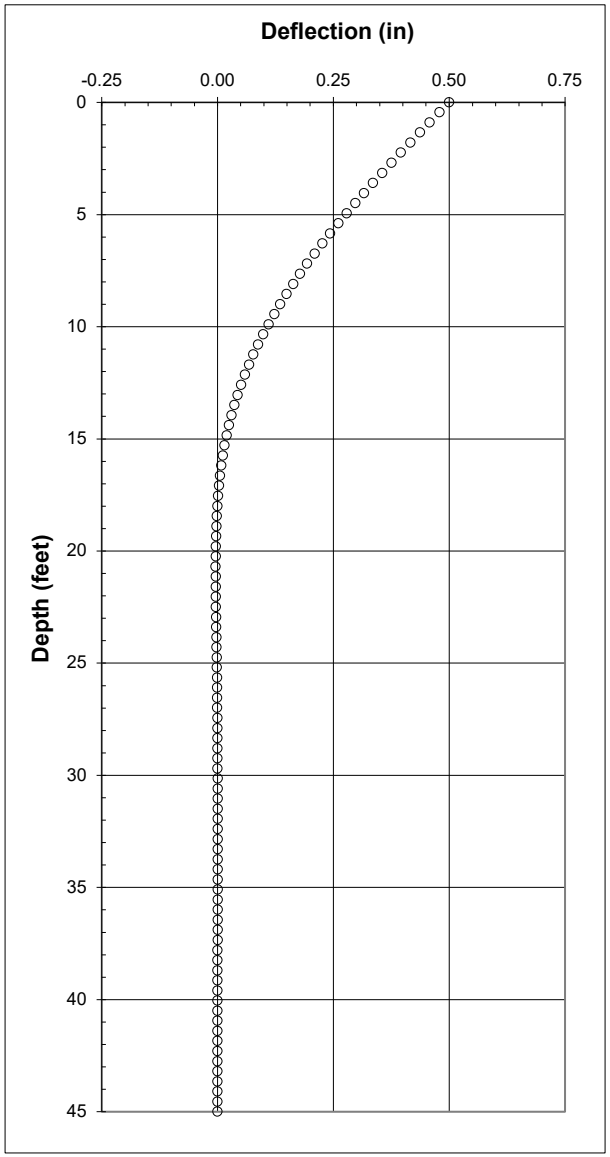
FIGURE 8A

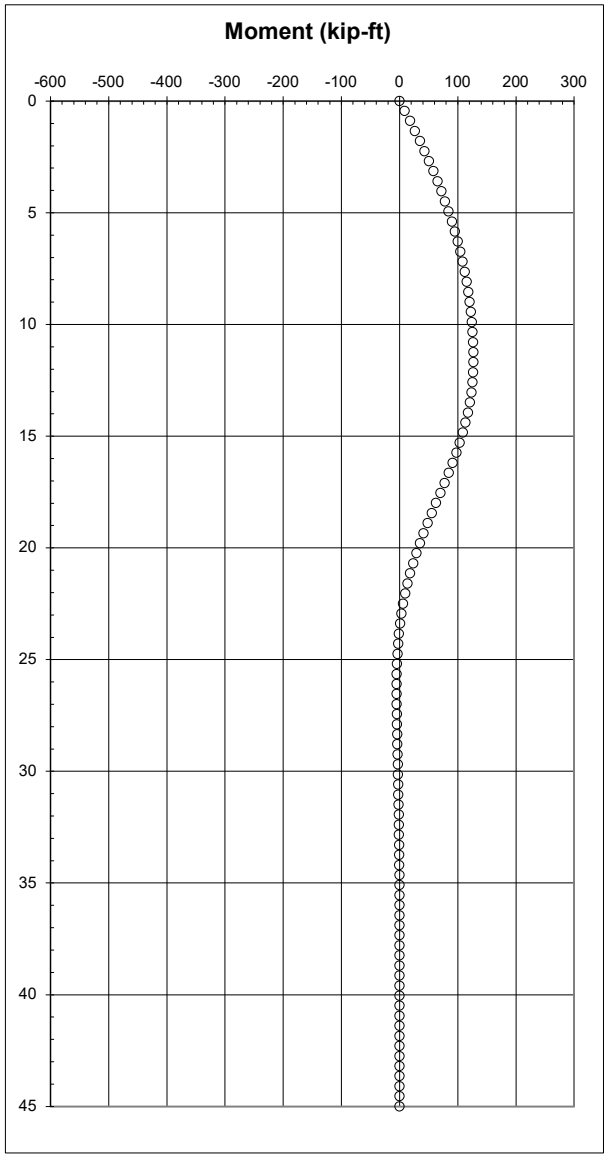


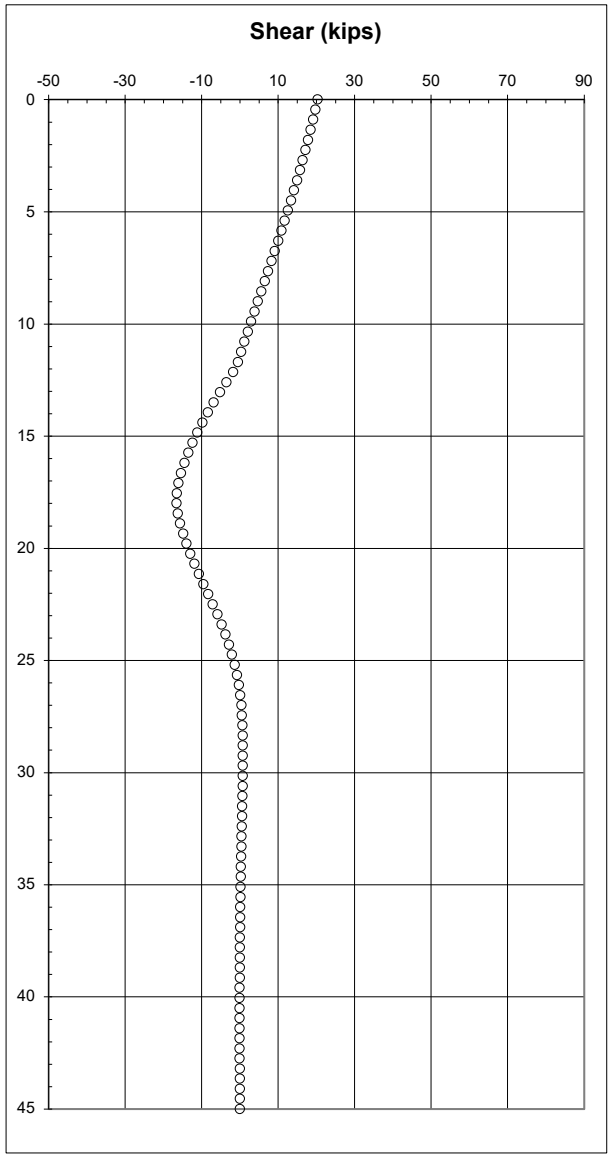
FIXED HEAD LATERAL PILE ANALYSIS
24" CIDH PILE - 1/4 INCH DEFLECTION; 31.0 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

FIGURE 8B



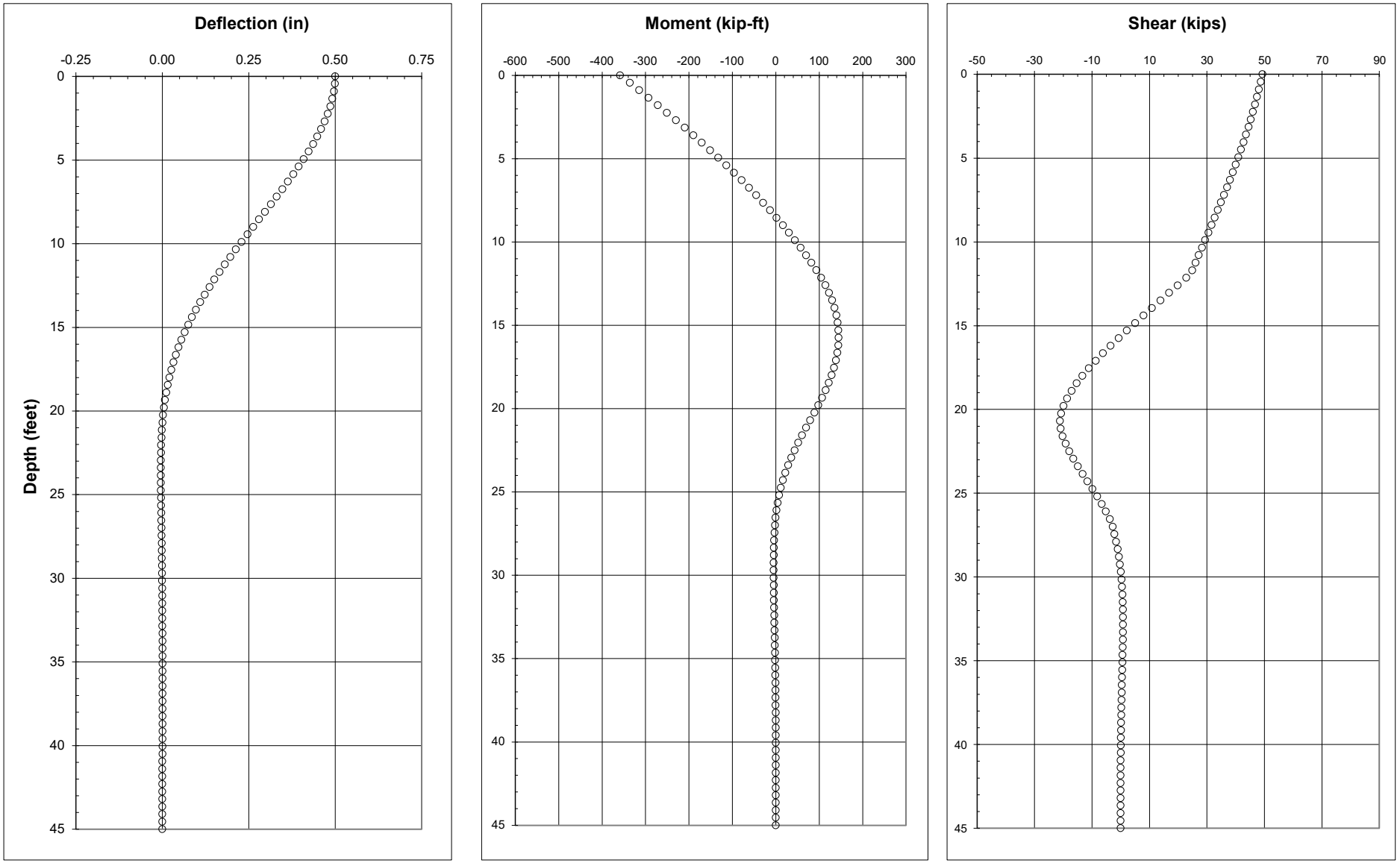




FREE HEAD LATERAL PILE ANALYSIS
24" CIDH PILE - 1/2 INCH DEFLECTION; 20.3 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

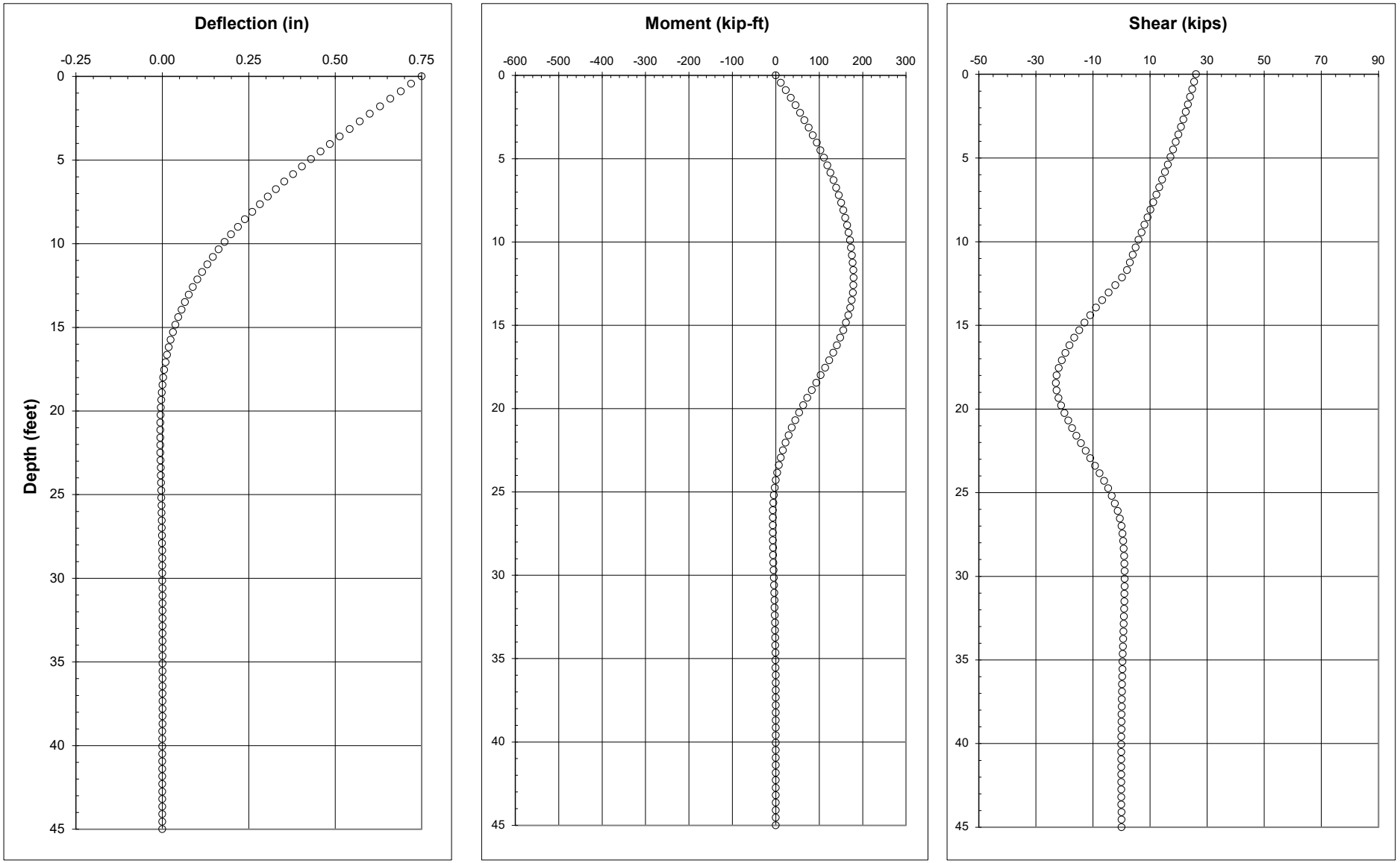
FIGURE 8C



FIXED HEAD LATERAL PILE ANALYSIS
24" CIDH PILE - 1/2 INCH DEFLECTION; 49.3 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

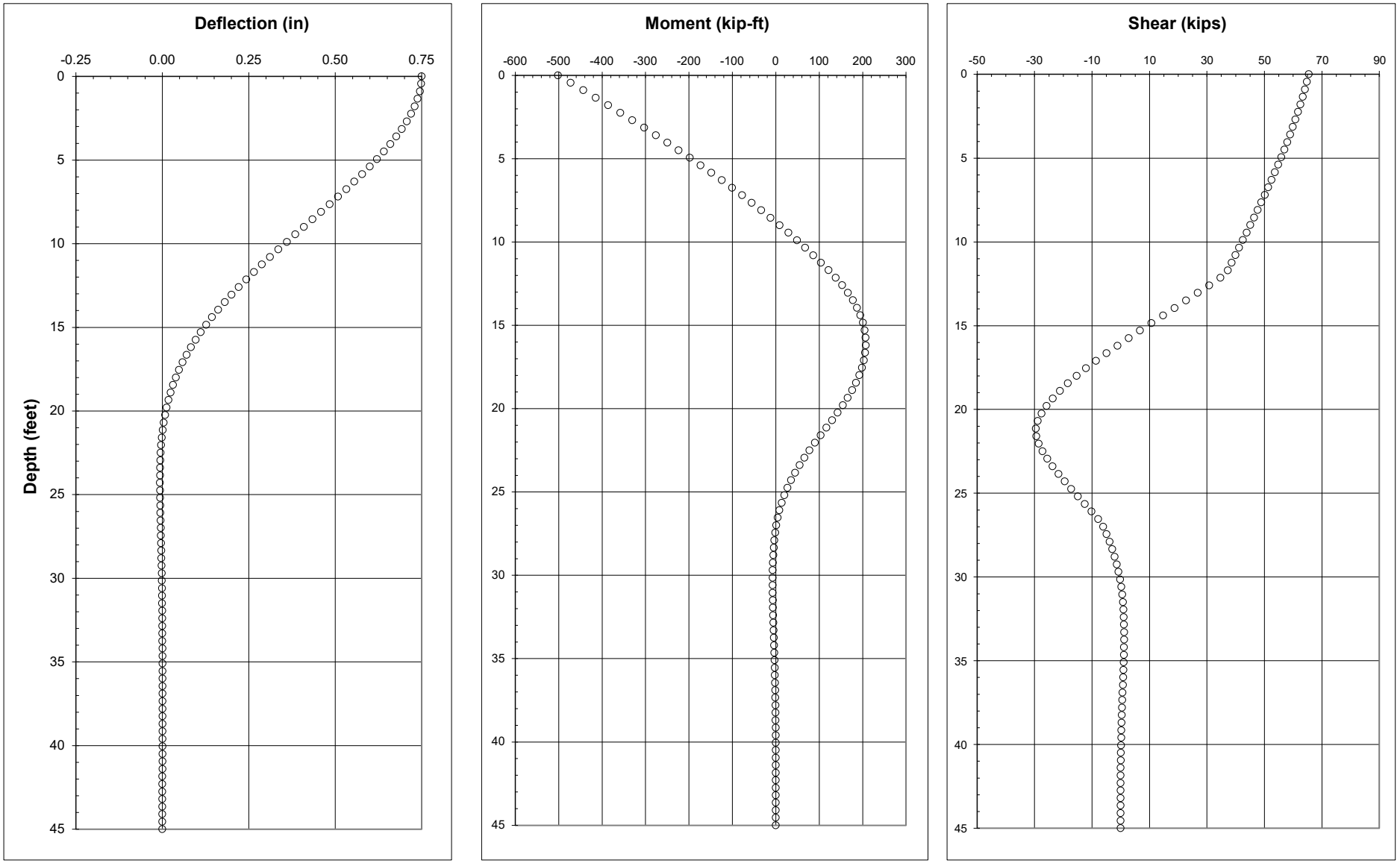
FIGURE 8D



FREE HEAD LATERAL PILE ANALYSIS
24" CIDH PILE - 3/4 INCH DEFLECTION; 26.1 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

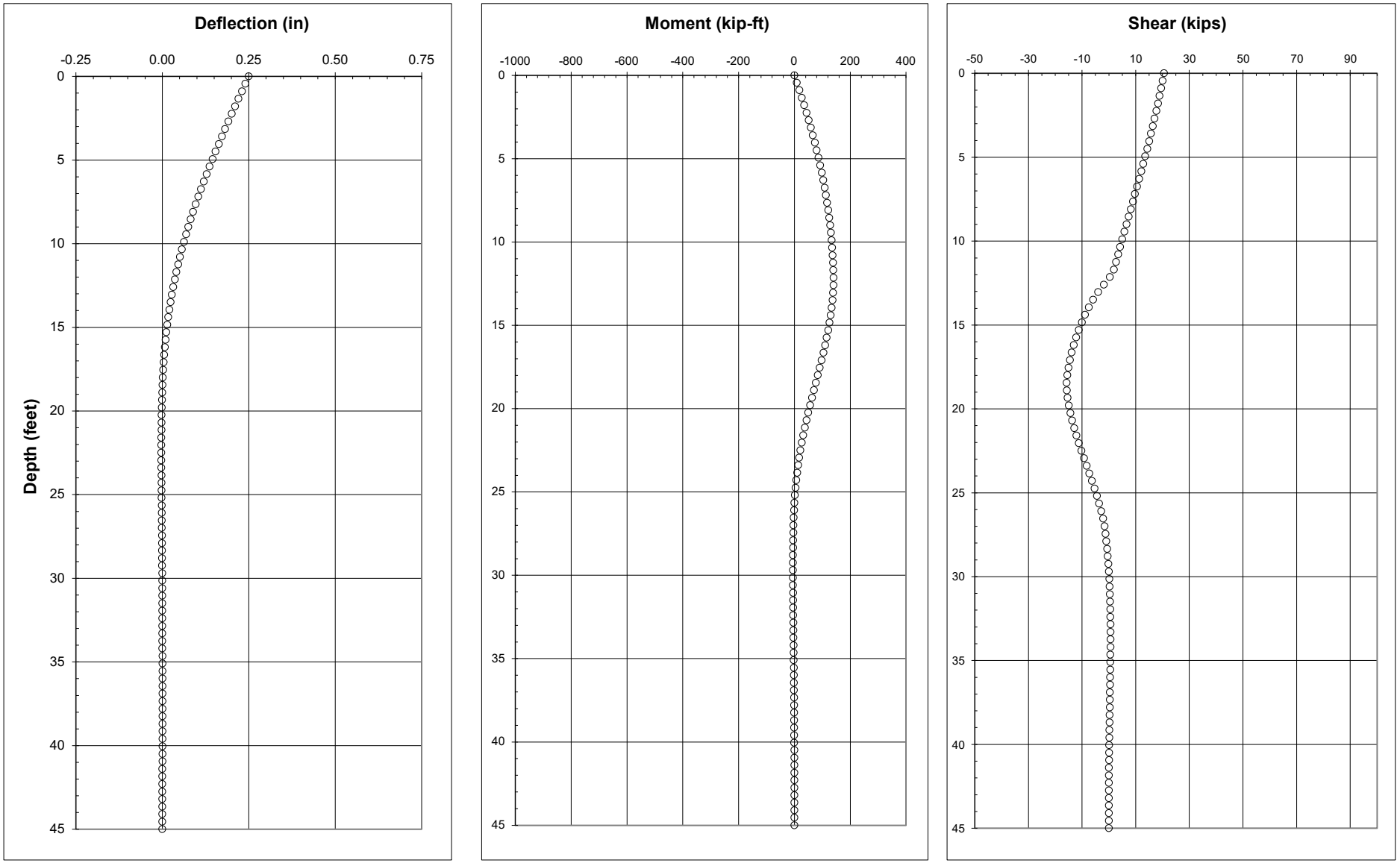
FIGURE 8E



FIXED HEAD LATERAL PILE ANALYSIS
24" CIDH PILE - 3/4 INCH DEFLECTION; 65.5 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

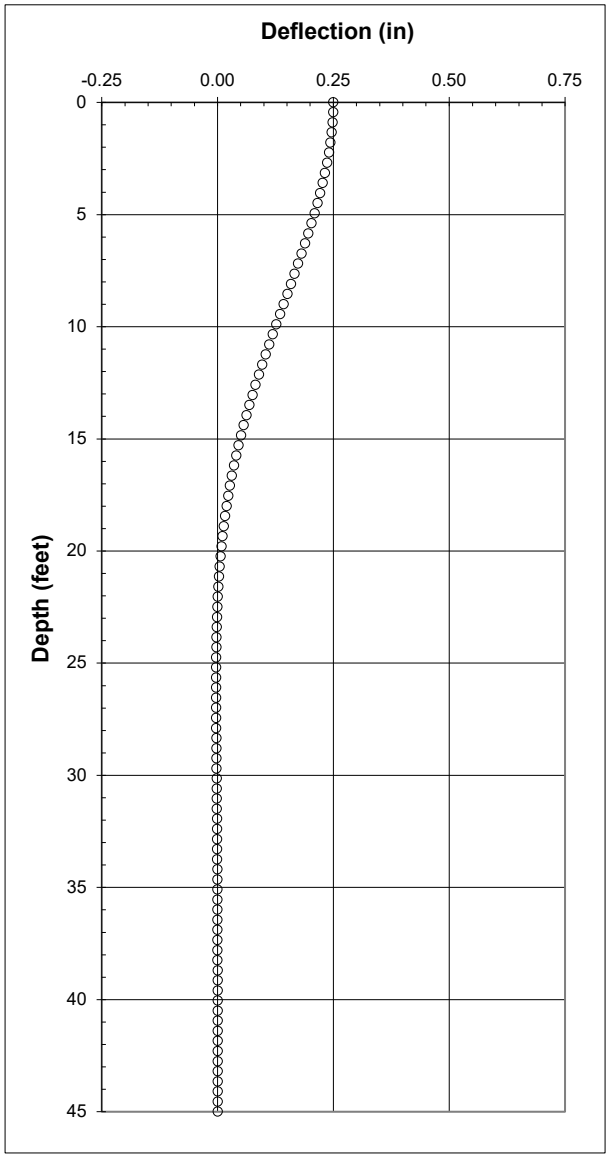
FIGURE 8F

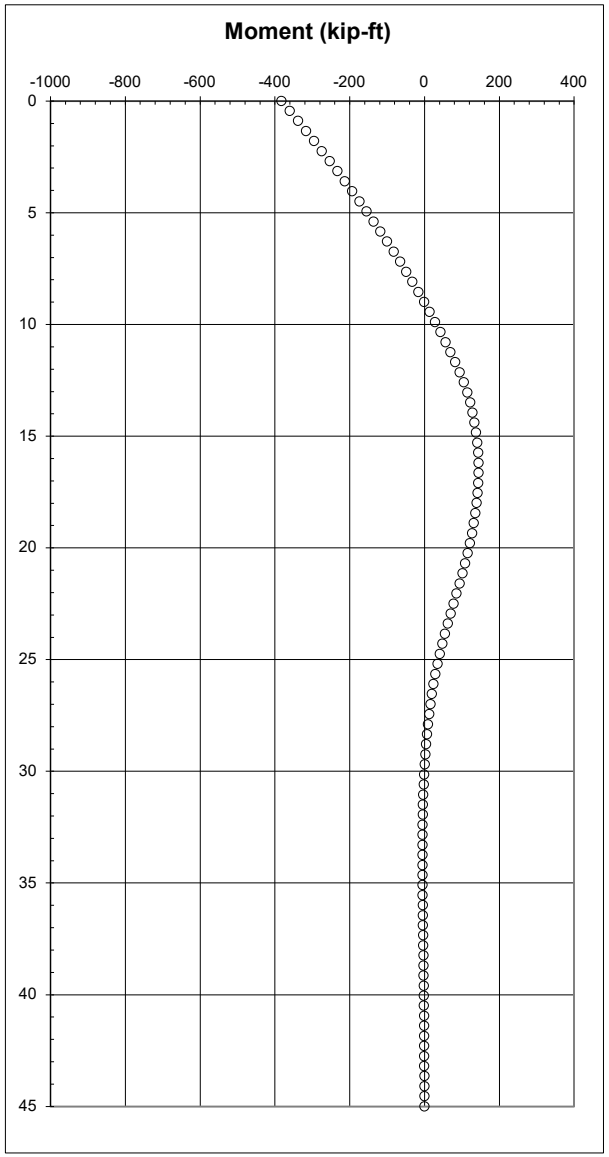


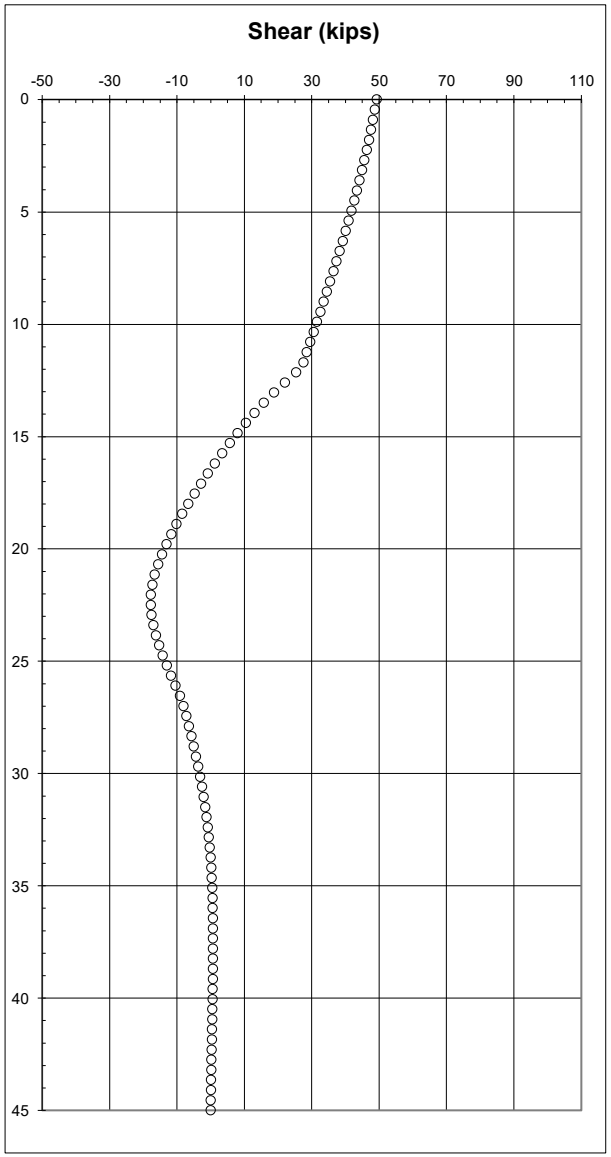
FREE HEAD LATERAL PILE ANALYSIS
30" CIDH PILE - 1/4 INCH DEFLECTION; 20.6 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

FIGURE 9A



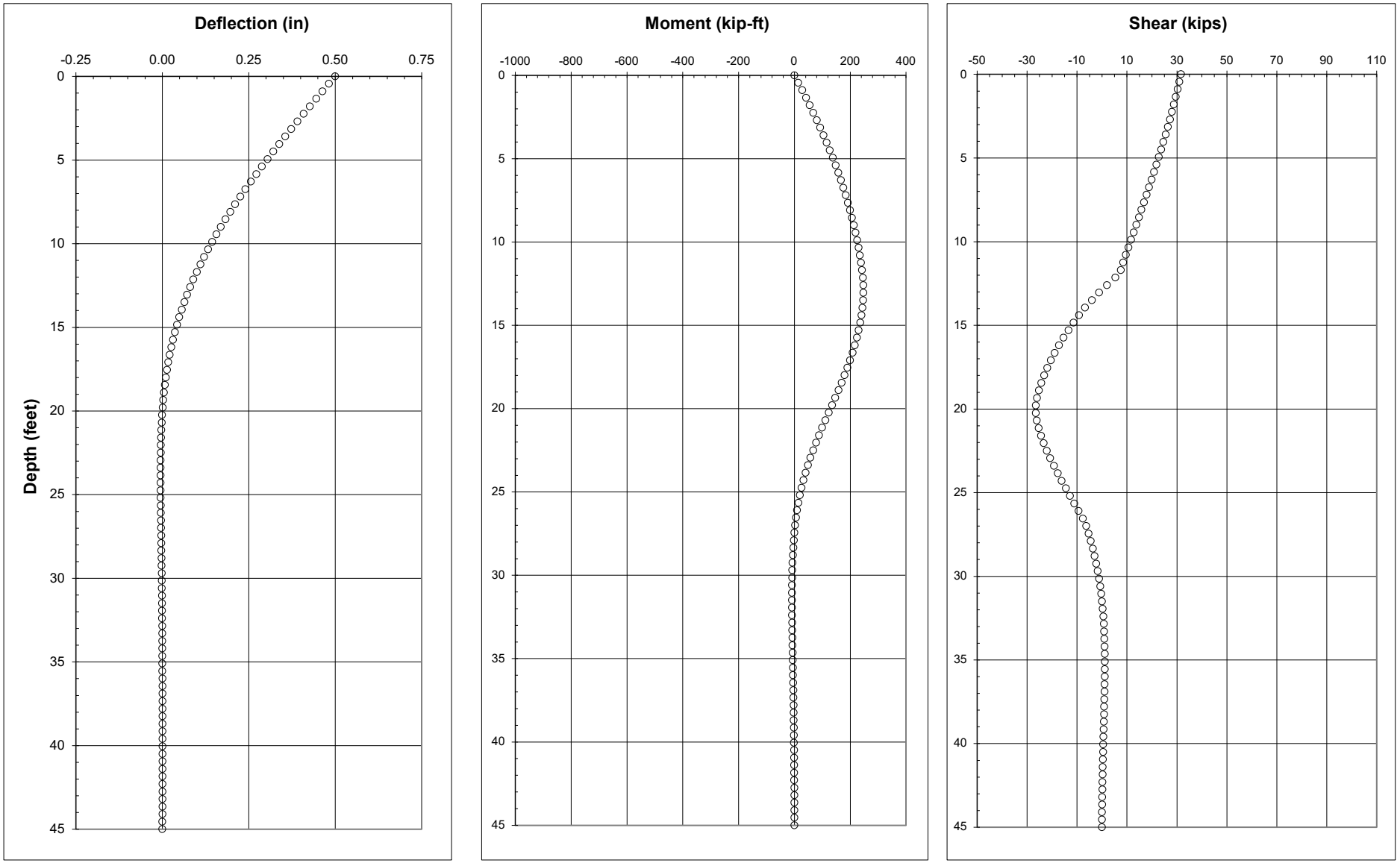




FIXED HEAD LATERAL PILE ANALYSIS
30" CIDH PILE - 1/4 INCH DEFLECTION; 49.3 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

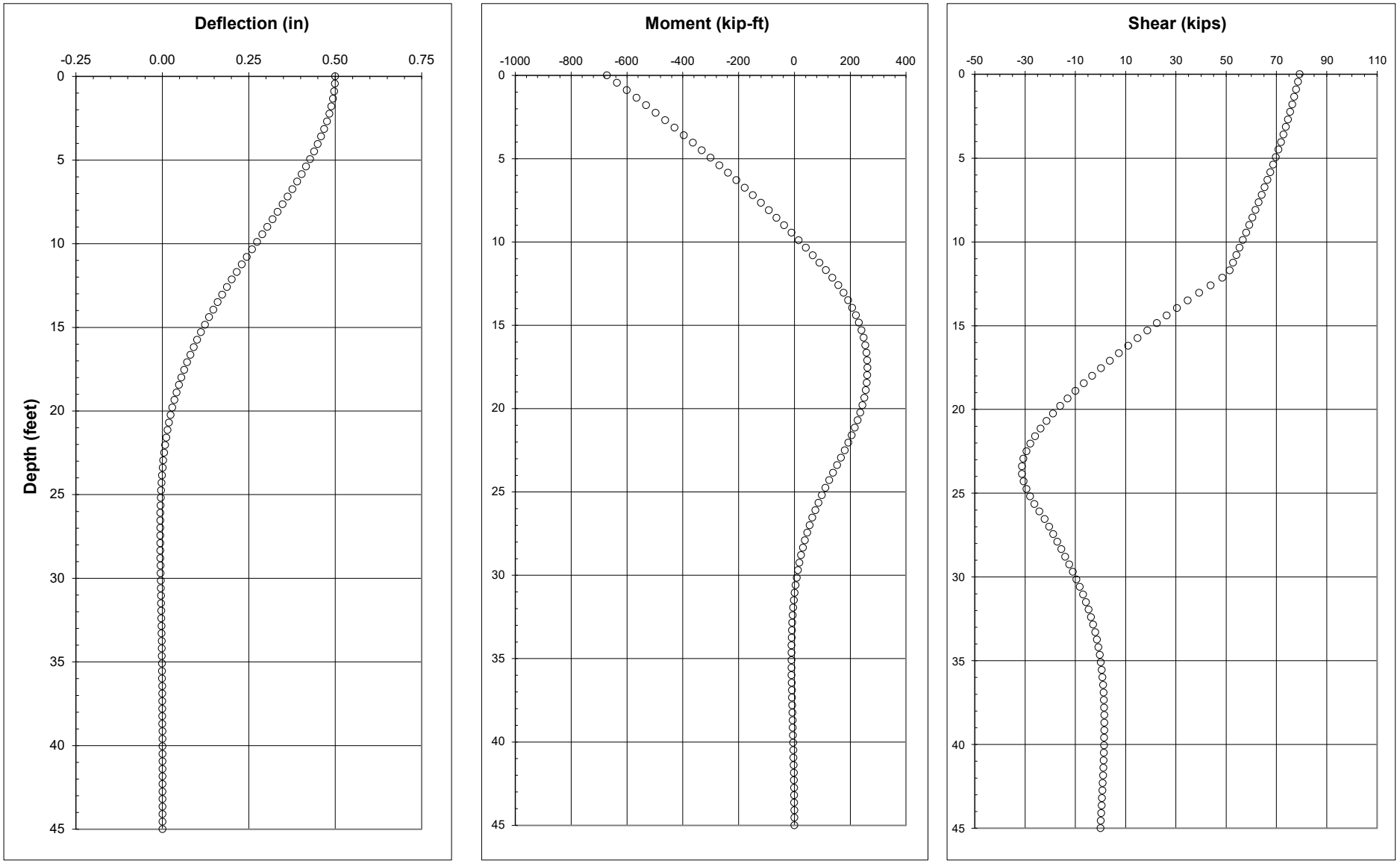
FIGURE 9B



FREE HEAD LATERAL PILE ANALYSIS
30" CIDH PILE - 1/4 INCH DEFLECTION; 31.7 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

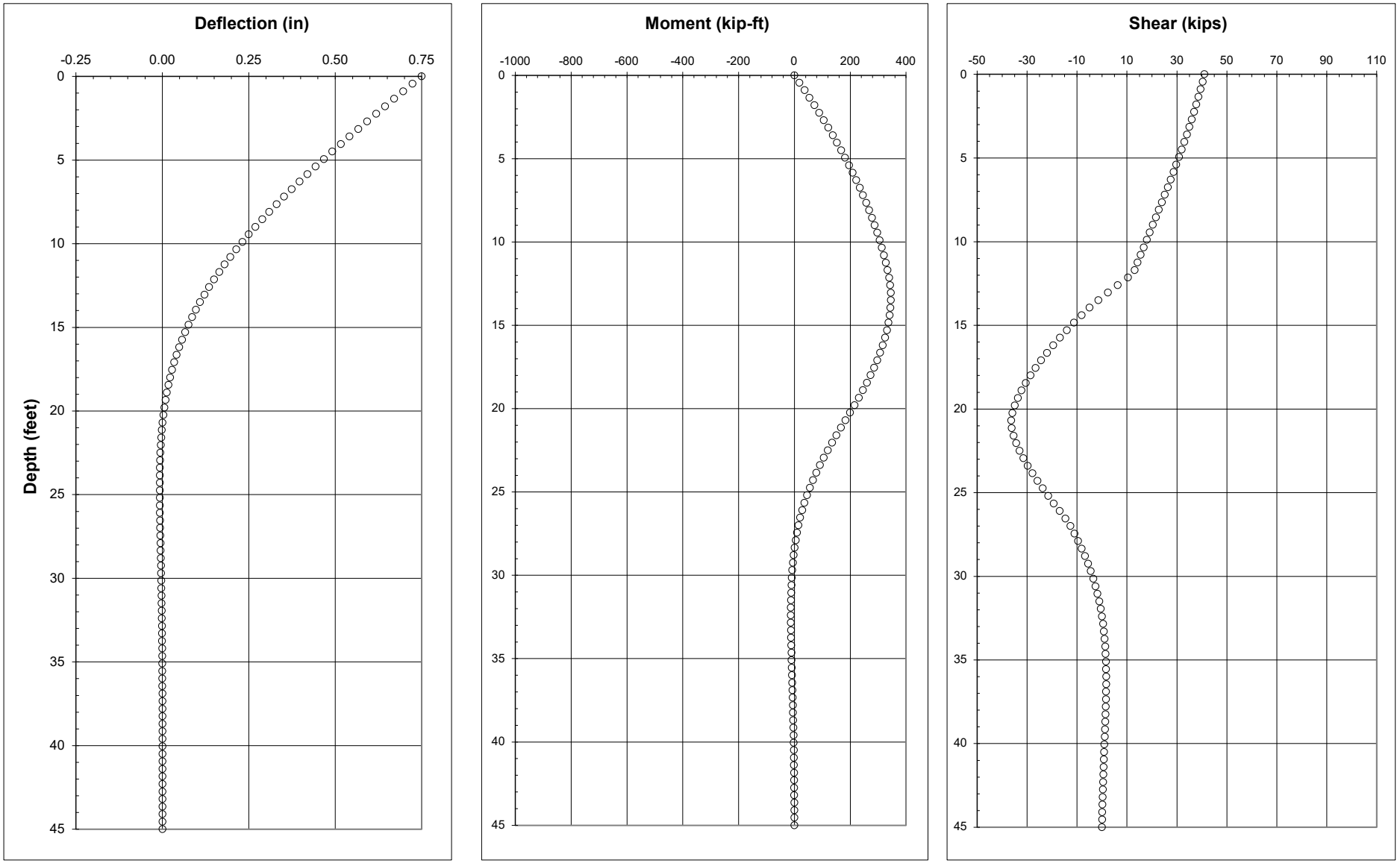
FIGURE 9C



FIXED HEAD LATERAL PILE ANALYSIS
30" CIDH PILE - 1/4 INCH DEFLECTION; 79.3 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

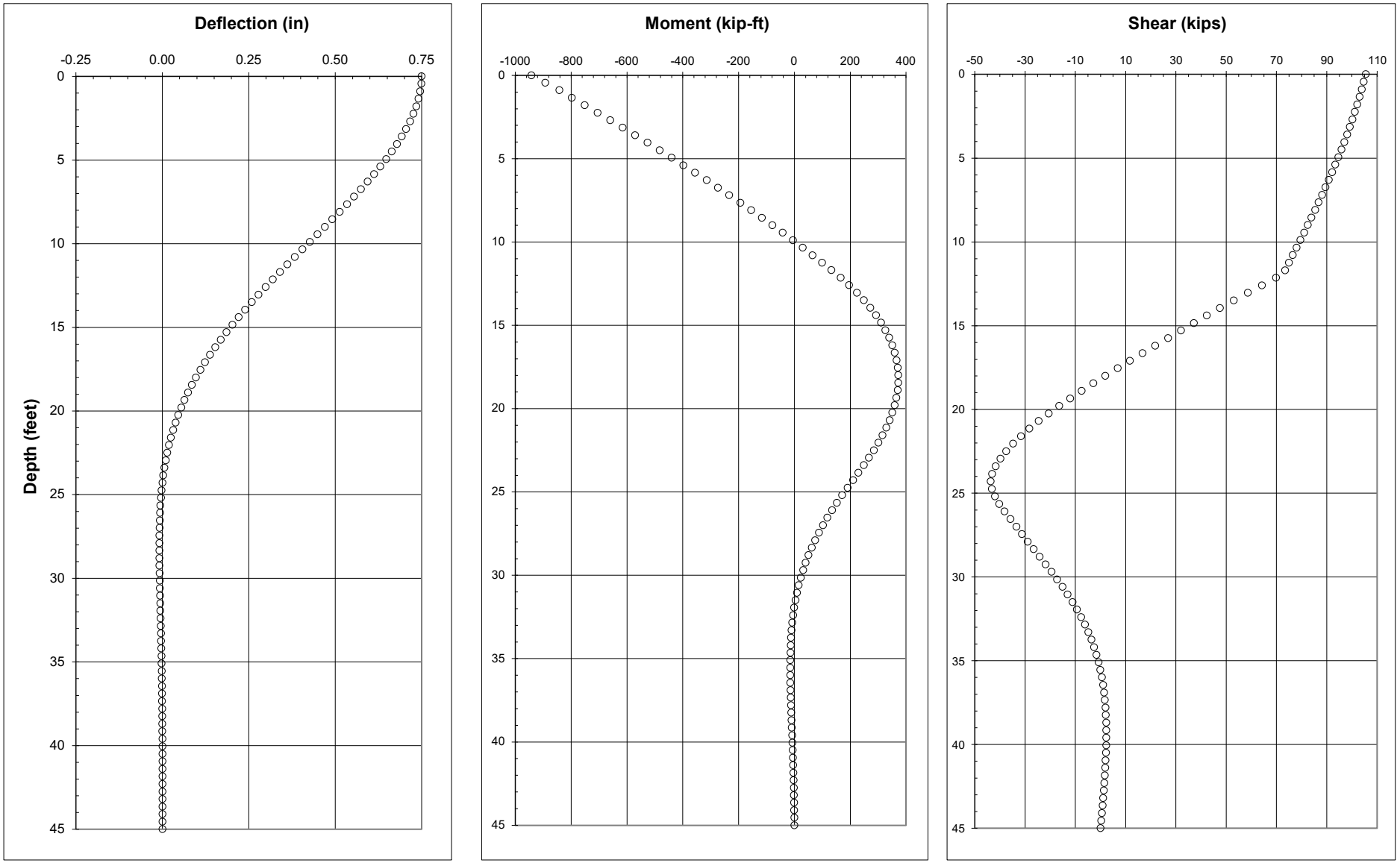
FIGURE 9D



FREE HEAD LATERAL PILE ANALYSIS
30" CIDH PILE - 1/4 INCH DEFLECTION; 41.1 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

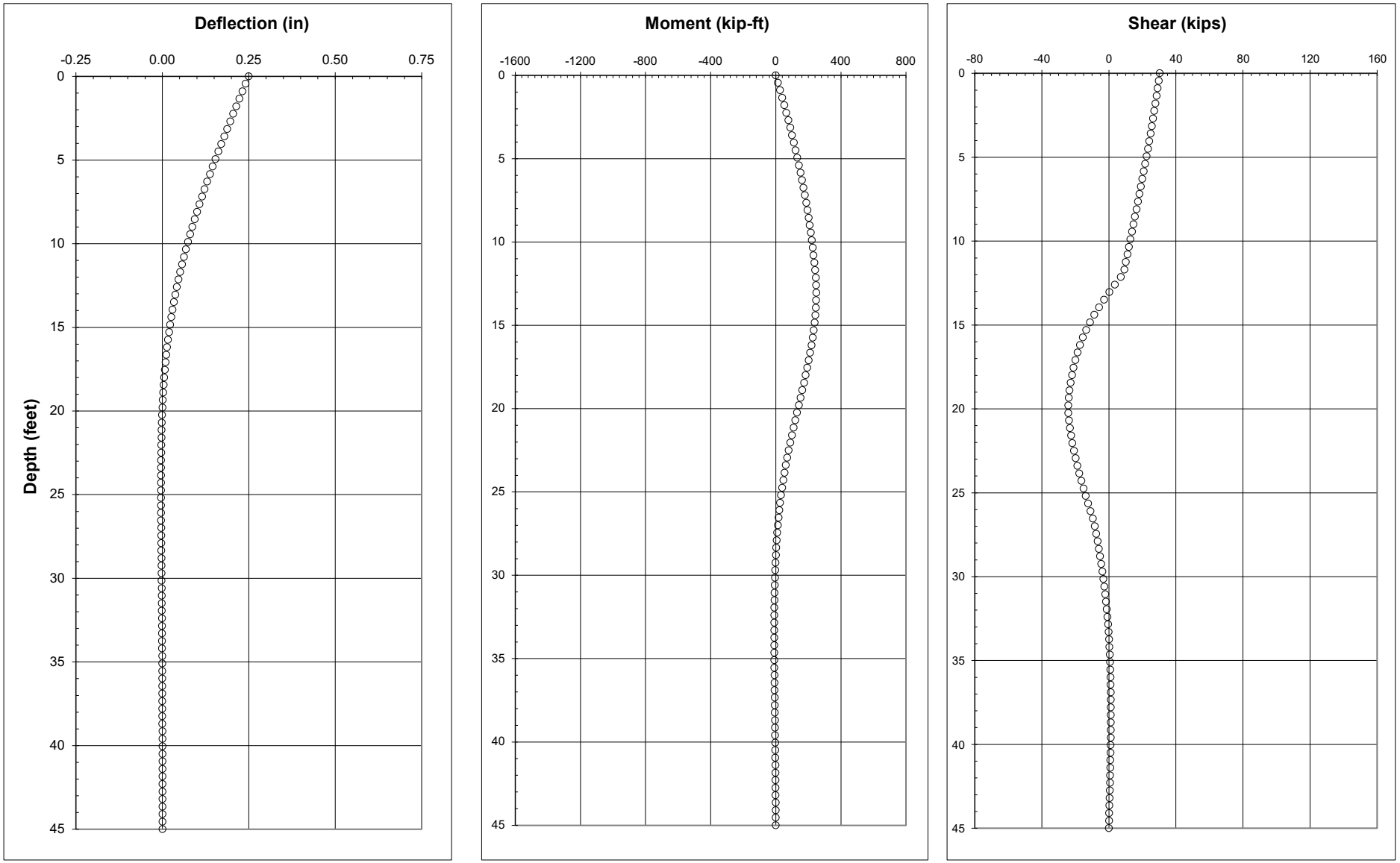
FIGURE 9E



FIXED HEAD LATERAL PILE ANALYSIS
30" CIDH PILE - 1/4 INCH DEFLECTION; 106.5 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

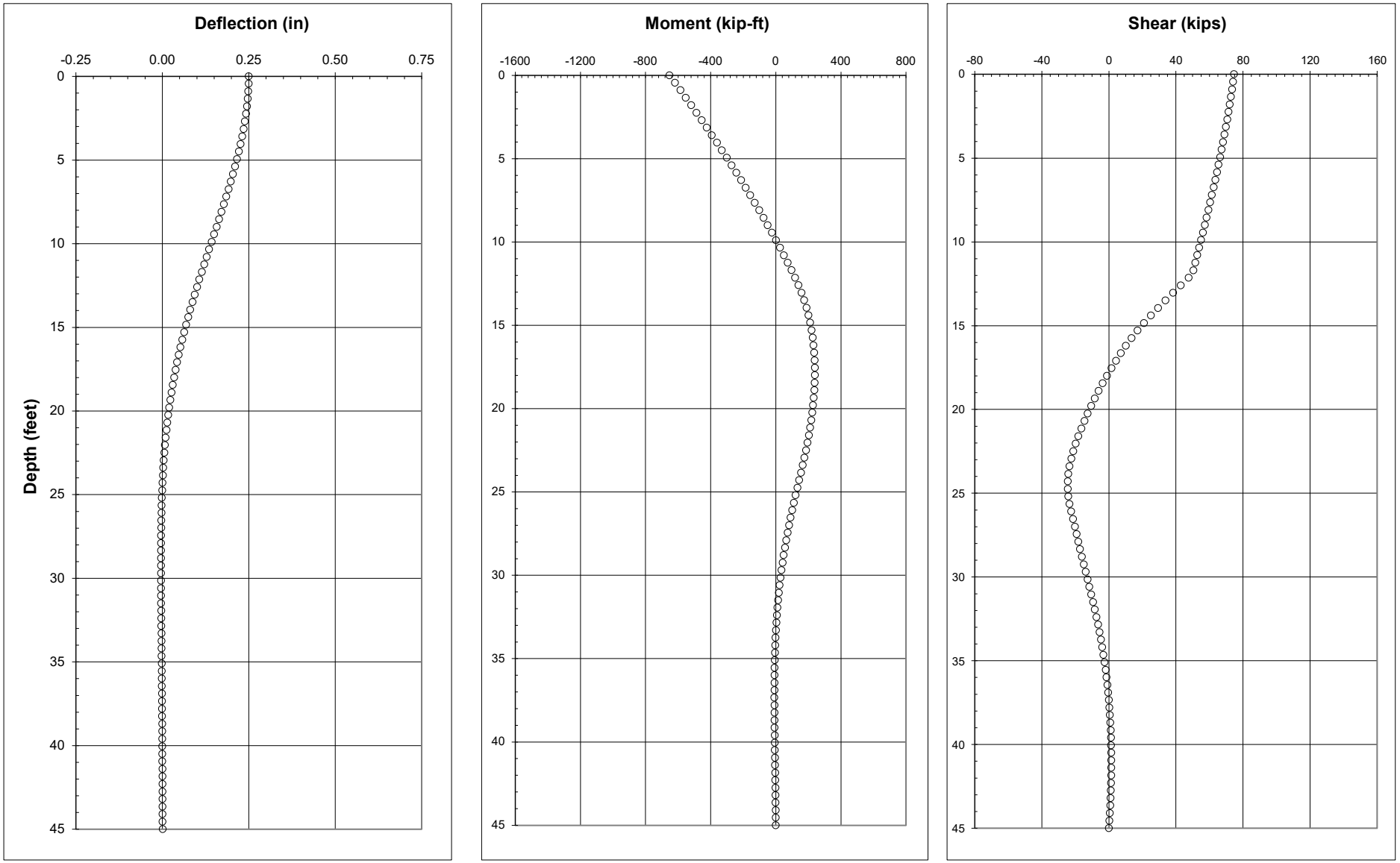
FIGURE 9F



FREE HEAD LATERAL PILE ANALYSIS
36" CIDH PILE - 1/4 INCH DEFLECTION; 30.4 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

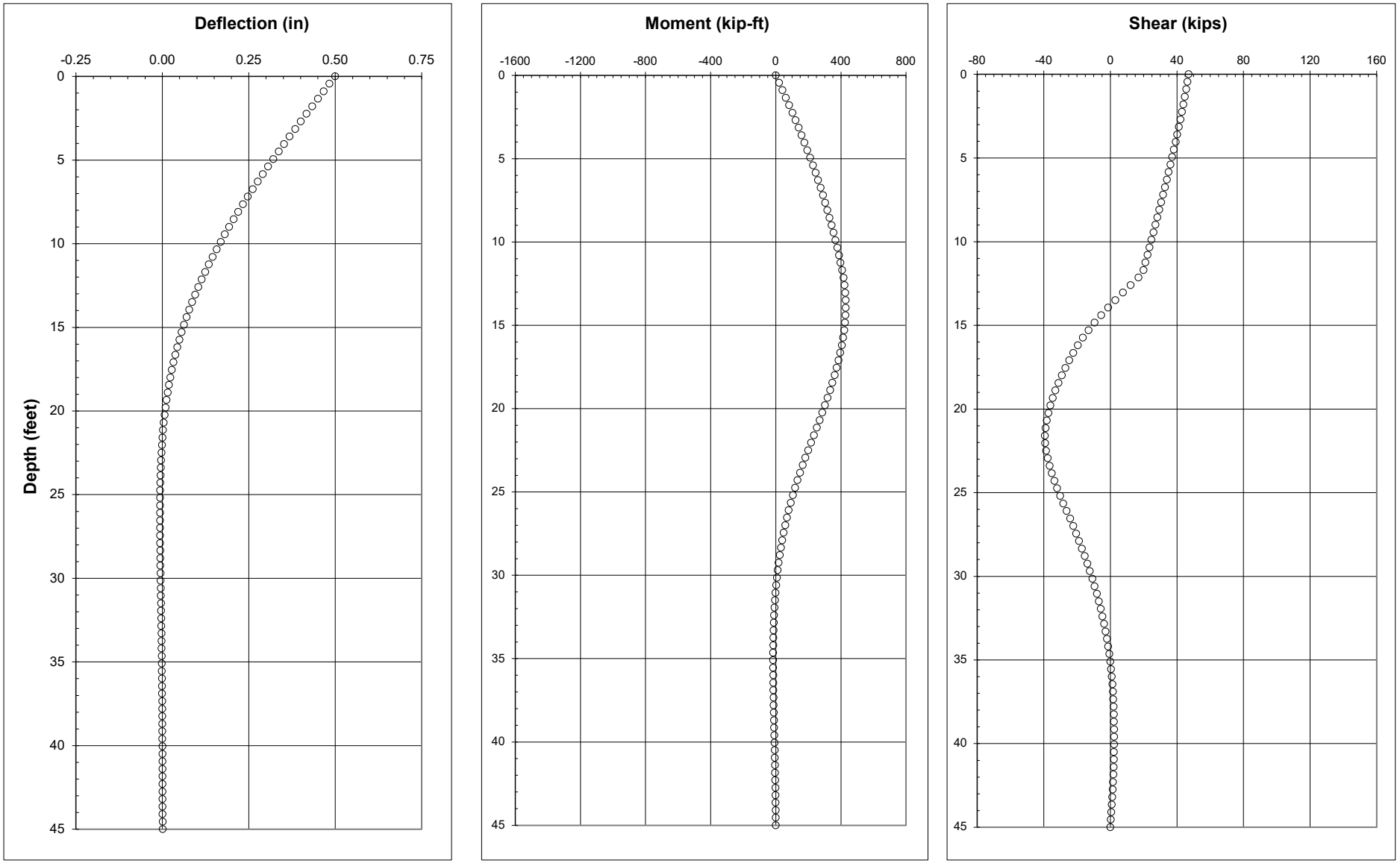
FIGURE 10A



FIXED HEAD LATERAL PILE ANALYSIS
36" CIDH PILE - 1/4 INCH DEFLECTION; 74.8 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

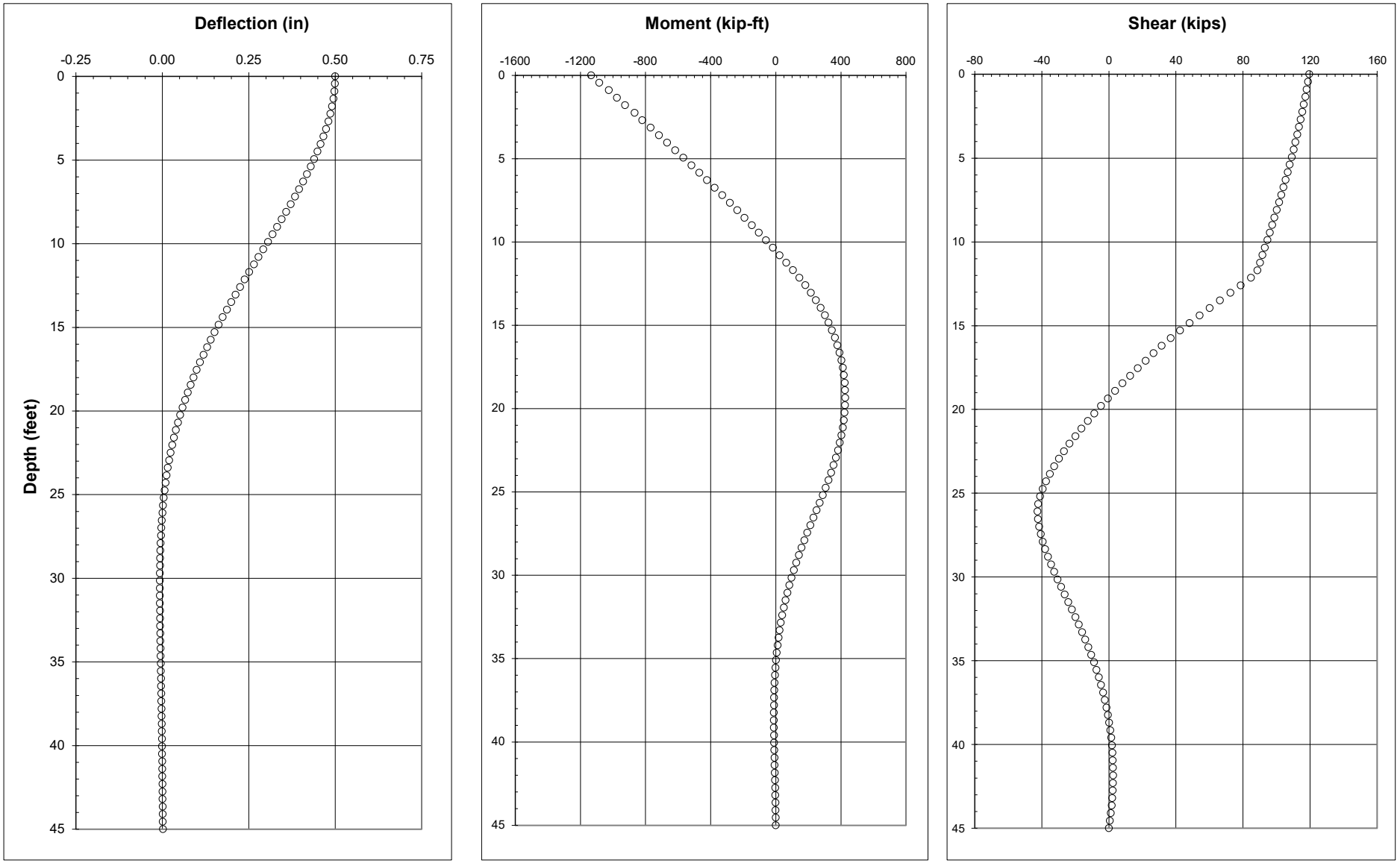
FIGURE 10B



FREE HEAD LATERAL PILE ANALYSIS
36" CIDH PILE - 1/4 INCH DEFLECTION; 47.1 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

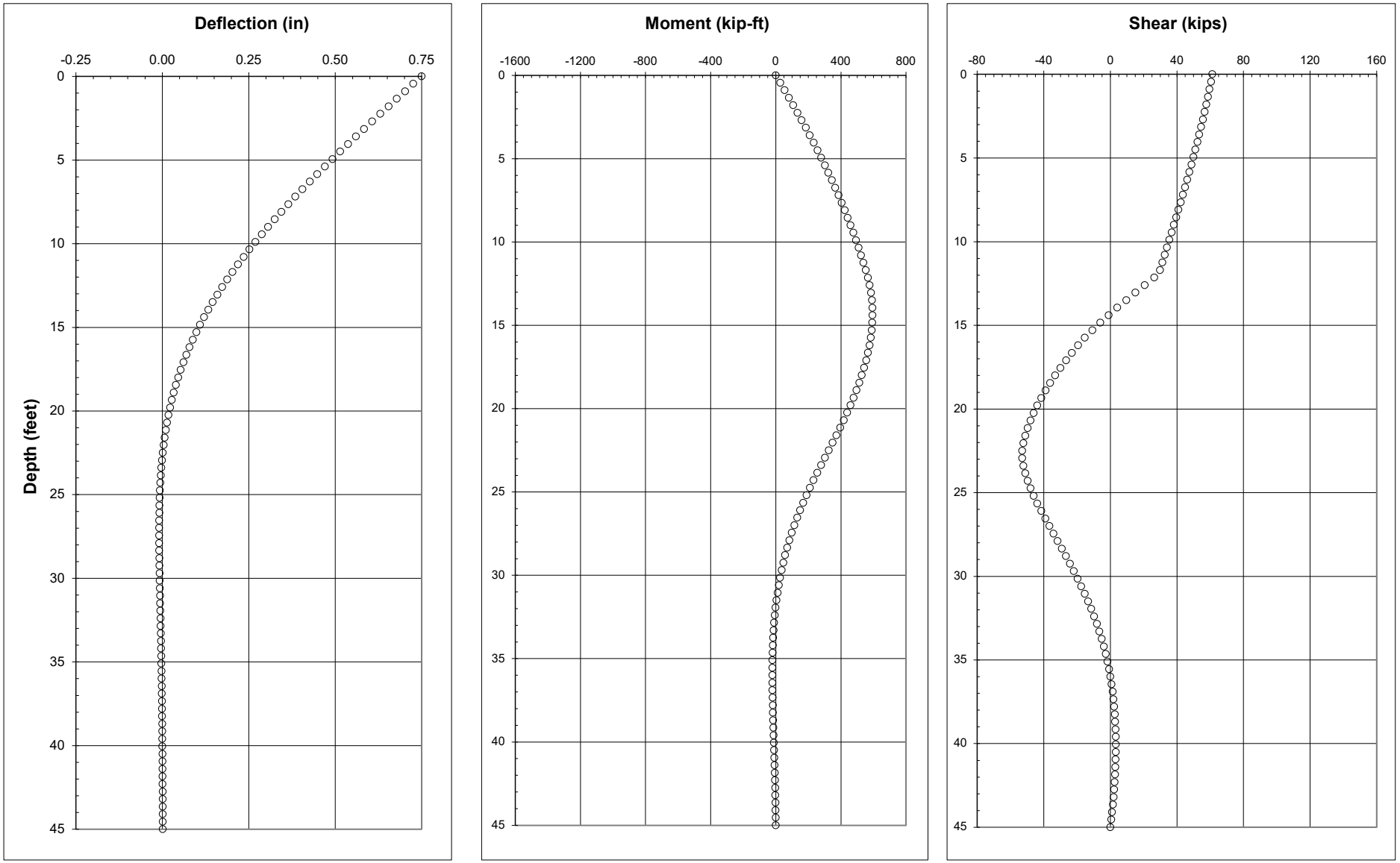
FIGURE 10C



FIXED HEAD LATERAL PILE ANALYSIS
36" CIDH PILE - 1/4 INCH DEFLECTION; 119.7 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

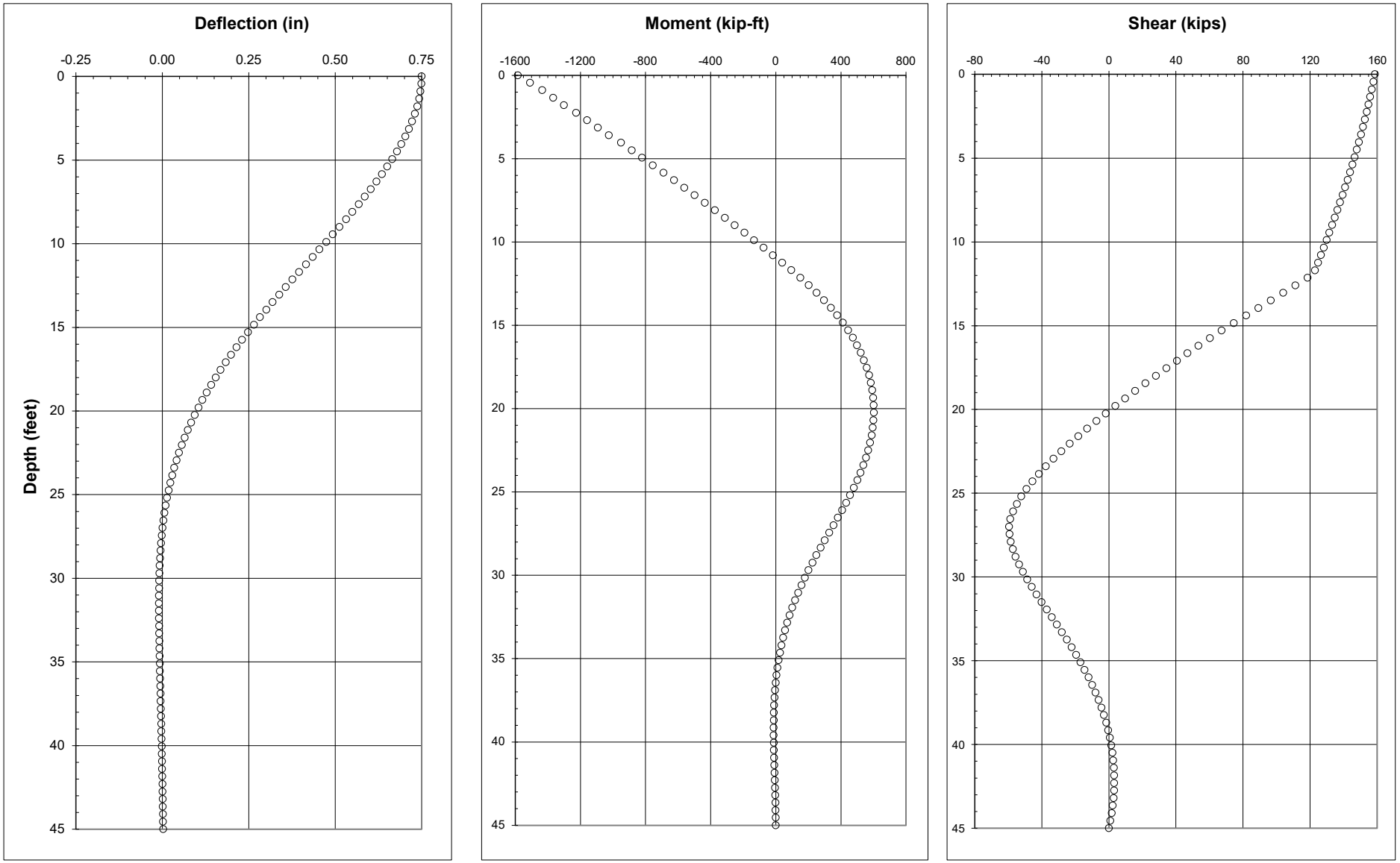
FIGURE 10D



FREE HEAD LATERAL PILE ANALYSIS
36" CIDH PILE - 1/4 INCH DEFLECTION; 61.3 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

FIGURE 10E



FIXED HEAD LATERAL PILE ANALYSIS
36" CIDH PILE - 1/4 INCH DEFLECTION; 158.7 KIPS
LAC+USC OLD PHARMACY WAREHOUSE, LOS ANGELES, CALIFORNIA
GPI Project No.: 3080.22I

GEOTECHNICAL PROFESSIONALS, INC.

FIGURE 10F

TABLE 1

ASCE 41-17: SITE-SPECIFIC SEISMIC REPONSE WORKSHEET - NEW BUILDINGS

Project	**LAC+USC Old Pharmacy Bld.**
Proj. No.	**3080.22I**
Latitude	**34.0607**
Longitude	**-118.2132**

NGA West2 Attentuation Relationships

1) Abrahamson-et al (2014)
2) Boore-et al (2014)
3) Cambell-Bozorgnia (2014)
4) Chiou-Youngs (2014)

Parameter	per ASCE 41-17
B_1	1.000
Damping, β	5%
T_L (sec)	8.0

BSE-2N	per ASCE 41-17
$S_{s,BSE-2N}$	1.989
$S_{1,BSE-2N}$	0.712
F_a	1.200
F_v	1.400
$S_{XS,BSE-2N}$	2.387
$S_{X1,BSE-2N}$	0.997
PGA = $0.4S_{XS}$	0.955
Site Class	C
T_o (sec)	0.084
T_s (sec)	0.418

C_{RS}	0.893
C_{R1}	0.894

BSE-1N	per ASCE 41-17
$S_{s,BSE-1N}$	1.326
$S_{1,BSE-1N}$	0.475
F_a	1.200
F_v	1.400
$S_{XS,BSE-1N}$	1.591
$S_{X1,BSE-1N}$	0.665
PGA = $0.4S_{XS}$	0.636
Site Class	C
T_o (sec)	0.084
T_s (sec)	0.418

SITE-SPECIFIC PARAMETERS

BSE-2N

$S_{XS,BSE-2N}$	**2.211**
$S_{X1,BSE-2N}$	**1.742**
PGA_{BSE-2N}	**0.974**

BSE-1N

$S_{XS,BSE-1N}$	**1.474**
$S_{X1,BSE-1N}$	**1.161**
PGA_{BSE-1N}	**0.649**

1)	2)	3)	4)	5)	6)	7)	8)	9)	10)	11)	12)
Period (sec)	BSE-2N General Hor. Response Spectrum (g)	BSE-1N General Hor. Response Spectrum (g)	Risk Coefficient, C_R	BSE-2N Lower Limit Spectrum (g)	Probabilistic MCE Spectrum (2% in 50 yrs) (g)	Probabilistic MCE_R Spectrum (2% in 50 yrs) w/ C_R (g)	84th Percentile Deterministic Spectrum	2/3 Site Specific MCE_R Spectrum	80% of 2019 CBC Design Spectrum	**Site Specific BSE-2N Response Spectrum**	**Site Specific BSE-1N Response Spectrum**
0.000	0.955	0.636	0.893	0.764	1.090	0.974	1.346	0.649	0.223	**0.974**	**0.649**
0.050	1.812	1.208	0.893	1.450	1.451	1.295	1.624	0.864	0.765	**1.295**	**0.864**
0.082	2.364	1.576	0.893	1.892	1.683	1.503	2.041	1.002	0.930	**1.503**	**1.002**
0.084	2.387	1.591	0.893	1.910	1.692	1.511	2.058	1.007	0.937	**1.511**	**1.007**
0.100	2.387	1.591	0.893	1.910	1.811	1.617	2.272	1.078	1.021	**1.617**	**1.078**
0.200	2.387	1.591	0.893	1.910	2.375	2.121	3.003	1.414	1.273	**2.121**	**1.414**
0.300	2.387	1.591	0.893	1.910	2.751	2.457	3.405	1.638	1.273	**2.457**	**1.638**
0.400	2.387	1.591	0.893	1.910	2.744	2.451	3.356	1.634	1.273	**2.451**	**1.634**
0.411	2.387	1.591	0.893	1.910	2.744	2.451	3.328	1.634	1.273	**2.451**	**1.634**
0.418	2.387	1.591	0.893	1.910	2.743	2.450	3.311	1.634	1.273	**2.450**	**1.634**
0.500	1.994	1.329	0.893	1.595	2.737	2.445	3.099	1.630	1.273	**2.445**	**1.630**
0.600	1.662	1.108	0.894	1.329	2.553	2.281	2.839	1.520	1.273	**2.281**	**1.520**
0.700	1.424	0.949	0.894	1.139	2.368	2.116	2.578	1.411	1.273	**2.116**	**1.411**
0.750	1.329	0.886	0.894	1.063	2.276	2.034	2.448	1.356	1.266	**2.034**	**1.356**
0.800	1.246	0.831	0.894	0.997	2.210	1.975	2.336	1.317	1.187	**1.975**	**1.317**
0.900	1.108	0.738	0.894	0.886	2.079	1.858	2.114	1.239	1.055	**1.858**	**1.239**
1.000	0.997	0.665	0.894	0.798	1.948	1.742	1.891	1.161	0.949	**1.742**	**1.161**
2.000	0.498	0.332	0.894	0.399	1.051	0.940	0.796	0.531	0.475	**0.796**	**0.531**
3.000	0.332	0.222	0.894	0.266	0.679	0.607	0.442	0.294	0.316	**0.475**	**0.316**
4.000	0.249	0.166	0.894	0.199	0.475	0.425	0.275	0.183	0.237	**0.356**	**0.237**
5.000	0.199	0.133	0.894	0.160	0.362	0.323	0.197	0.131	0.190	**0.285**	**0.190**

(DJB 04/28/20; Based Upon ASCE 41-17)

TABLE 1
ASCE 41-17: SITE-SPECIFIC SEISMIC REPONSE WORKSHEET - NEW BUILDINGS

EXPLANATION: NOTES AND REFERENCES

INPUT BLUE ONLY - BLACK/RED CALCULATED

Column Descriptions

01) Periods including To and Ts calculated from Section 11.4.6 (ASCE 7-16)

02) OSHPD, Seismic Design Maps Web Application - General Horizontal Response Spectrum (GHRS): $S_{BSE\text{-}2N}$: per ASCE 41-17, Section 2.4.1.7.1

03) OSHPD, Seismic Design Maps Web Application - General Horizontal Response Spectrum (GHRS): $S_{BSE\text{-}1N}$: per ASCE 41-17, Section 2.4.1.7.1

04) Risk Coefficient, C_R, for 0.2s and 1.0s periods (ASCE 7-16, Section 21.2.1.1); from OSHPD web application

05) Minimum Spectral Amplitude - 80% of GHRS for $S_{BSE\text{-}2N}$ (Column 2); (ASCE 41-13; Section 2.4.2.1.2) - NO LONGER APPLICABLE

06) USGS Unified Hazard Tool (UHT), 2% in 50 years Probabilistic Spectrum; scaled w/ MRC factors per Huang et al (2008); per ASCE 7-16, Section 21.2.1.1

07) USGS UHT, Probabilistic MCE_R Spectrum: Product of 2% in 50yr Spectrum and Risk Coefficient (Col. 4 * Col. 6); (ASCE 7-16, Section 21.2.1.1)

08) PEER Ground Motion Database, 84th Percentile Deterministic Spectrum; controlling fault source (ASCE 7-16; Section 21.2.2)

09) Uncorrected Design Response Spectrum (ASCE 7-16 Sec. 21.3), 2/3 * Lesser of Col. 7 & Greater of Cols. 5 & 8 (not less than 80% PGAM per Sec. 21.5.3)

10) 80% of 2019 CBC Design Spectra (Column 3), (ASCE 7-16, Section 21.3) Lower Limit of the Design Spectrum

11) Site Specific BSE-2N Response Spectrum (ASCE 41-17; Section 2.4.1.1); 1.5 Times Column 12

12) Site Specific BSE-1N Response Spectrum (ASCE 41-17; Section 2.4.1.2); Greater of Columns 9 and 10

T_L = Figure 22-12 ASCE 7-16 (typically 8 sec Southern California)

**Code-based values obtained from OSHPD Web Application: https://seismicmaps.org/

MUST CHECK THAT VALUES EXCEED MINIMUMS

BSE-2N		***BSE-1N***	
Minimum Allowable Value of $S_{XS,BSE\text{-}2N}$:	**2.211**	Minimum Allowable Value of $S_{XS,BSE\text{-}1N}$:	**1.474**
(Maximum of 90% of S_{XS} at any period)		(Maximum of 90% of S_{XS} at any period)	
Minimum Allowable Value of $S_{X1,BSE\text{-}2N}$:	**1.742**	Minimum Allowable Value of $S_{X1,BSE\text{-}1N}$:	**1.161**
(Maximum of $T*S_{XA}$ for periods from 1 to 5 seconds)		(Maximum of $T*S_{XA}$ for periods from 1 to 5 seconds)	

(DJB 04/28/20; Based Upon ASCE 41-17)

TABLE 2

ASCE 41-17: SITE-SPECIFIC SEISMIC REPONSE WORKSHEET - EXISTING BUILDINGS

Project	**LAC+USC Old Pharmacy Bld.**
Proj. No.	**3080.22I**
Latitude	**34.0607**
Longitude	**-118.2132**

NGA West2 Attentuation Relationships

1) Abrahamson-et al (2014)
2) Boore-et al (2014)
3) Cambell-Bozorgnia (2014)
4) Chiou-Youngs (2014)

Parameter	per ASCE 41-17
B_1	1.000
Damping, β	5%
T_L (sec)	8.0

BSE-2E	per ASCE 41-17
$S_{s,BSE-2E}$	1.517
$S_{1,BSE-2E}$	0.526
F_a	1.200
F_v	1.474
$S_{XS,BSE-2E}$	1.820
$S_{X1,BSE-2E}$	0.775
PGA = $0.4S_{XS}$	0.728
Site Class	C
T_o (sec)	0.085
T_s (sec)	0.426

C_{RS}	0.893
C_{R1}	0.894

BSE-1E	per ASCE 41-17
$S_{s,BSE-1N}$	0.700
$S_{1,BSE-1N}$	0.234
F_a	1.220
F_v	1.500
$S_{XS,BSE-1E}$	0.854
$S_{X1,BSE-1E}$	0.351
PGA = $0.4S_{XS}$	0.342
Site Class	C
T_o (sec)	0.082
T_s (sec)	0.411

SITE-SPECIFIC PARAMETERS

BSE-2E

$S_{XS,BSE-2E}$	**1.953**
$S_{X1,BSE-2E}$	**1.067**
PGA_{BSE-2E}	**0.826**

BSE-1E

$S_{XS,BSE-1E}$	**1.040**
$S_{X1,BSE-1E}$	**0.483**
PGA_{BSE-1E}	**0.433**

1)	2)	3)	4)	5)	6)	7)	8)	9)	10)	11)	12)
Period (sec)	BSE-2E General Hor. Response Spectrum (g)	BSE-1E General Hor. Response Spectrum (g)	Risk Coefficient C_R	Probabilistic MCE_R Spectrum (2% in 50 yrs) w/ C_R (g)	Spectral Accelerations (5% in 50 yrs) (g)	Spectral Accelerations (20% in 50 yrs) (g)	84th Percentile Deterministic Spectrum	Site Specific BSE-2N Response Spectrum	Site Specific BSE-1N Response Spectrum	**Site Specific BSE-2E Response Spectrum**	**Site Specific BSE-1E Response Spectrum**
0.000	0.728	0.342	0.893	0.974	0.826	0.433	1.346	0.974	0.649	**0.826**	**0.433**
0.050	1.369	0.653	0.893	1.295	1.166	0.624	1.624	1.295	0.864	**1.166**	**0.624**
0.082	1.782	0.854	0.893	1.503	1.385	0.748	2.041	1.503	1.002	**1.385**	**0.748**
0.085	1.821	0.854	0.893	1.511	1.405	0.759	2.058	1.511	1.007	**1.405**	**0.759**
0.100	1.821	0.854	0.893	1.617	1.506	0.816	2.272	1.617	1.078	**1.506**	**0.816**
0.200	1.821	0.854	0.893	2.121	1.953	1.040	3.003	2.121	1.414	**1.953**	**1.040**
0.300	1.821	0.854	0.893	2.457	2.032	1.030	3.405	2.457	1.638	**2.032**	**1.030**
0.400	1.821	0.854	0.893	2.451	1.895	0.936	3.356	2.451	1.634	**1.895**	**0.936**
0.411	1.821	0.854	0.893	2.451	1.880	0.925	3.328	2.451	1.634	**1.880**	**0.925**
0.426	1.821	0.824	0.893	2.450	1.859	0.911	3.311	2.450	1.634	**1.859**	**0.911**
0.500	1.551	0.702	0.893	2.445	1.756	0.839	3.099	2.445	1.630	**1.756**	**0.839**
0.600	1.292	0.585	0.894	2.281	1.591	0.753	2.839	2.281	1.520	**1.591**	**0.753**
0.700	1.108	0.502	0.894	2.116	1.427	0.667	2.578	2.116	1.411	**1.427**	**0.667**
0.750	1.034	0.468	0.894	2.034	1.344	0.624	2.448	2.034	1.356	**1.344**	**0.624**
0.800	0.969	0.439	0.894	1.975	1.289	0.596	2.336	1.975	1.317	**1.289**	**0.596**
0.900	0.862	0.390	0.894	1.858	1.178	0.540	2.114	1.858	1.239	**1.178**	**0.540**
1.000	0.775	0.351	0.894	1.742	1.067	0.483	1.891	1.742	1.161	**1.067**	**0.483**
2.000	0.388	0.176	0.894	0.940	0.506	0.226	0.796	0.796	0.531	**0.506**	**0.226**
3.000	0.258	0.117	0.894	0.607	0.320	0.143	0.442	0.475	0.316	**0.320**	**0.143**
4.000	0.194	0.088	0.894	0.425	0.228	0.103	0.275	0.356	0.237	**0.228**	**0.103**
5.000	0.155	0.070	0.894	0.323	0.177	0.079	0.197	0.285	0.190	**0.177**	**0.079**

(DJB 04/28/20; Based Upon ASCE 41-17)

TABLE 2

ASCE 41-17: SITE-SPECIFIC SEISMIC REPONSE WORKSHEET - EXISTING BUILDINGS

EXPLANATION: NOTES AND REFERENCES

INPUT BLUE ONLY - BLACK/RED CALCULATED

Column Descriptions

01) Periods including To and Ts calculated from Section 11.4.6 (ASCE 7-16)

02) OSHPD, Seismic Design Maps Web Application - General Horizontal Response Spectrum (GHRS): $S_{BSE\text{-}2N}$: per ASCE 41-17, Section 2.4.1.7.1

03) OSHPD, Seismic Design Maps Web Application - General Horizontal Response Spectrum (GHRS): $S_{BSE\text{-}1N}$: per ASCE 41-17, Section 2.4.1.7.1

04) Risk Coefficient, C_R, for 0.2s and 1.0s periods (ASCE 7-16, Section 21.2.1.1); from OSHPD web application

05) USGS UHT, Probabilistic MCE_R Spectrum: Product of 2% in 50yr Spectrum and Risk Coefficient (ASCE 7-16, Section 21.2.1.1); See Column 7 of Table 2

06) USGS Unified Hazard Tool (UHT), 5% in 50 years Probabilistic Spectrum; scaled w/ MRC factors per Huang et al (2008); per ASCE 41-17, Section 21.2.1.1

06) USGS Unified Hazard Tool (UHT), 2% in 50 years Probabilistic Spectrum; scaled w/ MRC factors per Huang et al (2008); per ASCE 7-16, Section 21.2.1.1

08) PEER Ground Motion Database, 84th Percentile Deterministic Spectrum; controlling fault source (ASCE 7-16; Section 21.2.2)

09) Site Specific BSE-2N Response Spectrum (ASCE 41-17; Section 2.4.1.1); See Column 11 of Table 2

10) Site Specific BSE-1N Response Spectrum (ASCE 41-17; Section 2.4.1.2); See Column 12 of Table 2

11) Site Specific BSE-2E Response Spectrum (ASCE 41-17; Section 2.4.1.3); Lesser of Columns 6 and 9

12) Site Specific BSE-1E Response Spectrum (ASCE 41-13; Section 2.4.1.4); Lesser of Columns 7 and 10

T_L = Figure 22-12 ASCE 7-16 (typically 8 sec Southern California)

**Code-based values obtained from OSHPD Web Application: https://seismicmaps.org/

APPENDIX A

APPENDIX A

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling three exploratory borings. The borings were advanced to depths ranging from 61 to 81½ feet below the existing ground surface. The locations of the explorations are shown on the Site Plan, Figure 2.

The exploratory borings were drilled using truck-mounted hollow-stem auger drill equipment. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D3550). The brass-rings have an inside diameter of 2.42 inches. The ring samples were driven into the soil by a 140-pound hammer dropping 30 inches. The number of blows needed to drive the sampler into the soil was recorded as the penetration resistance.

At selected locations, disturbed samples were obtained using a split-spoon sampler by means of the Standard Penetration Test (SPT, ASTM D6066). The spoon sampler was driven into the soil by a 140-pound hammer dropping 30 inches, employing the “free-fall” hammer described above. After an initial seating drive of 6 inches, the number of blows needed to drive the sampler into the soil a depth of 12 inches was recorded as the penetration resistance. These values are the raw uncorrected blow counts.

The field explorations for the investigation were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings are presented in Figures A-1 to A-3 in this appendix.

The boring and well locations were laid out in the field by measuring from existing site features. Upon completion, the boreholes were backfilled with a cement/bentonite slurry in accordance with the Los Angeles County Well Permit requirements and patched with concrete. The soil cuttings were placed in 55-gallon drums and disposed of off-site. Ground surface elevations at the exploration locations were estimated from Google Earth and should be considered approximate.

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
			B	0	4-Inch AC over 3-Inch BASE Fill: **SILTY SAND (SM)** brown, moist, loose, with gravel (2-inch diameter)	
18.4	99	13	D	5	**SILTY CLAY (CL)** dark brown, moist to very moist, firm to stiff, trace porosity	320 315
18.1	104	19	D	10	Natural: **CLAYEY SILT (ML)** dark brown, moist, stiff	
11.3		11	S		**SILTY SAND (SM)** dark grey, moist, loose, fine-grained, trace clay	
4.9					**SAND (SP)** light grey, slightly moist, fine-grained	310
2.4	113	62	D	15	@ 15 feet, dense	
1.7	110	75	D		@ 17.5 feet, yellowish brown, dry to slightly moist, dense to very dense, fine to medium-grained	305
2.3	113	50/6"	D	20	@ 20 feet, slightly moist, very dense, trace gravel	
2.4	112					
5.5	110	50/5"	D		@ 22.5 feet, slightly moist, with gravel (3-inch diameter)	300
24.8		70	D	25	**CLAYEY SILT (ML)** dark brown, wet, hard	295
16.3		27	S	30	@ 30 feet, very moist, very stiff	290
16.1	112	85/11"	D	35	Bedrock: Puente Formation- **SILTSTONE** grey, very moist, hard, slight cementation, interbedded with gray, fine grained sandstone	
					SANDSTONE grey, moist, dense, fine-grained	285

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 9-22-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 54

GPI

PROJECT NO.: 3080.22I

LAC USC OLD PHARMACY

LOG OF BORING NO. B-1

FIGURE A-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
9.6		37	S	40	**SANDSTONE** grey, moist, dense, fine-grained	
7.3						280
8.2	104	50/6"	D	45	@ 45 feet, very dense	
					@ 47.5 feet, Interbedded **SANDSTONE AND CLAYSTONE** grey, wet, dense and hard	275
24.4		45	S	50		
						270
10.3	126	50/5"	D	55	@ 55 feet, moist, very dense	
					CLAYSTONE dark grey, slightly moist, hard, with thin layers of sandstone, (wet)	265
22.5		50/5"	S	60	@ 60 feet, very moist	
						260
				65		
					SANDSTONE grey, very moist, very dense, medium to coarse-grained	255
14.2		50/6"	S	70	@ 70 feet, with cementation	
						250
				75		
						245

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 9-22-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 54

GPI

PROJECT NO.: 3080.22I

LAC USC OLD PHARMACY

LOG OF BORING NO. B-1

FIGURE A-1

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	13.0		50/6"	S	80	**SANDSTONE** grey, moist to very moist, very dense, medium to coarse-grained	
						Total Depth 81.5 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 9-22-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 54

GPI

PROJECT NO.: 3080.22I

LAC USC OLD PHARMACY

LOG OF BORING NO. B-1

FIGURE A-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
			B	0	5-Inch AC	305
					Fill: **SILTY CLAY (CL)** dark brown, very moist, firm to stiff, with gravel (2-inch diameter), with brick and concrete fragments	
					SANDY SILT (ML) olive brown, wet, stiff	
21.8	102	17	D	5		300
					SILTY CLAY (CL) very dark brown, very moist, stiff, trace gravel, with glass fragments	
24.6	97	20	D	10		295
23.0	98	21	D			
23.5	98	10	D	15	Natural: **SANDY CLAY (CL)** dark brown, very moist, firm, with caliche	290
27.6		9	S		@ 17.5 feet, wet, stiff	
29.0	92	17	D	20		285
21.3		7	S		@ 22.5 feet, very moist, firm	
13.6	114	82	D	25	Bedrock: Puente Formation- **SANDSTONE** grey brown, very moist, very dense	280
13.2		65	S	30		275
20.9	106	50/6"	D	35	@ 35 feet, grey, wet, with gravel (<1-inch diameter)	270

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 9-23-22
EQUIPMENT USED: 8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft): 26

GPI

PROJECT NO.: 3080.22I
LAC USC OLD PHARMACY

LOG OF BORING NO. B-2

FIGURE A-2

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
12.2		50/5"	S	40	**CONGLOMERATIC SANDSTONE** grey, moist, very dense, with gravel (2-inch diameter)	265
12.6	131	50/6"	D	45	@ 45 feet, grey, moist, very dense, with claystone interbeds	260
24.2 15.2		50/6"	S	50	@ 50 feet, very moist to wet	255
21.7	101	95/9"	D	55	@ 55 feet, wet	250
8.5		50/6"	S	60	@ 60 feet, moist, steep bedding, sandstone/pebble conglomerate contact	245
					Total Depth 61.5 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 9-23-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 26

GPI

PROJECT NO.: 3080.22I

LAC USC OLD PHARMACY

LOG OF BORING NO. B-2

FIGURE A-2

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
			B	0	6-Inch AC over 4-Inch BASE	
					Fill: **SILTY SAND (SM)** brown, moist, loose to medium dense, with gravel	305
8.3	114	30	D	5	**CLAYEY SAND (SC)** dark brown, moist, medium dense, trace gravel	300
					@ 8 feet, with wood fragments	
13.2	117	43	D	10	@ 10 feet, very moist, with concrete, mortar, and brick fragments	295
19.6		8	S		**SANDY CLAY (CL)** dark brown, very moist, firm to stiff @ 12 feet, with glass fragments	
21.0	98	28	D	15	@ 15 feet, very stiff @ 16 feet, with brick fragments	290
13.1	108	49	D			
27.7		25	S	20	Bedrock: Puente Formation- **SILTSTONE** dark grey, wet, hard, hydrocarbon odor, with interbedded sandstone and claystone	285
7.3	118	48	D		**SANDSTONE** olive brown, slightly moist to moist, dense	
20.4	105				@ 23 feet, wet	
20.1		27	S	25	@ 25 feet, medium dense	280
6.5	115	89	D	30	@ 30 feet, grey, slightly moist to moist, very dense, coarse-grained	275
24.3	98				@ 31 feet, wet	
21.8		51	S	35	@ 35 feet, dense to very dense	270

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 9-23-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 35

GPI

PROJECT NO.: 3080.22I

LAC USC OLD PHARMACY

LOG OF BORING NO. B-3

FIGURE A-3

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
19.0	105	92/7"	D	40	**SANDSTONE** grey, wet, very dense	265
12.1		50/5"	S	45	@ 45 feet, moist	260
15.1	116	50/3"	S	50	@ 50 feet, very moist	255
12.0		50/6"	S	55	**CONGLOMERATIC SANDSTONE** grey, moist, very dense	250
18.4	113	50/5"	D	60	@ 60 feet, wet	245
					Total Depth 61 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 9-23-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 35

GPI

PROJECT NO.: 3080.22I

LAC USC OLD PHARMACY

LOG OF BORING NO. B-3

FIGURE A-3

APPENDIX B

APPENDIX B

LABORATORY TESTS

INTRODUCTION

Representative undisturbed soil samples and bulk samples were carefully packaged in the field and sealed to prevent moisture loss. The samples were then transported to our Cypress office for examination and testing assignments. Laboratory tests were performed on selected representative samples as an aid in classifying the soils and to evaluate the physical properties of the soils affecting foundation design and construction procedures. Detailed descriptions of the laboratory tests are presented below under the appropriate test headings. Test results are presented in the figures that follow.

MOISTURE CONTENT AND DRY DENSITY

Moisture content and dry density were determined from a number of the ring samples. The samples were first trimmed to obtain volume and wet weight and then were dried in accordance with ASTM D2216. After drying, the weight of each sample was measured, and moisture content and dry density were calculated. Moisture content and dry density values are presented on the boring logs in Appendix A.

ATTERBERG LIMITS

Liquid and plastic limits were determined for select samples in accordance with ASTM D4318. The results of the Atterberg Limits tests are presented in Figure B-1.

GRAIN SIZE DISTRIBUTION

A selected soil sample was dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. That portion of the material retained on the No. 200 sieve was oven-dried and weighed to determine the percentage of the material passing the No. 200 sieve. The percentage passing the No. 200 sieve is presented below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-1	15	Sand (SP)	4

DIRECT SHEAR

Direct shear tests were performed on undisturbed and remolded bulk samples in accordance with ASTM D3080. The bulk samples were remolded to approximately 90 percent of maximum density (ASTM D1557). The samples were placed in the shear machine, and a normal load comparable to the in-situ overburden stress was applied. The samples were inundated, allowed to consolidate, and then were sheared to failure. The

tests were repeated on additional test specimens under increased normal loads. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear tests are presented in Figures B-2 to B-5.

CONSOLIDATION

One-dimensional consolidation testing was performed on selected undisturbed samples in accordance with ASTM D2435. After trimming the ends, the samples were placed in the consolidometer and loaded to either 0.4 or 0.5 ksf. Thereafter, the samples were incrementally loaded to a maximum load of either 25.6 or 32.0 ksf. The samples were inundated at either 2.0 or 3.2 ksf. Sample deformation was measured to 0.0001 inch. Rebound behavior was investigated by unloading the samples back to 0.4 or 0.5 ksf. Results of the consolidation tests, in the form of percent consolidation versus log pressure, are presented in Figures B-6 to B-8. The amount of collapse upon inundation is shown below as percent compression of the sample.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	TOTAL COMPRESSION (%)		
			BEFORE SATURATION	AFTER SATURATION	PERCENT COLLAPSE
B-2	10	Silty Clay (CL)	2.04	2.04	0.0
B-2	15	Sandy Clay (CL)	2.13	2.12	0.0
B-3	10	Clayey Sand (SC)	2.43	2.66	0.2

COMPACTION TEST

A maximum dry density/optimum moisture test was performed in accordance with ASTM D1557 on a representative bulk sample of the site soils. The test results are as follows:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)
B-2	0 – 5	Silty Clay (CL)	125	10.0

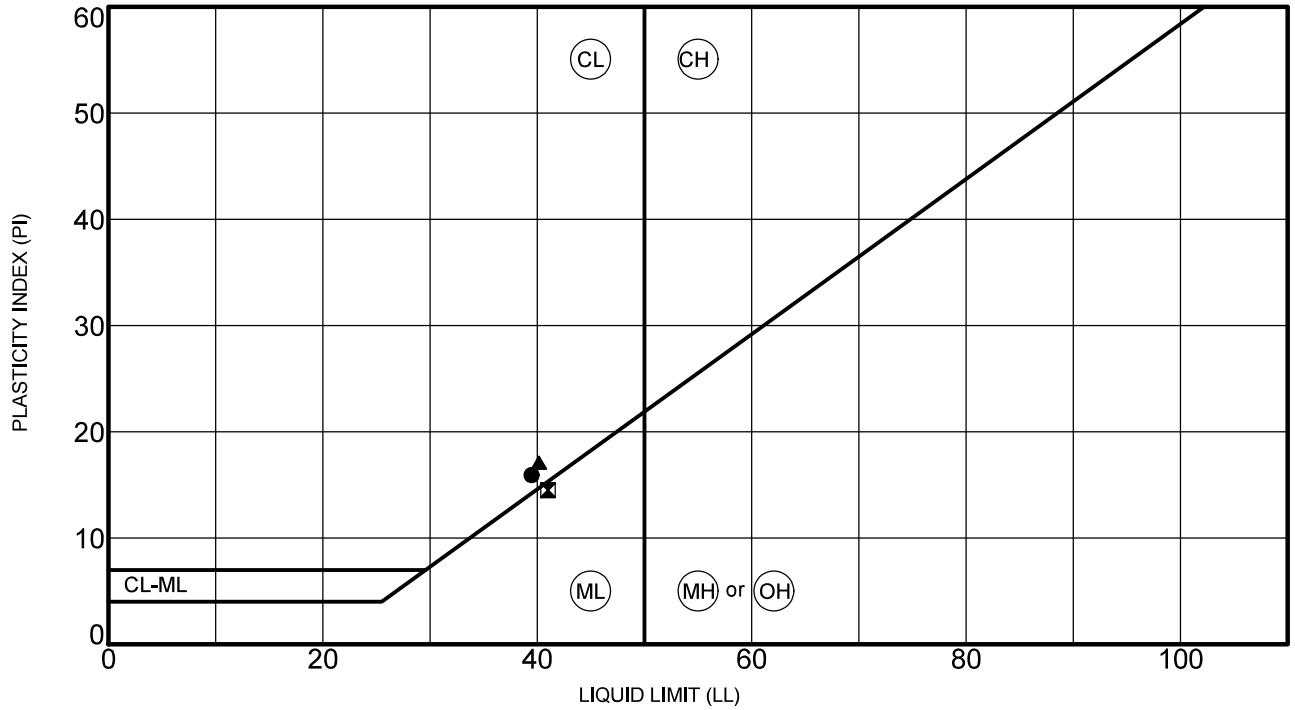
EXPANSION INDEX

Expansion index tests were performed on select bulk samples. The tests were performed in accordance with ASTM D4829, to assess the expansion potential of on-site soils. The results of the test are summarized below:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX	EXPANSION POTENTIAL
B-1	0 – 5	Silty Sand (SM)	11	Low
B-2	0 – 5	Silty Clay (CL)	23	Low

CORROSIVITY

Soil corrosivity testing was performed by HDR a soil sample provided by GPI. The test results are summarized in Table 1 of this Appendix.



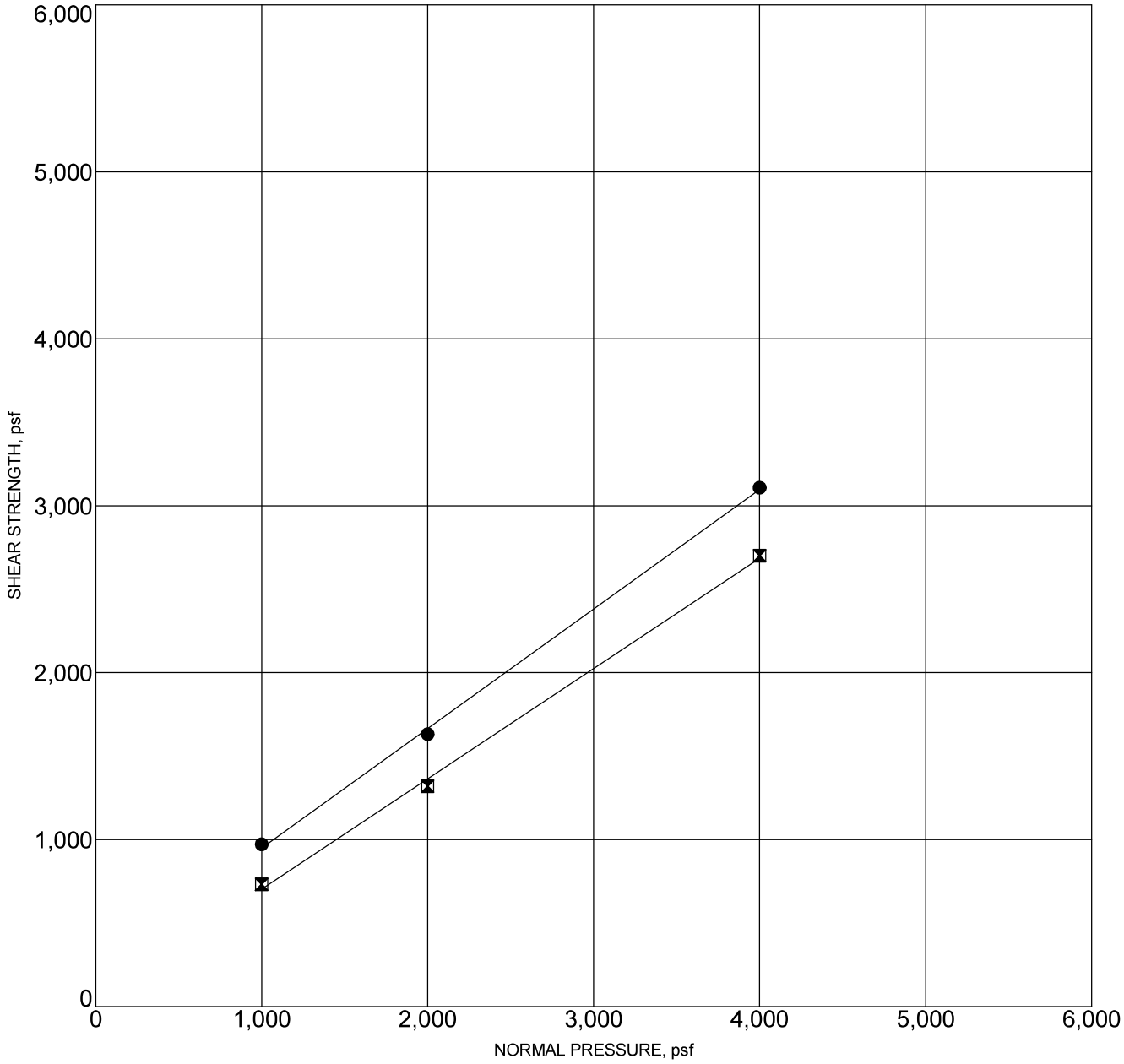
	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
●	B-1	5.0	39	24	16		SILTY CLAY (CL)
⊠	B-1	30.0	41	27	15		CLAYEY SILT (ML)
▲	B-3	15.0	40	23	17		SANDY CLAY (CL)

PROJECT: LAC USC OLD PHARMACY PROJECT NO. 3080.22I

GPI

ATTERBERG LIMITS TEST RESULTS

FIGURE B-1



● **PEAK STRENGTH**
Friction Angle= 36 degrees
Cohesion= 234 psf

⧓ **ULTIMATE STRENGTH**
Friction Angle= 33 degrees
Cohesion= 42 psf

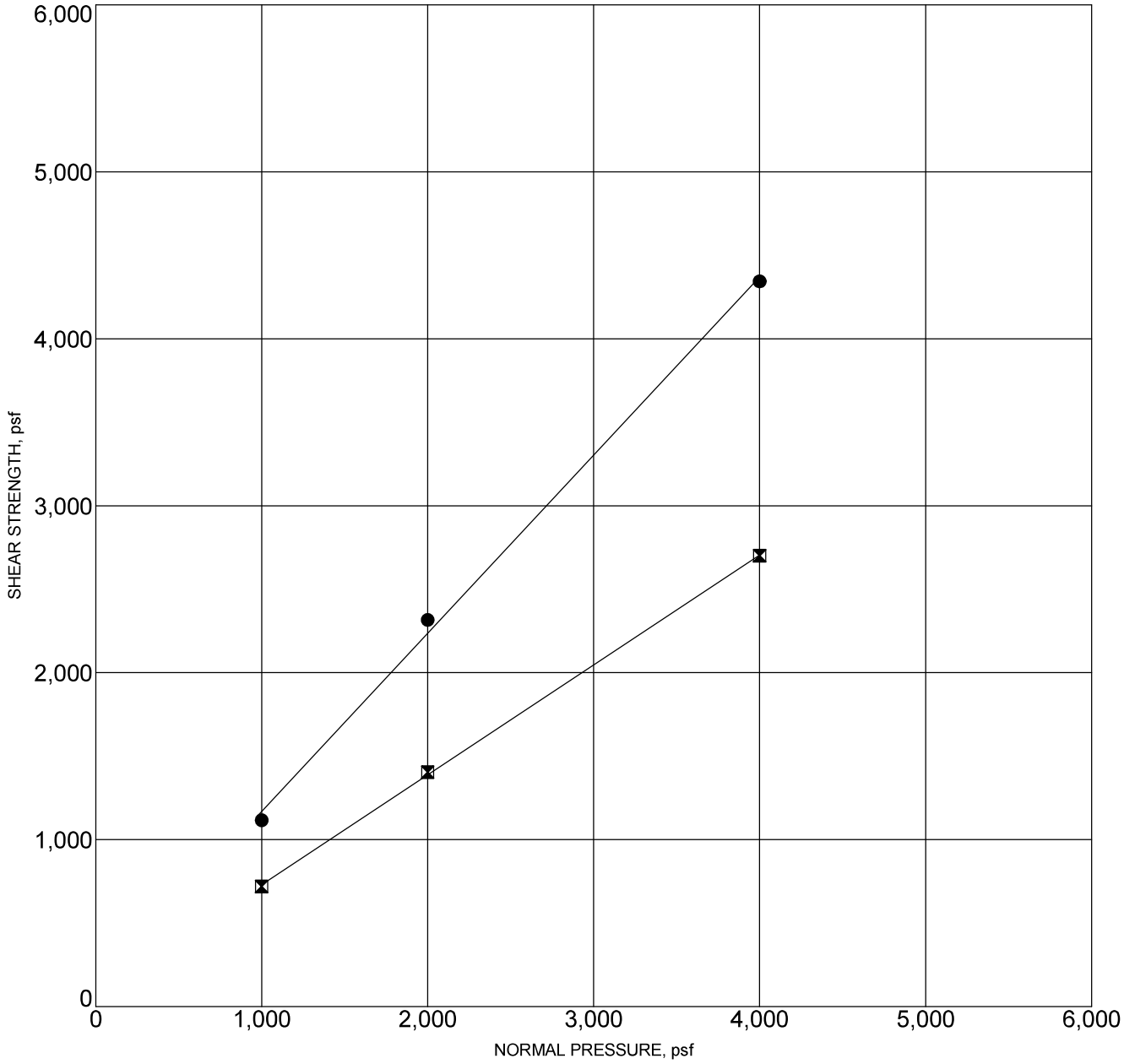
Sample Location		Classification	DD,pcf	MC,%
B-1	10.0	CLAYEY SILT (ML)	104	18.1

PROJECT: LAC USC OLD PHARMACY PROJECT NO.3080.22I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-2



● **PEAK STRENGTH**
Friction Angle= 47 degrees
Cohesion= 102 psf

⧓ **ULTIMATE STRENGTH**
Friction Angle= 33 degrees
Cohesion= 72 psf

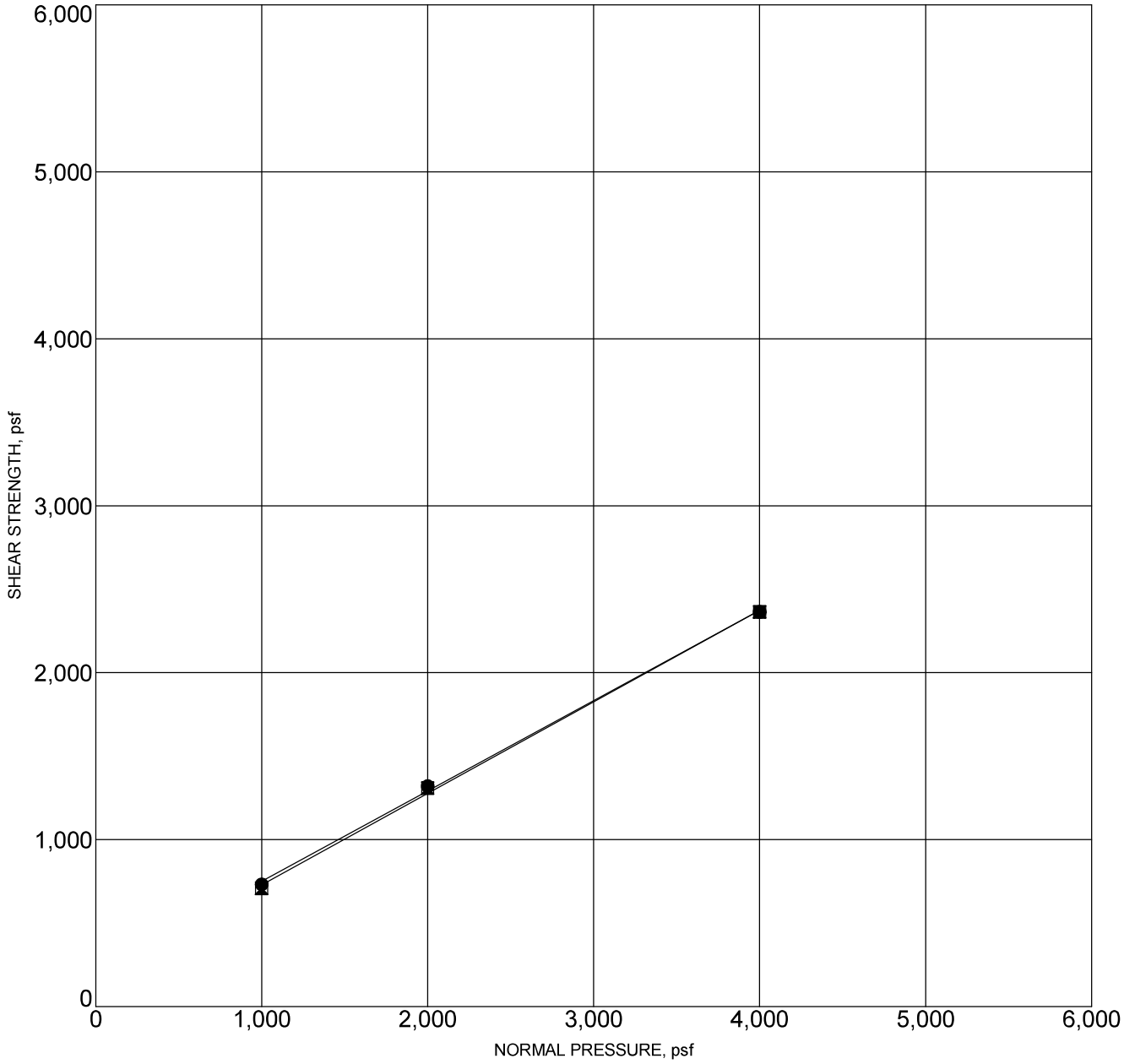
Sample Location		Classification	DD,pcf	MC,%
B-1	17.5	SAND (SP)	110.5	1.7

PROJECT: LAC USC OLD PHARMACY PROJECT NO.3080.22I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-3



● **PEAK STRENGTH**
Friction Angle= 28 degrees
Cohesion= 210 psf

⧓ **ULTIMATE STRENGTH**
Friction Angle= 29 degrees
Cohesion= 180 psf

Note: Samples remolded to 90% of maximum dry density

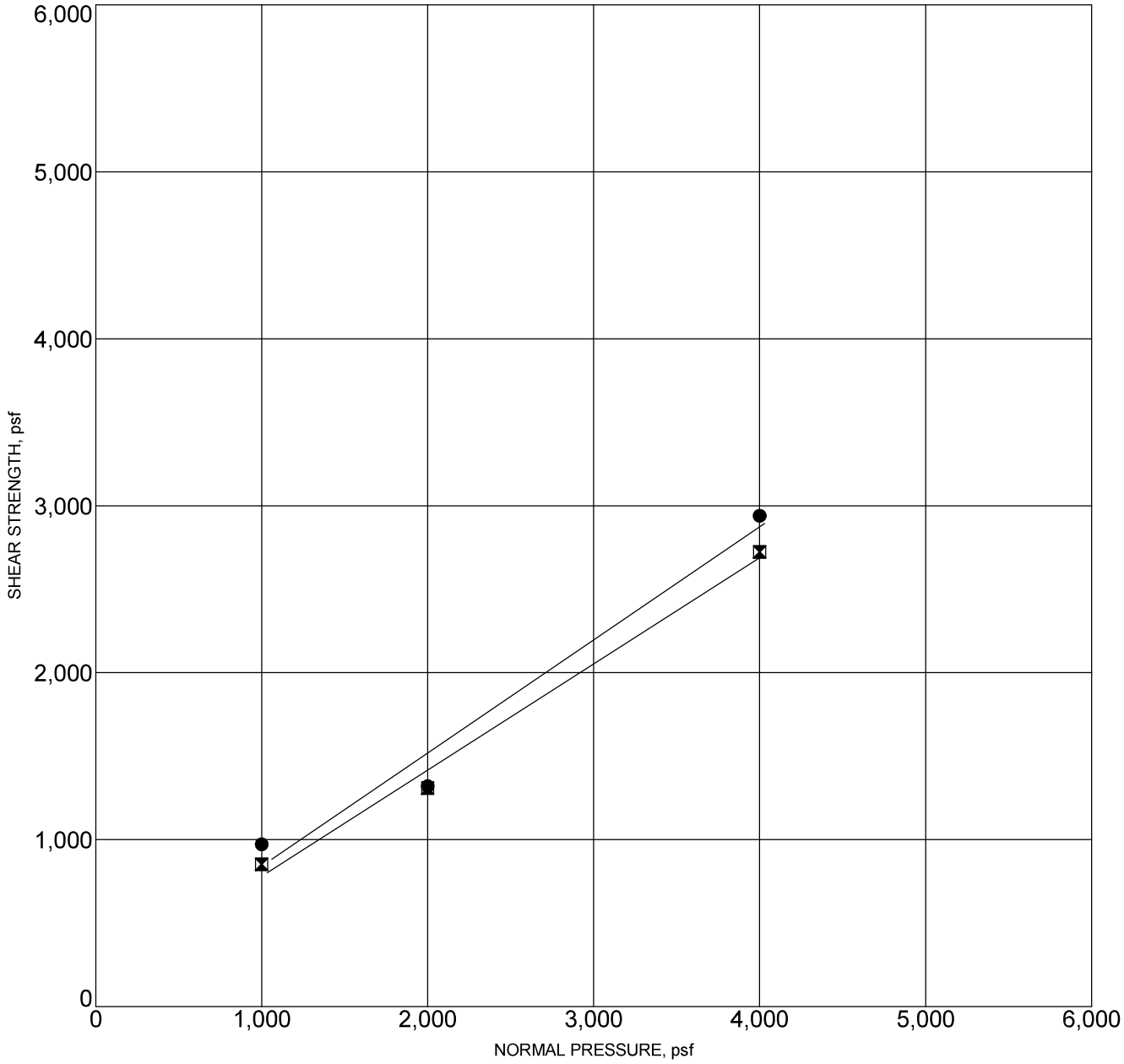
Sample Location		Classification	DD,pcf	MC,%
B-2	0-5	SILTY CLAY (CL)	113	10.0

PROJECT: LAC USC OLD PHARMACY PROJECT NO.3080.22I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-4



● **PEAK STRENGTH**
Friction Angle= 34 degrees
Cohesion= 162 psf

⧓ **ULTIMATE STRENGTH**
Friction Angle= 32 degrees
Cohesion= 144 psf

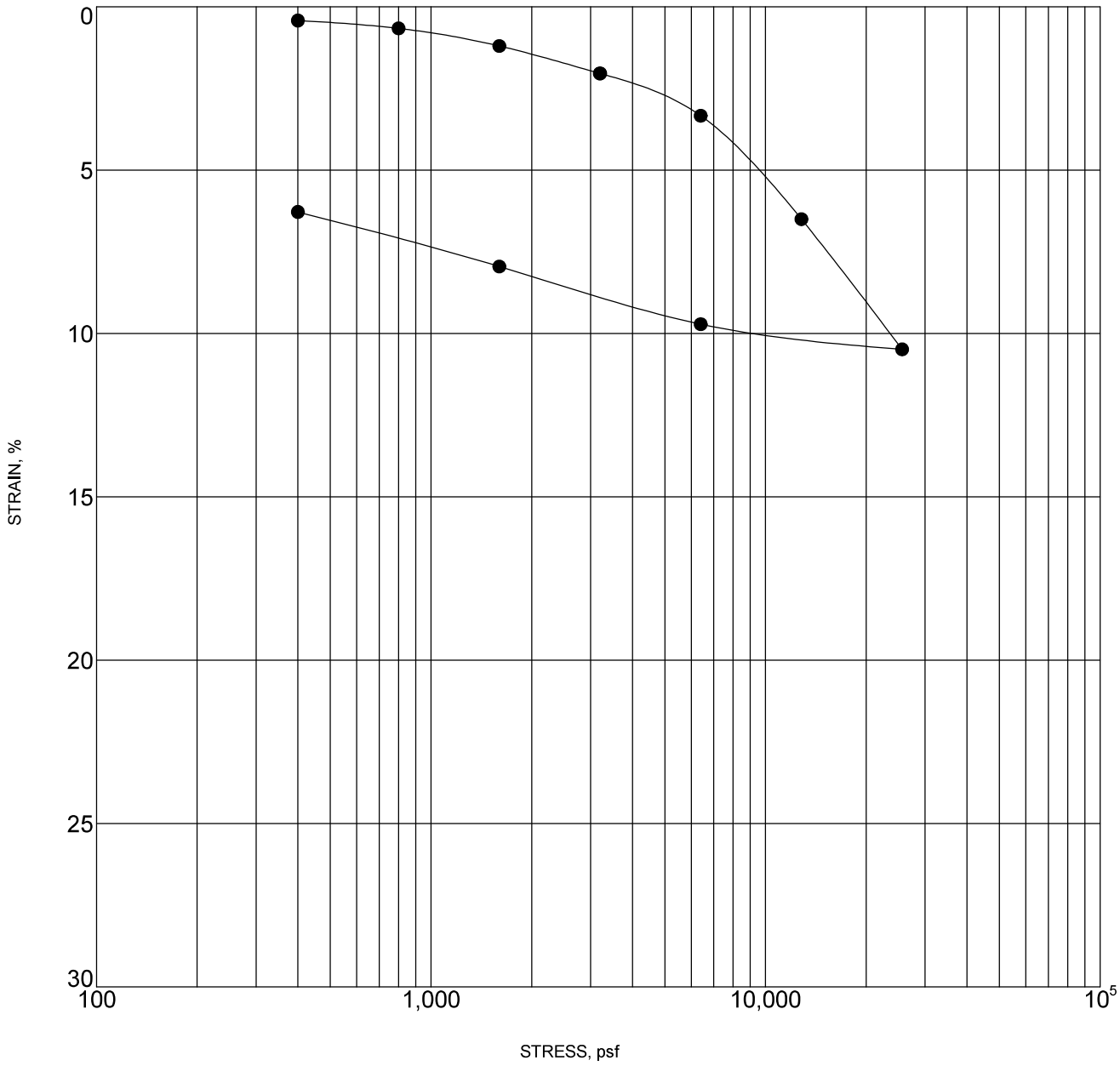
Sample Location		Classification	DD,pcf	MC,%
B-3	5.0	CLAYEY SAND (SC)	114	8.3

PROJECT: LAC USC OLD PHARMACY PROJECT NO.3080.22I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-5



Sample inundated at 3200 psf

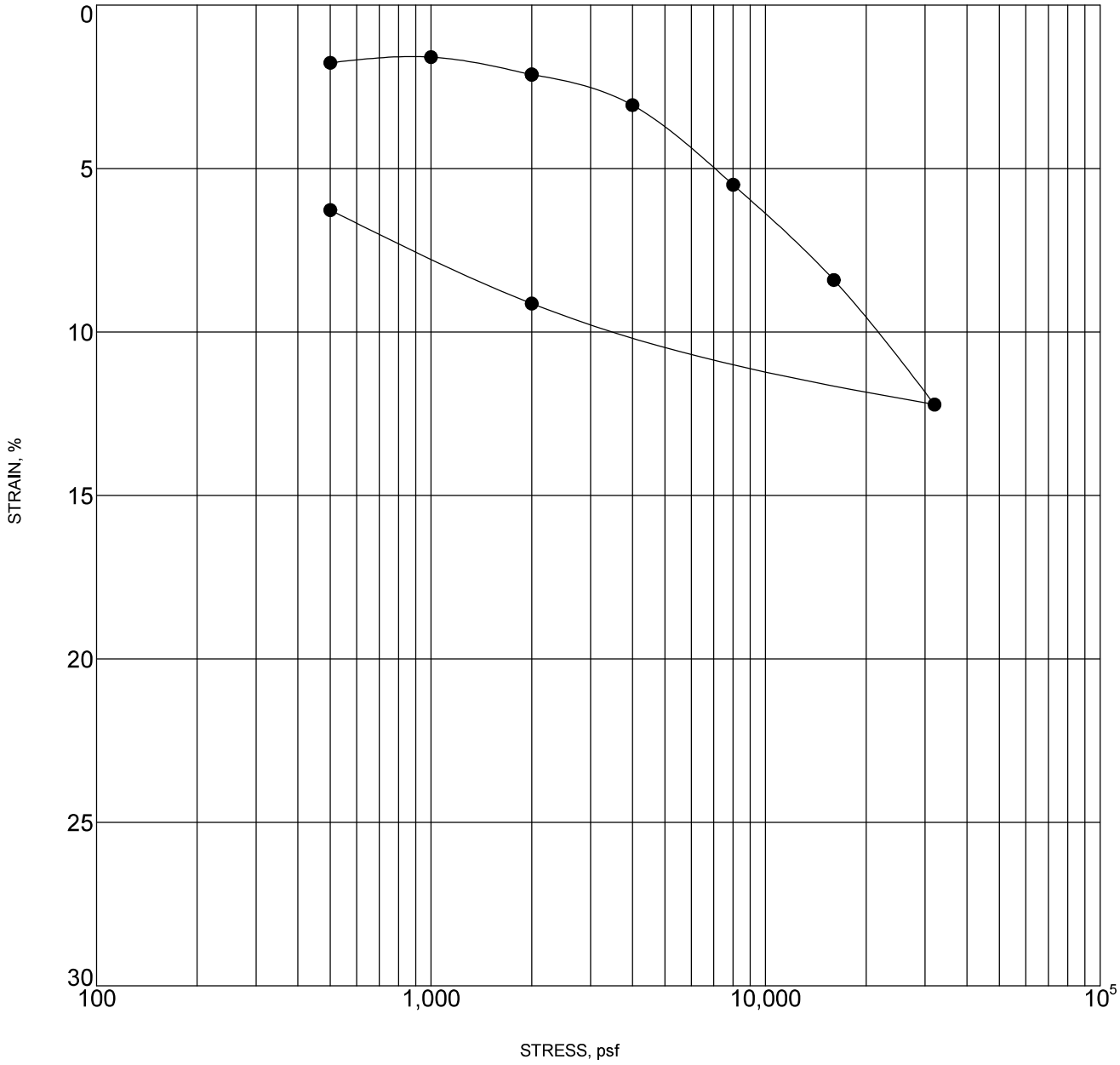
	Sample Location		Classification	DD,pcf	MC,%
●	B-2	10.0	SILTY CLAY (CL)	97	24.6

PROJECT: LAC USC PHARMACY PROJECT NO.: 3080.22I

GPI

CONSOLIDATION TEST RESULTS

FIGURE B-6



Sample inundated at 2000 psf

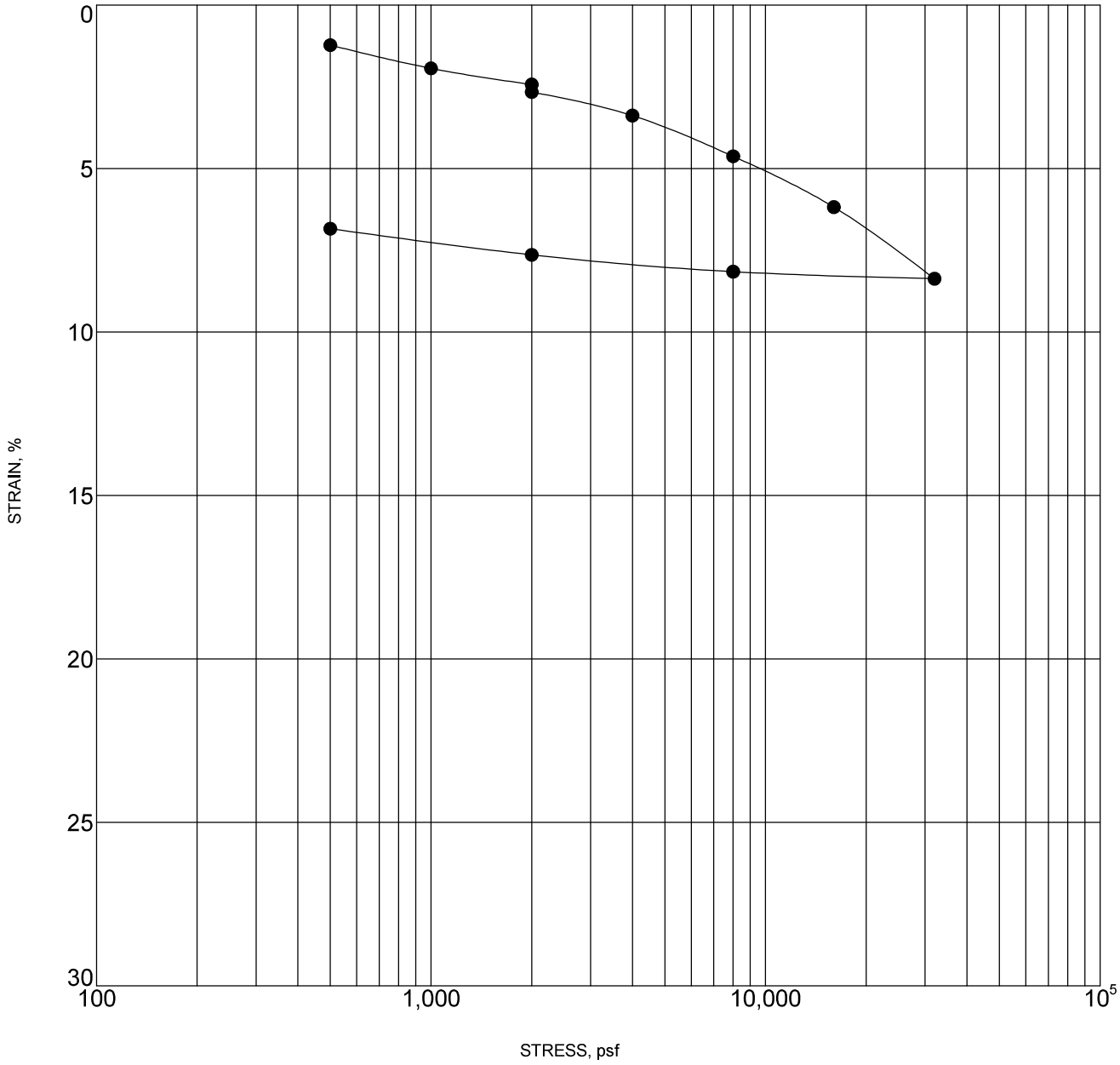
	Sample Location		Classification	DD,pcf	MC,%
●	B-2	15.0	SANDY CLAY (CL)	98	23.5

PROJECT: LAC USC PHARMACY PROJECT NO.: 3080.22I

GPI

CONSOLIDATION TEST RESULTS

FIGURE B-7



Sample inundated at 2000 psf

Sample Location		Classification	DD,pcf	MC,%
● B-3	10.0	CLAYEY SAND (SC)	117	13.2

PROJECT: LAC USC PHARMACY PROJECT NO.: 3080.22I

GPI

CONSOLIDATION TEST RESULTS

FIGURE B-8

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
LAC + USC Old Pharmacy
Your #3080.22I, HDR Lab #22-0938LAB
5-Oct-22

Sample ID			B-1 @ 0-5'	B-2 @ 0-5'
Resistivity		**Units**		
as-received		ohm-cm	8,000	8,000
saturated		ohm-cm	1,400	1,320
pH			7.7	7.3
Electrical Conductivity		mS/cm	0.20	0.27
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	67	65
magnesium	Mg^{2+}	mg/kg	26	27
sodium	Na^{1+}	mg/kg	55	155
potassium	K^{1+}	mg/kg	4.1	4.5
ammonium	NH_4^{1+}	mg/kg	ND	ND
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	317	323
fluoride	F^{1-}	mg/kg	3.4	33
chloride	Cl^{1-}	mg/kg	4.5	8.4
sulfate	SO_4^{2-}	mg/kg	263	325
nitrate	NO_3^{1-}	mg/kg	1.0	22
phosphate	PO_4^{3-}	mg/kg	1.4	0.7
Other Tests				
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

APPENDIX C

APPENDIX C

GEOLOGIC-SEISMIC HAZARD EVALUATION

INTRODUCTION

This geologic and seismic assessment presents a summary of geologic and seismic conditions at the LAC+USC Old Pharmacy Warehouse located at 1100 N. Mission Road in Los Angeles, California. The purpose of this assessment was to identify and evaluate geologic issues, which are likely to be factors with respect to the geotechnical and structural assessment of the existing building. In order to accomplish this objective, the following scope of services was performed:

- Research and review of available published and unpublished geologic literature and maps pertaining to the site and vicinity (see References), as well as subsurface exploration data from our recent investigation.

- Geologic analysis of the reviewed information.

- Preparation of this assessment report, which includes a summary of the researched information and a discussion of the possible geologic-seismic hazards that may affect the subject site and the proposed construction.

SITE CONDITIONS

The Old Pharmacy Warehouse is located within the limits of the existing LAC+USC Medical Center and within the northern portion of the Boyle Heights neighborhood of Los Angeles. More specifically, the Medical Center and the Old Pharmacy Warehouse are located along the southern edge of the low-relief, hillside terrain of the Repetto Hills.

The Old Pharmacy Warehouse currently consists of five stories plus a partial basement as well as roof-top entrances. The footprint of the existing structure is 8,100 square feet (approximately 110 feet by 73 feet). The existing structure was originally constructed in 1916 and received major upgrades in 1953. At this time, we understand only a minor portion of the structure is occupied while the remaining portions of the building are used as storage.

In general, the topography of the Medical Center complex slopes down towards the central portion of the campus with ground surface elevations generally ranging from roughly +325 feet along N. Mission Road, to a low of roughly +300 feet along Marengo Street, back up to roughly +350 feet along N. State Street, and generally continuing to increase in elevation towards the southeast.

The subject Old Pharmacy Warehouse building spans a grade change at the site. On the north side of the subject building, ground surface elevations generally range from approximately +323 to +325 feet. On the south side of the subject building, ground surface elevations generally range from +305 to +306 feet. Because of the grade change, the

bottom floor of the subject building is a partial basement that is one-level below grade on the north side of the building and at-grade on the south side of the building. To the east and west of the subject building, the elevation differentials are separated by retaining walls ranging in approximate height from approximately 15 to 20 feet.

REGIONAL AND LOCAL GEOLOGIC SETTING

Regional Geology

The subject site is located in the northeast portion of a regional geologic structure termed the Los Angeles Basin, a northeast-trending structural basin filled with Tertiary age marine sedimentary rocks mantled by recent and Pleistocene age non-marine alluvial sediments deposited within washes and streams flowing southward from the San Gabriel Mountains and the Repetto Hills to the north of the site.

Regionally, the site is located near the border between two of California's geomorphic provinces, the Transverse ranges to the north and the Peninsular Ranges to the south. The active Raymond Fault, located approximately 4 miles north of the site, forms the boundary of the two geomorphic provinces. The Raymond Fault is part of an east-west trending active fault complex termed the Santa Monica-Hollywood-Raymond Fault System. This fault system generally forms the southern boundary of the Santa Monica and San Gabriel Mountains north of the fault system, and the Los Angeles Basin, where the site is located, south of the fault system.

The Transverse Ranges are characterized by east-west trending mountain ranges, including the Santa Monica and San Gabriel Mountains, which are oriented oblique to the trend of the other major structural trends in California, including the San Andreas Fault, Sierra Nevada Mountains, and other mountain ranges in Southern California, which trend northwesterly.

The Peninsular Ranges are characterized by northwesterly trending active faults and mountain ranges related to the San Andreas and other major fault systems in the province. The province extends from the Los Angeles Basin, where the project is located, southeast to Baja California.

In general, the bedrock geology exposed in the Repetto Hills north of the site indicates folds and faults with a northwest trend. Although, the faults and folds exposed in the hills are not currently zoned as "active," a growing body of evidence suggests that these structures may be active in response to activity on the Elysian Park blind thrust fault. This fault is zoned as active and is thought to be responsible for the Elysian Park Anticline and similar, smaller folds to the northeast.

A few surface faults have been mapped within the Repetto Hills, most of which are unnamed. Many minor faults probably exist within the hills and have resulted as a product of the folding (compressional) processes. No mapped faults project through the site area.

Site Geologic Conditions

The subsurface geologic conditions, as detailed on published geologic maps (Lamar, 1970), indicate the site is initially underlain by surficial, younger and older quaternary-age alluvium generated from floodplain deposits consisting of silts, sands, and gravels. Roughly 300 to 400 feet east of the subject building, still within the limits of the medical campus, are mapped outcroppings of Puente Formation sandstone bedrock materials. The Puente Formation sandstone bedrock (Tpss) is classified as well bedded, medium- to coarse-grained, and light brown to grey. Nearby siltstone outcroppings (Tpsl) are described as well bedded, light brown and light grey.

As encountered in our exploratory borings and prior nearby explorations by others, the subsurface materials at the site generally consisted of undocumented fill soils overlying younger and older alluvial soils and, at depth, Puente Formation bedrock materials. Undocumented fills on the north side of the subject building (approximate Elevation +325 feet) were up to 10 feet in depth and generally consisted of silty sands. The natural materials on the north side of the building consisted of approximately 5 feet of younger alluvial clays and sands and approximately 20 feet of older alluvial clays and sands. The natural soils are further underlain by Puente Formation sandstone and claystone bedrock encountered roughly 35 feet below the existing grade (approximate Elevation +290 feet).

On the south side of the subject building (approximate Elevation +305 feet), we encountered 15 to 18 feet of undocumented fill soils, consisting predominantly of clays with lesser deposits of silts and sands. The undocumented fill soils also contained debris consisting of concrete, lumber, brick, and glass fragments. The undocumented fills are likely old, given the age of the subject building. Underlying the undocumented fills at the southwest corner of the subject building, we encountered approximately 10 feet of younger alluvial soils consisting predominantly of sandy clays. We did not encounter natural alluvial soils (younger or older) between the undocumented fills and Puente Formation bedrock in our exploration located at the southeast corner of the subject building. The Puente Formation sandstone and claystone bedrock was encountered beginning at depths of 18 and 25 feet below existing grades in our explorations located on the south side of the subject building (correlating to approximate Elevations of +287 and +280 feet).

The geologic conditions in the site area are shown on the Regional Geologic Map, Figure C-1, and the Geologic Cross Section, Figure C-2. We should note that the subject site is located approximately 3,000 feet west of the geologic cross section presented in Figure C-2.

Groundwater Conditions

Data published by the State of California indicates that the historical shallowest groundwater depth in the site vicinity to be approximately 20 feet below prevailing site grades. We encountered groundwater at a depth of 55 feet below existing grades in our exploration located on the north side of the subject building (roughly corresponding to Elevation +270 feet). In our explorations located on the south side of the subject building,

we encountered groundwater at depths of 25 to 35 feet below existing grades (roughly corresponding to Elevations of +280 and +270 feet).

Prior nearby explorations by AECOM (2019), located roughly 500 feet east/northeast of the subject site, encountered groundwater between depths of 32 and 38 feet below site grades (roughly corresponding to Elevations +287 to +296 feet). Prior nearby explorations by RMA Group (2018), located roughly 500 feet to the southeast of the subject site, did not encounter groundwater within 45 feet of existing grades (deeper than Elevation +311 feet). Prior nearby explorations by Geotechnologies, Inc. (2019), located roughly 1,700 feet to the southeast did not encounter groundwater within the 40-foot depth explored (deeper than Elevation +325 feet).

Details of the historical groundwater depths in the vicinity of the site are shown on the Historic Groundwater Map, Figure C-3.

TECTONIC SETTING

Regional Fault Systems

The geologic structure of southern California is dominated by northwest trending faults associated with the San Andreas Fault System. Faults such as the Newport-Inglewood, Whittier, Palos Verdes Hills, and San Jacinto are all considered active and are all associated with the San Andreas, which collectively form the boundary between the North American and Pacific tectonic plates. Most of these faults have ruptured the ground surface historically and/or produced significant earthquakes.

Anomalous to the general northwest structural fabric are a series of active west trending reverse or thrust faults. The majority of these occur as north dipping planes projecting along the southern base of the Santa Monica and San Gabriel Mountains in the greater Los Angeles area. The known active thrust faults in the region include the Cucamonga, Sierra Madre, San Fernando, Raymond, Santa Monica, and Hollywood faults.

Concealed Faults

Another category of fault known as "blind thrusts" was recognized as a significant seismic hazard following the 1987 magnitude 6.0 Whittier Narrows Earthquake and then again by the 1994 San Fernando magnitude 6.7 Earthquake. A blind thrust is a deeply buried shallow dipping thrust fault, which does not project to the ground surface. Blind thrusts are capable of generating a major earthquake that may cause uplift in the form of anticlinal hills. Some uplands that surround the Los Angeles Basin, including the Elysian Park and Repetto Hills, are products of blind thrusts. Because blind thrusts do not intersect the ground surface, primary surface fault rupture is considered unlikely. Major portions of the Los Angeles Basin are now believed to be underlain by various blind thrusts ramps. Due to continued north-south convergence (shortening) across the Los Angeles Basin, slippage along these features will generate earthquakes.

At the present time, the potential magnitudes and recurrence intervals of blind thrust produced earthquakes cannot be quantified with confidence due to the fact that many characteristics of these features (including areal extent and Quaternary slip rates) are poorly understood. Nonetheless, the proximity to densely populated urban centers and their history of producing damaging earthquakes clearly demonstrate the risk that blind thrusts pose to large metropolitan areas such as Los Angeles and surrounding cities.

Nearby Seismogenic Sources

We reviewed the 2008 National Seismic Hazard Maps Source Parameters (USGS, 2008) to identify known active faults within a 100 km radius of the project site. The names and distances of the faults lying within approximately 25 kilometers of the project site are provided in the following table (Table 1). We present a map showing the significant regional faults in Figure C-4, Regional Fault Map.

Table 1 – Significant Regional Faults

Fault Name	Approximate Distance* (km)
Elysian Park Blind Thrust	0.6
Raymond	6.8
Hollywood	6.8
Santa Monica – Connected (Alt)	8.6
Puente Hills Blind Thrust (LA)	8.9
Verdugo	9.5
Newport Inglewood – Connected	15.3
Sierra Madre – Connected	16.7
Elsinore	16.8
Puente Hills Blind Thrust (Santa Fe Springs)	18.3
Santa Monica (Alt)	18.5
Clamshell-Sawpit	23.5
Puente Hills (Coyote Hills)	24.2
Sierra Madre (San Fernando)	25.4

* Defined as the closest distance to projection of rupture area along fault trace.

The site does not lie within an Alquist-Priolo Earthquake Fault Zone as designated by the California Geological Survey (CGS, 2017). Surface faults have not been mapped projecting towards or through the site area.

Brief details of some of the faults closest to the subject site are as follows:

Elysian Park Blind Thrust

The north to south structural convergence in the region is a result of deep-seated fault movement along features called “blind thrusts.” These are buried low angle north and some south dipping faults which do not project to the ground surface but cause uplift by folding during major earthquakes. In 1987, the magnitude 5.9 Whittier Narrows Earthquake occurred on a previously unknown blind thrust, which has now been given the name Elysian Park Blind Thrust or Structural Zone. This fault underlies the Elysian Park Hills at

3 km and deepens northward to 10 km of depth. Because of the 1987 event, the fault has been placed into an active category and has been tentatively mapped to underlie a major portion of the eastern Los Angeles Basin and adjacent San Gabriel Valley to the north. Subsequent to this earthquake was the 1994 M6.7 Northridge Earthquake in the San Fernando Valley. This earthquake occurred along a previously unknown similar blind thrust fault. This type of active faulting and resulting earthquake activity are considered relatively common in regions undergoing convergence. The Elysian Park Thrust has a length of 34 km, slip rate of 1.50 mm/yr and is capable of generating a maximum earthquake of M6.7 (Shaw and Suppe, 1996).

Puente Hills Blind Thrust

The Puente Hills Blind Thrust (Shaw, 1999) is a north dipping blind thrust extending from the Santa Fe anticline northward to the Montebello anticline. Movement on the fault is responsible for the 1987 Whittier Narrows earthquake. Research on the earthquake and its aftershocks, as well as fault plane reflections, have resulted in the conclusions that the fault is located between 3 and 7 kilometers below sea level. Data on the slip rate and possible recurrence intervals are still being researched.

Raymond Fault

The Raymond Fault projects roughly east-west from the San Gabriel Mountains to at least the Arroyo Seco in South Pasadena. The fault has a length of about 15 miles. This north dipping thrust fault lies about 6.8 kilometers to the north of the subject site. A well-developed linear scarp up to 100 feet high clearly defines the feature. Fault investigations indicate the fault has moved at least once in the last 10,000 years (Holocene Time) placing it into an active category. Minor earthquakes have occurred within the Raymond Basin to the north and in 1991 a moderate sized event (M5.8) occurred beneath Pasadena presumably on the Raymond Fault though no surface rupture was reported. The Raymond Fault has been assigned a maximum magnitude of 6.5 and a slip rate of 0.5 mm/yr.

Hollywood Fault

The Hollywood Fault is mapped along the foot of the southern flank of the east-west trending Santa Monica Mountains about 6.8 kilometers to the northwest of the site at its closest approach. Mapping of the feature indicates the fault has a length projecting from the coast eastward to the Los Angeles River channel. The fault has been studied by several groups including Dr. Kerry Sieh at CIT (1993). The location of the fault is poorly constrained in the field due to alluvial cover and urban development. The fault is believed to be a high angle, north dipping thrust fault and has been partly responsible for uplift of the Santa Monica Mountains. Carbon dating methods indicate the fault has moved at least once between 8,000 to 17,000 years ago, which places it into a likely active category. No significant historic earthquakes have been associated with the fault. The feature has been assigned a maximum magnitude of 6.4 and a slip rate of 1.0 mm/yr. The fault has been recently reassessed by the State of California. However, the revised locations do not impact the school site.

SEISMIC EXPOSURE

As is the case with most locations in Southern California, the subject site is located in a region that is characterized by moderate to high seismic activity. The project site and vicinity have experienced strong ground shaking due to earthquakes in historic time. The locations of earthquake epicenters with respect to the subject site are shown graphically on Figure C-5, Regional Seismicity.

SECONDARY SEISMIC EFFECTS

General

Secondary effects of seismic activity normally considered as possible hazards to a particular site include several types of ground failure as well as induced flooding. Various types of ground failures, which might occur as a consequence of severe ground shaking of a site, include landsliding, ground subsidence, ground lurching, shallow ground rupture and liquefaction. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsurface soils, and groundwater conditions, in addition to other factors. Based on a review of available published literature, landsliding, ground subsidence, ground lurching, and shallow ground ruptures are considered unlikely at the site.

Various types of seismically induced flooding, which may be considered as potential hazards to a particular site, include flooding due to a tsunami (seismic sea wave), a seiche, or failure of a major water retention structure upstream of the project. The site is located approximately 15 miles inland from the Pacific Ocean at elevations of approximately +305 to +325 feet above mean sea level. The site is also not located within a flood zone as determined by FEMA and the County of Los Angeles.

However, the site does lie in proximity to one open body of water. The lake at Lincoln Park is located approximately 0.62 miles (3,300 feet) to the northeast and sits at an elevation of approximately +345 feet, roughly 20 to 40 feet above the prevailing elevation at the subject site. Historical topographic maps of the site area appear to indicate the subject site is located in proximity to a past drainage channel associated with Lincoln Park Lake.

Due to the distance to the coast and distance to the nearby lake, the probability of flooding due to a tsunami is considered to be nonexistent. Based on the proximity to the nearby Lincoln Park Lake and the site appearing to be located in past drainage channel associated with that lake, the probability of flooding due to seiche is considered to be low.

Liquefaction Considerations

Loosely compacted/deposited granular soils located below the water table can fail through the process of liquefaction during strong earthquake-induced ground shaking. In this process, there is a rapid decrease in shearing resistance of cohesionless soils, caused by a temporary increase in the pore water pressure. Factors known to influence liquefaction potential include soil type and depth, grain size, relative density, ground-water level, degree of saturation, and both intensity and duration of ground shaking.

As a result of liquefaction, a typical building structure may be exposed to several hazards, including liquefaction-induced settlement, foundation bearing failure, and lateral displacement or lateral spreading. The surface manifestation of liquefaction in deeper soil deposits often takes place in the form of sand boils and ground subsidence. Such phenomena often lead to loss of adequate support for building foundations (bearing failures) and cause tilting, excessive movement, and cracking of superstructures. The severity of ground subsidence depends largely on the relative thickness of the surficial non-liquefiable layer compared to the thickness of layers undergoing liquefaction.

According to the published State Seismic Hazard Zones map for the Los Angeles Quadrangle, the subject structure lies just on the edge of and outside an area designated by the State Geologist as a “zone of required investigation” due to the potential for earthquake-induced liquefaction. The limits of the liquefaction zone appear to correspond to a historical drainage channel associated with Lincoln Park Lake located northeast of the subject site. Details of the liquefaction potential in the vicinity of the site are shown on Figure C-6, Seismic Hazard Zones Map. For details and results on our liquefaction and seismic settlement evaluation, refer to Section 4.2.3 in the text of our report.

SUMMARY OF GEOLOGIC CONSTRAINTS

Based on the results of our geotechnical investigation and a review of the information provided in the referenced literature, it is recommended that the following geologic constraints be taken into account during the initial planning stages of the proposed development.

- The subject site is located in a seismically active area of southern California. The type and magnitude of seismic hazards that may affect the site are dependent on both the distance to causative faults and the intensity and duration of the seismic event. The subject site will likely experience strong ground shaking caused by earthquakes on active, regional faults in the future. The proposed project should be designed and constructed in accordance with the seismic design parameters provided in the building code and our final geotechnical investigation report.

- Active faults have not been mapped projecting towards or through the site.

- The subject site is located on the edge of an area designated by the State Geologist as a “zone of required investigation” for liquefaction hazard. As such, the potential for damage due to liquefaction is considered to be low to moderate.

- Based on a review of available published literature, ground subsidence, ground lurching and shallow ground rupture are considered unlikely at the site.

REFERENCES

Barrows, A.G., 1974, A review of the Geology and Earthquake History of the Newport-Inglewood Structural Zone, Southern California, CDMG, Special Report 114.

Bullard, T.F., Letus, W.R., 1993, Quaternary Fold Deformation Associated with Blind Thrust Faulting, Los Angeles Basin, California: Jour. of Geophy. Research Vol 98, No. B5, May 1993.

California Geological Survey, 1998, Seismic Hazard Zone Report for the Los Angeles 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 029.

California Geological Survey, 2017, Earthquake Zones of Required Investigation, Los Angeles Quadrangle, released June 15, 2017.

Conrey, B.L., 1967, Early Pliocene Sedimentary History of the Los Angeles Basin: California Geologic Survey, SR-93.

Dibblee, T.W., 1991, Geologic Map of the Los Angeles Quadrangle, Los Angeles County, California: Dibblee Geological Foundation Map DF-22.

Hart, E. W., 1997 (Revised), Fault Rupture Hazard Zones in California: Calif. Geol. Survey, SP-42.

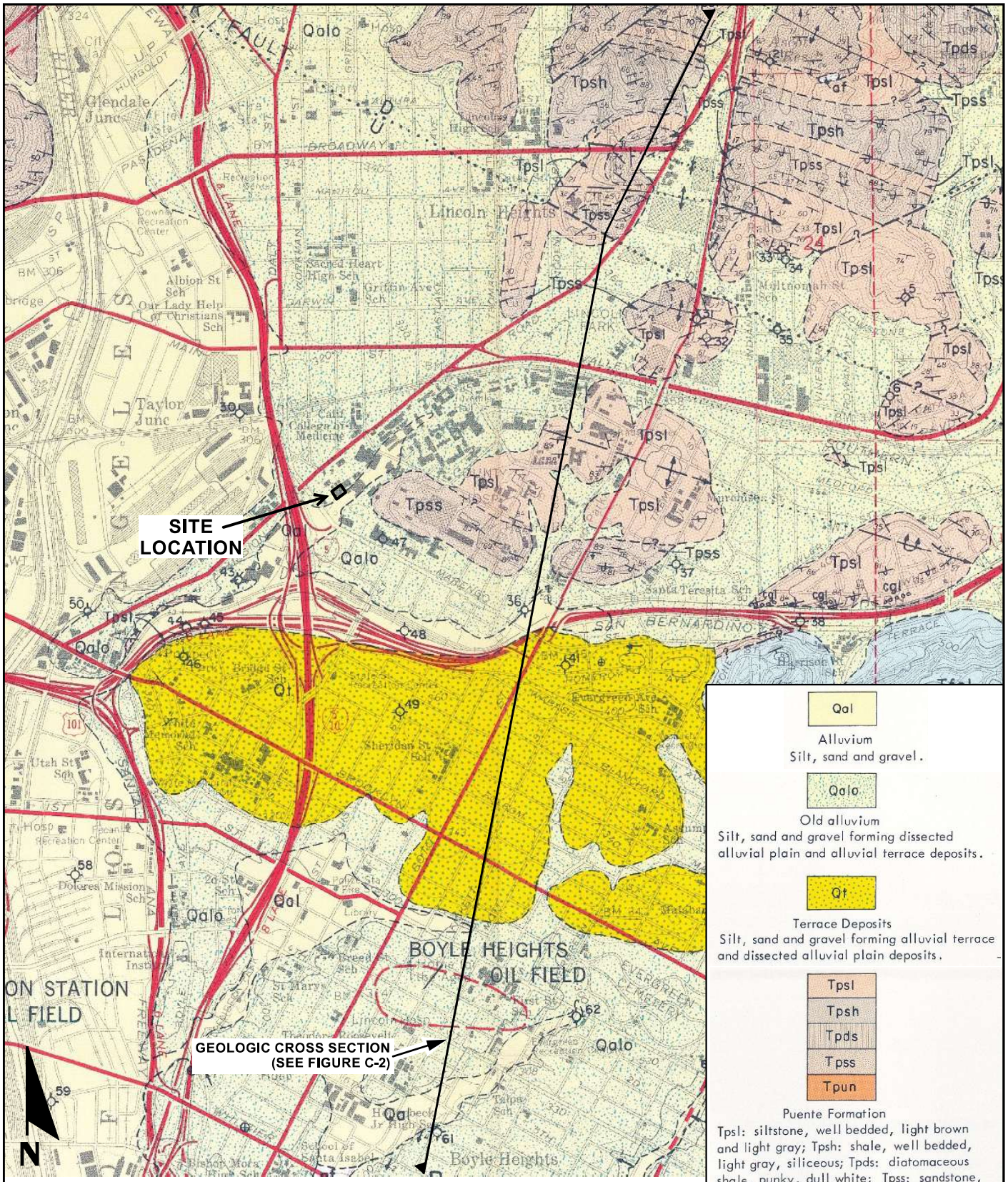
Lamar, D.L., Geology of the Elysian Park - Repetto Hills Area, Los Angeles County, California: Calif. Geol. Survey SR-101.

Shaw, J.H., Suppe, J., 1996, Earthquake Hazards of Active Blind Thrusts under the Central Los Angeles Basin, California, Journal of Geophysical Research, Vol. 101, No. B4, pp 8623-8642, April 1996.

Shaw, J.H., 1999, Seismic Reflection Transect and Geologic Cross Section Across the Central Los Angeles Basin and San Pedro Bay, Annual Report to Southern California Earthquake Center.

USGS, 2008, National Seismic Hazard Maps Source Parameter website, http://geohazards.usgs.gov

Ziony, J.I., 1985, Evaluating Earthquake Hazards in the Los Angeles Region - An Earth Science Perspective: U.S. Geol. Survey, Prof. Paper 1360.



BASE MAP REPRODUCED FROM THE GEOLOGIC MAP OF THE ELYSIAN PARK-REPETTO HILLS AREA, LOS ANGELES COUNTY, CALIFORNIA PREPARED BY LAMAR, D.L.: DATED 1970

GPI GEOTECHNICAL PROFESSIONALS, INC.

LAC-USC OLD PHARMACY BUILDING

GPI PROJECT NO.: 3080.22I | SCALE: 1" = 2000'

REGIONAL GEOLOGIC MAP

FIGURE C-1

B

61
RICHFIELD OIL CORP.
Boyle Comm. 17-1

SITE LOCATION*

ELYSIAN PARK FAULT

HIGHLAND PARK FAULT

1000'
SEA LEVEL
1000'
2000'
3000'
4000'
5000'
6000'
7000'

5868'

MATCHLINE

* Site located between roughly 3,000 feet west of geologic cross section (See Figure C-1)

B'

EXPLANATION

QUATERNARY — PLEISTOCENE
- Qun — Undifferentiated alluvium and terrace deposits
- Qt — Terrace deposits

TERTIARY — PLIOCENE — FERNANDO FORMATION
- Tfcg — Conglomerate
- Tfss — Sandstone
- Tfsl — Siltstone

TERTIARY — UPPER MIOCENE — PUENTE FORMATION
- Tpsl — Siltstone
- Tpsh — Shale
- Tpss — Sandstone
- Tpun — Undifferentiated sedimentary rocks

TERTIARY — MIDDLE MIOCENE — TOPANGA FORMATION
- Ttss — Sandstone
- Ttcg — Conglomerate
- Ttsl — Siltstone
- Ttun — Undifferentiated sedimentary rocks

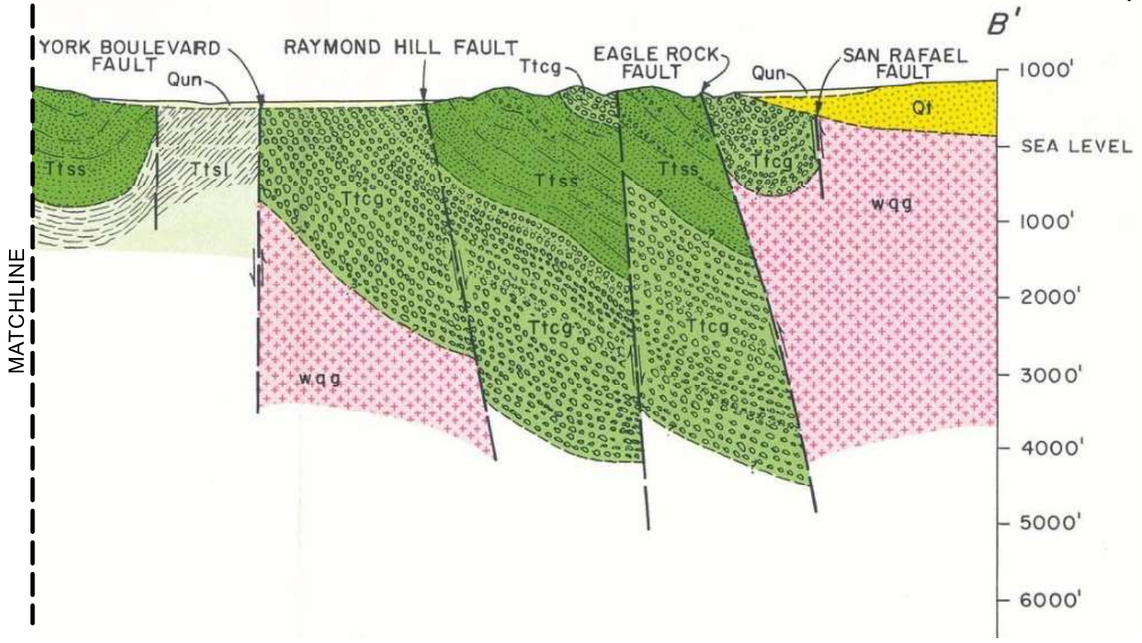
JURASSIC? CRETACEOUS(?)
- wqm, wqg — Wilson Quartz Diorite (wqm: massive; wqg: gneissic)
- Jsm — Santa Monica Slate

A|T — Fault
Arrows indicate relative movement
T is side that moved toward observer;
A is side that moved away

LITHOLOGIC SYMBOLS
- Shale
- Siltstone
- Sandstone
- Conglomerate

Bend in section
See plate 1 for location of lines of section

0 2000 4000 FEET



BASE MAP REPRODUCED FROM GEOLOGIC SECTIONS ACROSS THE ELYSIAN PARK-REPETTO HILLS AREA, LOS ANGELES COUNTY, CALIFORNIA PREPARED BY LAMAR: DATED 1970

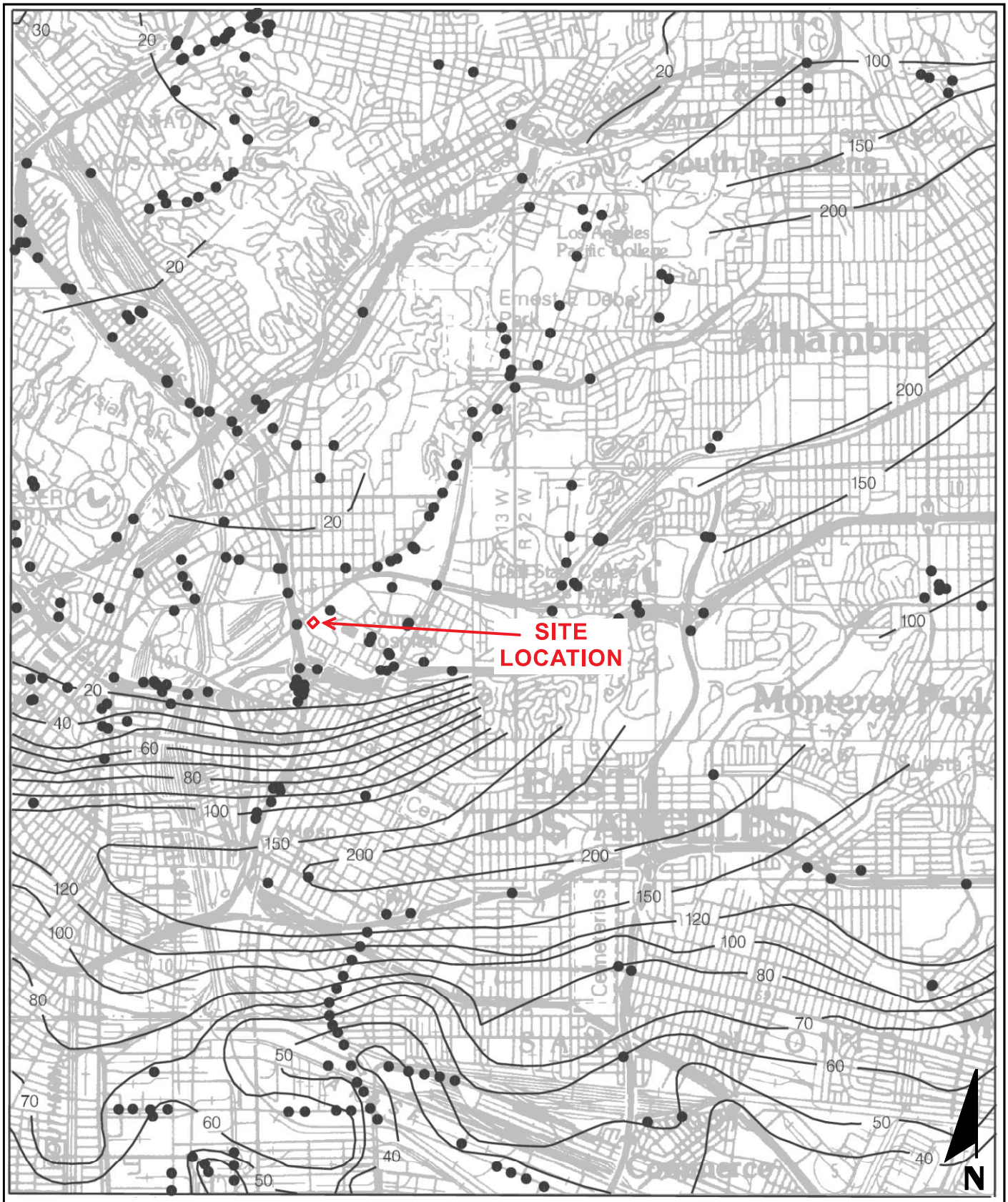
GPI GEOTECHNICAL PROFESSIONALS, INC.

LAC-USC OLD PHARMACY BUILDING

GPI PROJECT NO.: 3080.22I | SCALE: 1" = 2000'

GEOLOGIC CROSS SECTION

FIGURE C-2



BASE MAP REPRODUCED FROM HISTORICAL GROUNDWATER MAP (PLATE 1.2) FROM SEISMIC HAZARD ZONE REPORT 029 (1998)

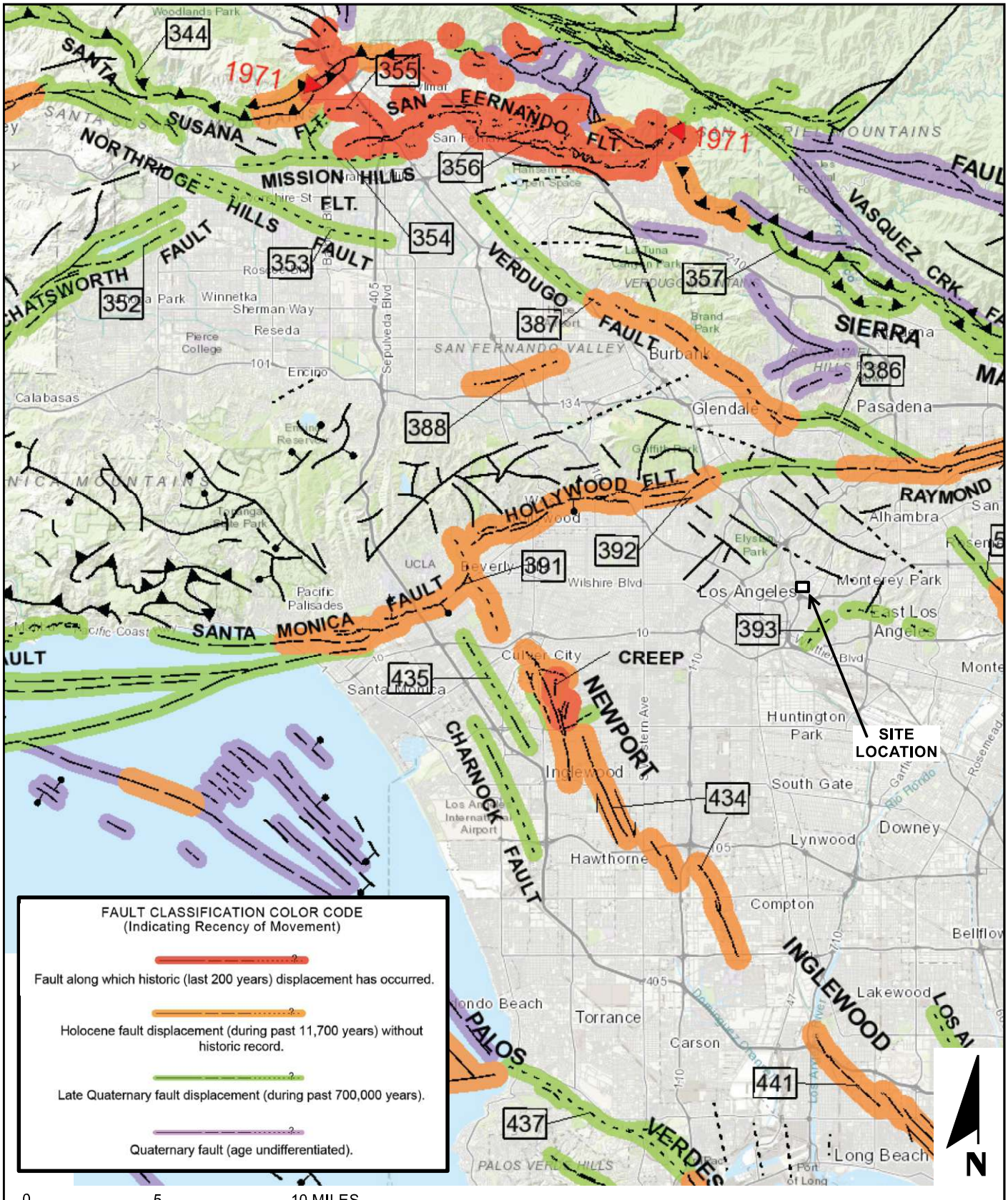
GPI GEOTECHNICAL PROFESSIONALS, INC.

LAC-USC OLD PHARMACY BUILDING

GPI PROJECT NO. 3080.22I | SCALE: 1" = 1 MILE

HISTORICAL GROUNDWATER MAP

FIGURE C-3



BASE MAP REPRODUCED FROM FAULT ACTIVITY MAP OF CALIFORNIA BY THE CALIFORNIA GEOLOGICAL SURVEY, C.W. JENNINGS, W.A. BRYANT: DATED 2010

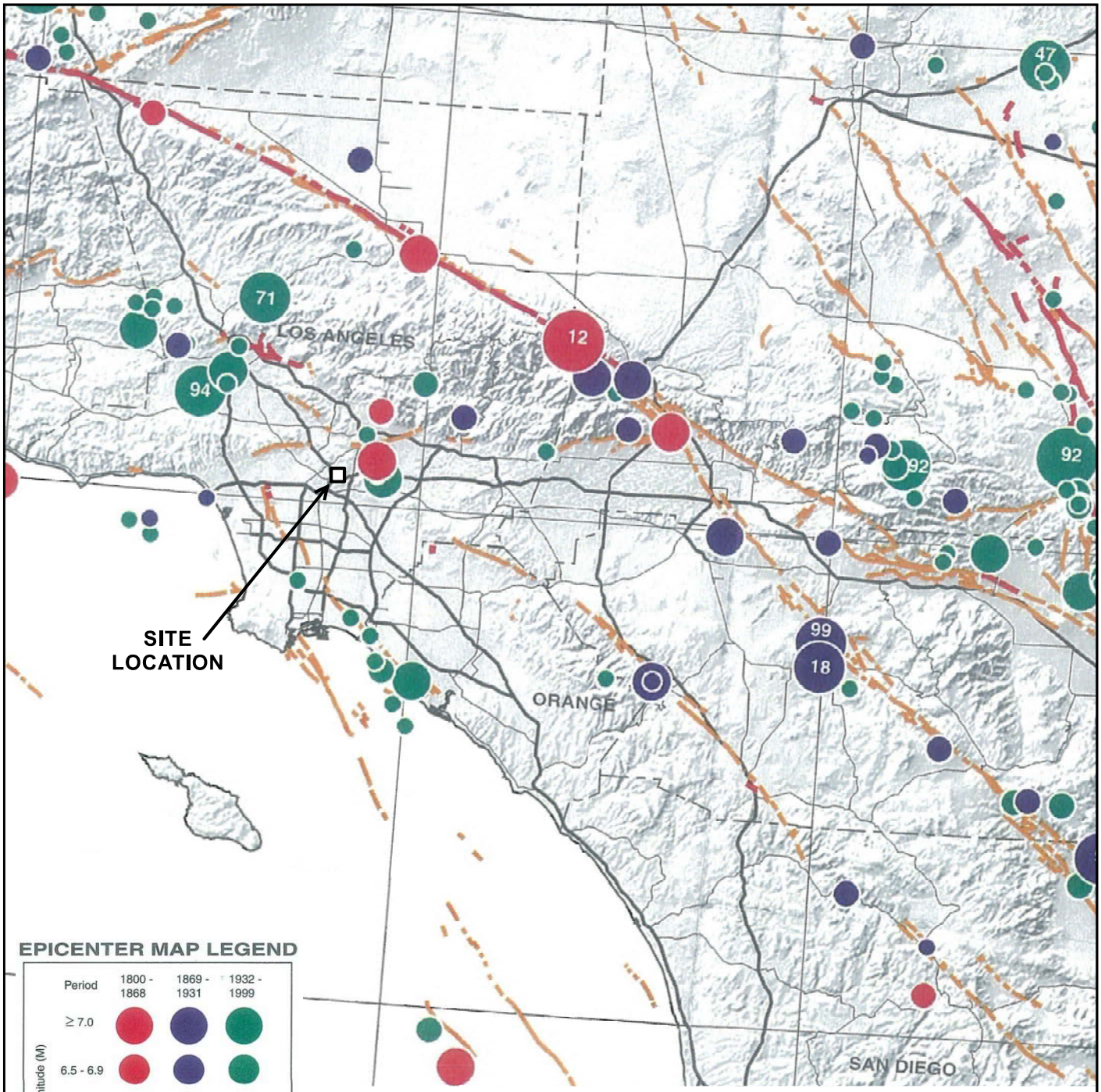
GPI GEOTECHNICAL PROFESSIONALS, INC.

REGIONAL FAULT MAP

LAC-USC OLD PHARMACY BUILDING

GPI PROJECT NO. 3080.22I | SCALE: 1" = 5 MILES

FIGURE C-4



REGIONAL FAULT MAP REPRODUCED FROM EARTHQUAKE FAULTS PLAN FROM MONTEREY PARK GENERAL PLAN BY BING YEN & ASSOCIATES, INC.: DATED JULY 2001

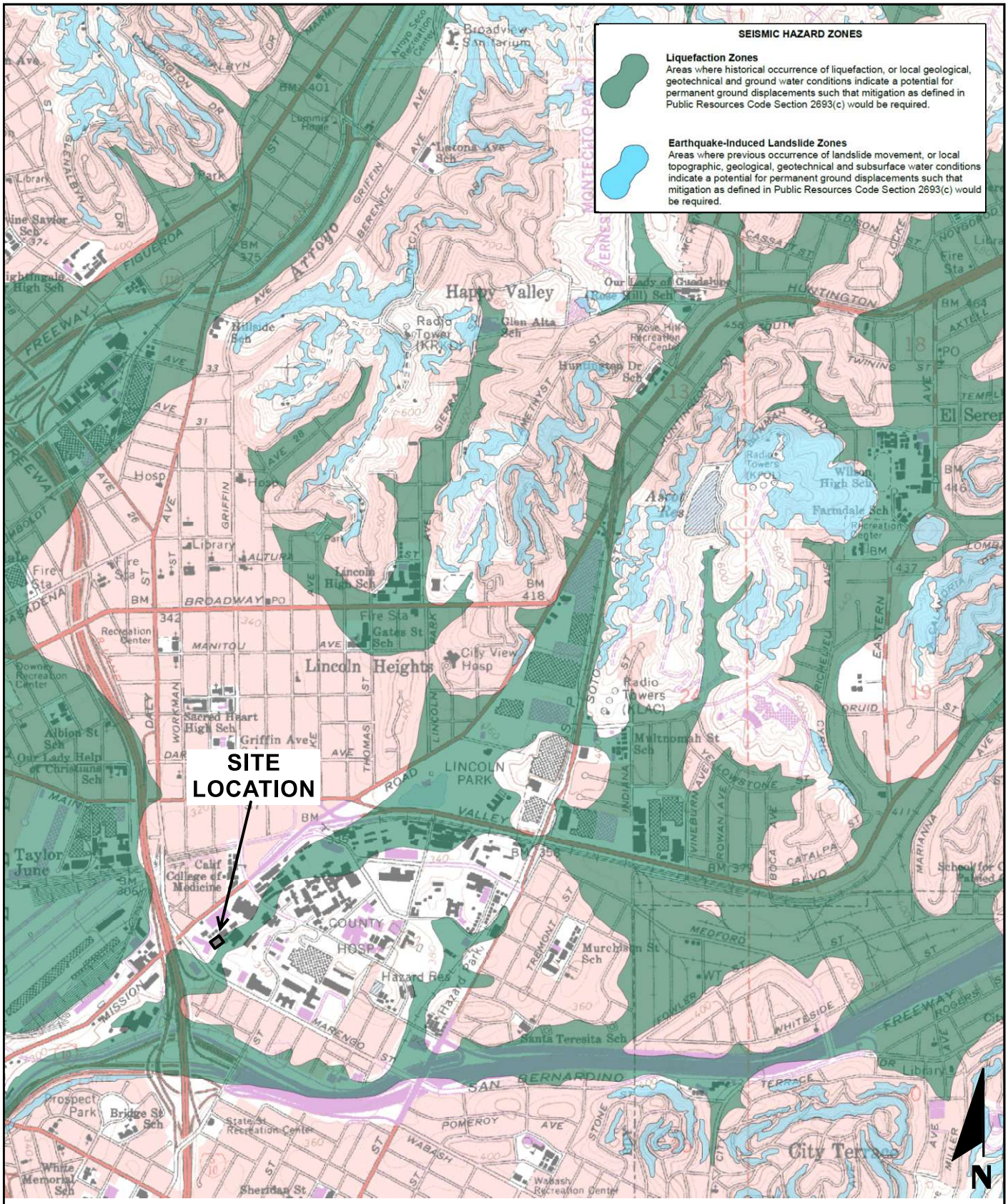
GPI GEOTECHNICAL PROFESSIONALS, INC.

LAC-USC OLD PHARMACY BUILDING

GPI PROJECT NO. 3080.22I | SCALE: 1" = 20 MILES

REGIONAL SEISMICITY

FIGURE C-5



BASE MAP REPRODUCED FROM EARTHQUAKE ZONES OF REQUIRED INVESTIGATION: LOS ANGELES QUADRANGLE PREPARED BY THE CALIFORNIA GEOLOGICAL SURVEY: RELEASED 06/15/2017

GPI GEOTECHNICAL PROFESSIONALS, INC.

LAC-USC OLD PHARMACY BUILDING

GPI PROJECT NO. 3080.22I | SCALE: 1" = 2000'

SEISMIC HAZARD ZONES MAP

FIGURE C-6

APPENDIX D

TERRA GEOSCIENCES

SEISMIC SHEAR-WAVE SURVEY

LAC + USC OLD PHARMACY BUILDING

1100 NORTH MISSION ROAD

CITY OF LOS ANGELES, CALIFORNIA

Project No. 223872-1

September 21, 2022

Prepared for:

Geotechnical Professionals Inc.
5736 Corporate Avenue
Cypress, CA 90630

Consulting Engineering Geology & Geophysics

P.O. Box 1090, Loma Linda, CA 92354 • 909 796-4667

Geotechnical Professionals Inc.
5736 Corporate Avenue
Cypress, CA 90630

Attention Mr. Justin J. Kempton, GE, Principal

Regarding: Seismic Shear-Wave Survey
LAC + USC Old Pharmacy Building
1100 North Mission Road
City of Los Angeles, California
GPI Project No. 3080.22I

INTRODUCTION

As requested, this firm has performed two seismic shear-wave surveys using the multi-channel analysis of surface waves (MASW) and microtremor array measurements (MAM) methods for the above-referenced site. The purpose of this survey was to assess the one-dimensional average shear-wave velocity structure beneath the subject survey area to a depth of at least 100 feet. Locally as mapped by Yerkes (1997), the site is shown to be surficially mantled by late Pleistocene age older alluvium, comprised of moderately- to well-consolidated and slightly- to well-cemented gravel, sand, silt, and clay, in turn underlain presumably by progressively older earth materials at depth.

The locations of the seismic traverses have been approximated on a captured Google™ Earth image (Google™ Earth, 2022), which is presented as the Seismic Line Location Map, Plate 3. Additionally, photographic views of the survey lines are presented on Plates 1 and 2 for visual and reference purposes. As authorized by you, the following services were performed during this study:

- ***Review of available pertinent published and unpublished geologic and geophysical data in our files pertaining to the site.***
- ***Performing a seismic surface-wave survey by a licensed State of California Professional Geophysicist that included two traverses for shear-wave velocity analysis purposes.***
- ***Preparation of this report, presenting the results of our findings with respect to the shear-wave velocities of the subsurface earth materials.***

Accompanying Map, Illustrations, and Appendices

Plates 1 & 2 - Site Photographs
Plate 3 - Seismic Line Location Map
Appendix A - Shear-Wave Models and Data
Appendix B - References

SUMMARY OF SHEAR-WAVE SURVEY

Methodology

The fundamental premise of this survey uses the fact that the Earth is always in motion at various seismic frequencies. These relatively constant vibrations of the Earth's surface are called microtremors, which are very small with respect to amplitude and are generally referred to as background "noise" that contain abundant surface waves. These microtremors are caused by both human activity (i.e., cultural noise, traffic, factories, etc.) and natural phenomenon (i.e., wind, wave motion, rain, atmospheric pressure, etc.) which have now become regarded as useful signal information. Although these signals are generally very weak, the recording, amplification, and processing of these surface waves has greatly improved by the use of technologically improved seismic recording instrumentation and recently developed computer software. For this application, we are mainly concerned with the Rayleigh wave portion of the seismic signals, which is also referred to as "ground roll" since the Rayleigh wave is the dominant component of ground roll.

For the purposes of this study, there are two ways that the surface waves were recorded, one being "active" and the other being "passive." Active means that seismic energy is intentionally generated at a specific location relative to the survey spread and recording begins when the source energy is imparted into the ground (i.e., MASW survey technique). Passive surveying, also called "microtremor surveying," is where the seismograph records ambient background vibrations (i.e., MAM survey technique), with the ideal vibration sources being at a constant level. Longer wavelength surface waves (longer-period and lower-frequency) travel deeper and thus contain more information about deeper velocity structure and are generally obtained with passive survey information. Shorter wavelength (shorter-period and higher-frequency) surface waves travel shallower and thus contain more information about shallower velocity structure and are generally collected with the use of active sources. For the most part, higher frequency active source surface waves will resolve the shallower velocity structure and lower frequency passive source surface waves will better resolve the deeper velocity structure. Therefore, the combination of both of these surveying techniques provides a more accurate depiction of the subsurface velocity structure.

The assemblage of the data that is gathered from these surface wave surveys results in development of a dispersion curve. Dispersion, or the change in phase velocity of the seismic waves with frequency, is the fundamental property utilized in the analysis of surface wave methods. The fundamental assumption of these survey methods is that the signal wavefront is planar, stable, and isotropic (coming from all directions) making it independent of source locations and for analytical purposes uses the spatial autocorrelation method (SPAC). The SPAC method is based on theories that are able to detect "signals" from background "noise" (Okada, 2003). The shear wave velocity (V_s) can then be calculated by mathematical inversion of the dispersive phase velocity of the surface waves which can be significant in the presence of velocity layering, which is common in the near-surface environment.

Field Procedures

Two seismic shear-wave survey traverses (Seismic Lines SW-1 and SW-2) were performed along selected portions of the subject site, as directed by you, which are approximated on the Seismic Line Location Map, Plate 3. The traverses were located in the field by use of Google™ Earth imagery (2022) and GPS coordinates. For data collection, the field survey employed a twenty-four channel Geometrics StrataVisor™ NZXP model signal-enhancement refraction seismograph. This survey employed both active (MASW) and passive (MAM) source methods to ensure that both quality shallow and deeper shear-wave velocity information was recorded (Park et al., 2005).

Both the MASW and MAM surveys used the same linear geometry array that consisted of a 184-foot-long spread using a series of twenty-four 4.5-Hz geophones that were spaced at regular eight-foot intervals. For the MASW survey, the ground vibrations were recorded using a one second record length at a sampling rate of 0.5-milliseconds. Two seismic records were obtained using 30-foot offsets from the beginning and end of each survey line, utilizing a 16-pound sledge-hammer as the energy source to produce the seismic waves. Each of these shot points used multiple hammer impacts (stacking) to improve the signal to noise ratio of the data.

The MAM survey did not require the introduction of any artificial seismic sources and only background ambient noise was recorded. The ambient ground vibrations were recorded using a thirty-two second record length at a two-millisecond sampling rate with 20 separate seismic records being obtained for quality control purposes. The seismic-wave forms and associated frequency spectrum that were displayed on the seismograph screen were used to assess the recorded seismic wave data for quality control purposes in the field. The acceptable records were digitally recorded on the in-board seismograph computer and subsequently transferred to a flash drive so that they could be subsequently transferred to our office computer for analysis.

Data Reduction

For analysis and presentation of the shear-wave profile and supportive illustrations, this study used the SeisImager/SW™ computer software program developed by Geometrics, Inc. (2004-2021). Both the active (MASW) and passive (MAM) survey results were combined for this analysis (Park et al., 2005). The combined results maximize the resolution and overall depth range in order to obtain one high resolution V_s curve over the entire sampled depth range. These methods economically and efficiently estimate one-dimensional subsurface shear-wave velocities using data collected from standard primary-wave (P-wave) refraction surveys, however, it should be noted that surface waves by their physical nature cannot resolve relatively abrupt or small-scale velocity anomalies. Processing of the data proceeded by calculating the dispersion curves from the input data which subsequently created initial shear-wave models based on the observed data. These models were then inverted in order to converge on the best fit of the initial models and the observed data, creating the final shear-wave profiles (Seismic Lines SW-1 and SW-2) as presented within Appendix A.

Summary of Data Analysis

Data acquisition went very smoothly and the quality was considered to be good. Analysis revealed that the average shear-wave velocity ("weighted average") in the upper 100 feet of the subject survey area for Seismic Line SW-1 is **1,253.5** feet per second, with Seismic Line SW-2 being **1,247.1** feet per second. These average velocities classify the underlying soils to that of Site Class "***Sc***" (Very Stiff/Very Dense Soil and Soft Rock), which has a velocity range from 1,200 to 2,500 feet/second (ASCE, 2017; Standard 7-16; Table 20.3-1).

The "weighted average" velocity is computed from a formula that is used by the ASCE (2010; Section 20.4, Equation 20.4-1) to determine the average shear-wave velocity for the upper 100 feet of the subsurface (V100). This formula is as follows:

$$V100' = 100/[(T1/V1) + (T2/V2) + ...+ (TN/VN)]$$

Where t1, t2, t3,...,tn, are the thicknesses for layers 1, 2, 3,...n, up to 100 feet, and v1, v2, v3,...,vn, are the seismic velocities (feet/second) for layers 1, 2, 3,...n.

The shear-wave models display these calculated layers and associated velocities (feet/second) to the maximum obtained depths of 235 feet (SW-1) and 209 feet (SW-2), where locally sampled (dark gray shaded area on shear-wave model represents the constrained data). The associated Dispersion Curves (for both the active and passive methods) which show the data quality and picks, along with the resultant combined dispersion curve model, are also included within Appendix A for visual and reference purposes.

It should be noted that when compared with traditional borehole shear-wave surveys, which use vertical body waves, the sources of error (if present) using horizontal surface waves for this project are not believed to be greater than 15 percent.

CLOSURE

The field survey was performed by the undersigned on September 17, 2022, using "state of the art" geophysical equipment and techniques along the selected portion of the subject study area as directed by you. It is important to note that the fundamental limitation for seismic surveys is known as nonuniqueness, wherein a specific seismic data set does not provide sufficient information to determine a single "true" earth model. Therefore, the interpretation of any seismic data set uses "best-fit" approximations along with the geologic models that appear to be most reasonable for the local area being surveyed. Client should also understand that when using the theoretical geophysical principles and techniques discussed in this report, sources of error are possible in both the data obtained and, in the interpretation, and that the results of this survey may not represent actual subsurface conditions.

These are all factors beyond **Terra Geosciences** control and no guarantees as to the results of this survey can be made. We make no warranty, either expressed or implied. If the client does not understand the limitations of this geophysical survey, additional input should be sought from the consultant.

Respectfully submitted,
TERRA GEOSCIENCES



Donn C. Schwartzkopf
Principal Geophysicist
PGP 1002



SITE PHOTOGRAPHS



View looking southwest along Seismic Line SW-1.



View looking northeast along Seismic Line SW-1.

SITE PHOTOGRAPHS



View looking southwest along Seismic Line SW-1.



View looking northeast along Seismic Line SW-1.

SEISMIC LINE LOCATION MAP



Base Map: Google™ Earth (2022); Approximate locations of seismic shear-wave traverses SW-1 & SW-2 shown as red/yellow lines.

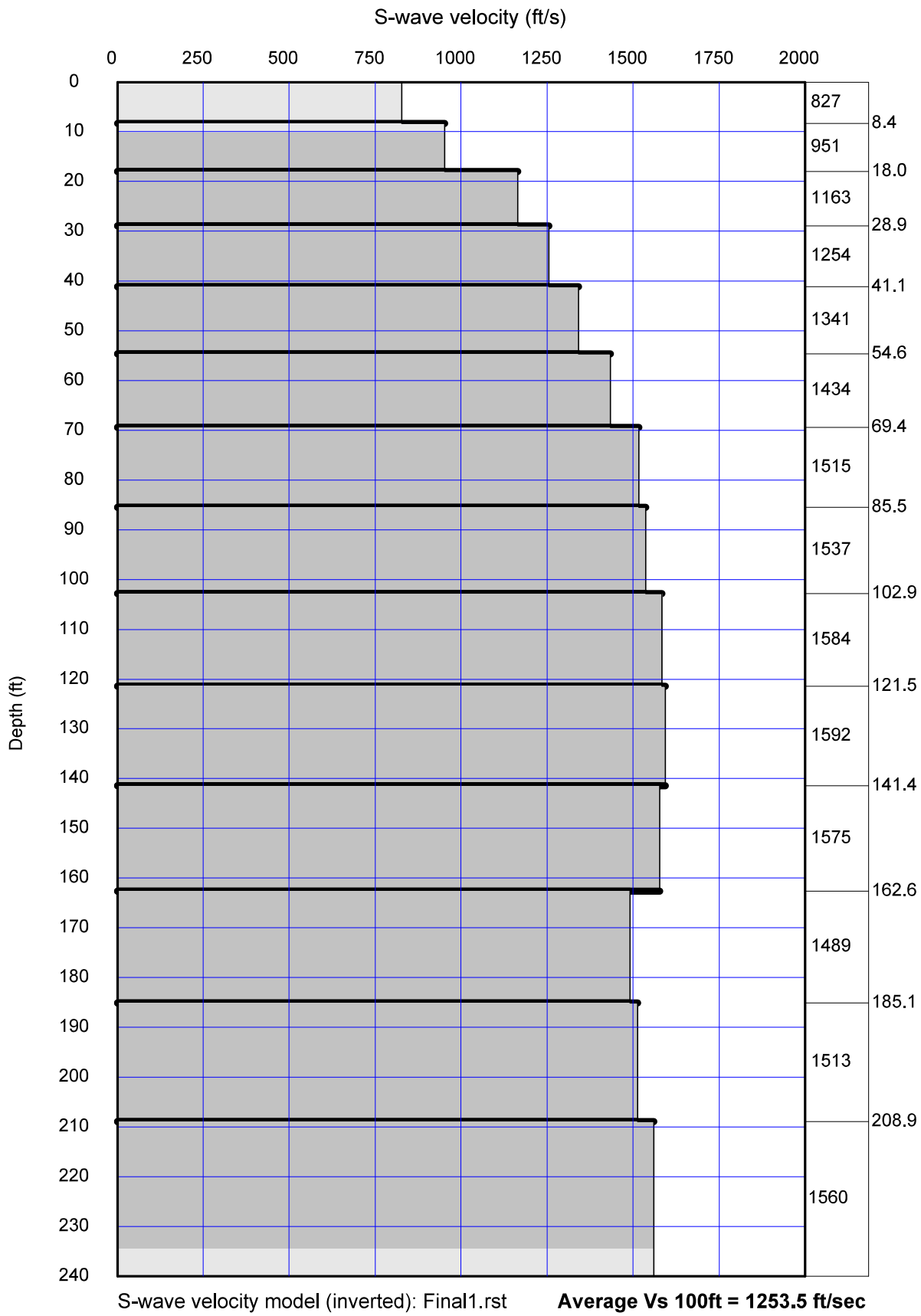
APPENDIX A

SHEAR-WAVE MODELS AND DATA



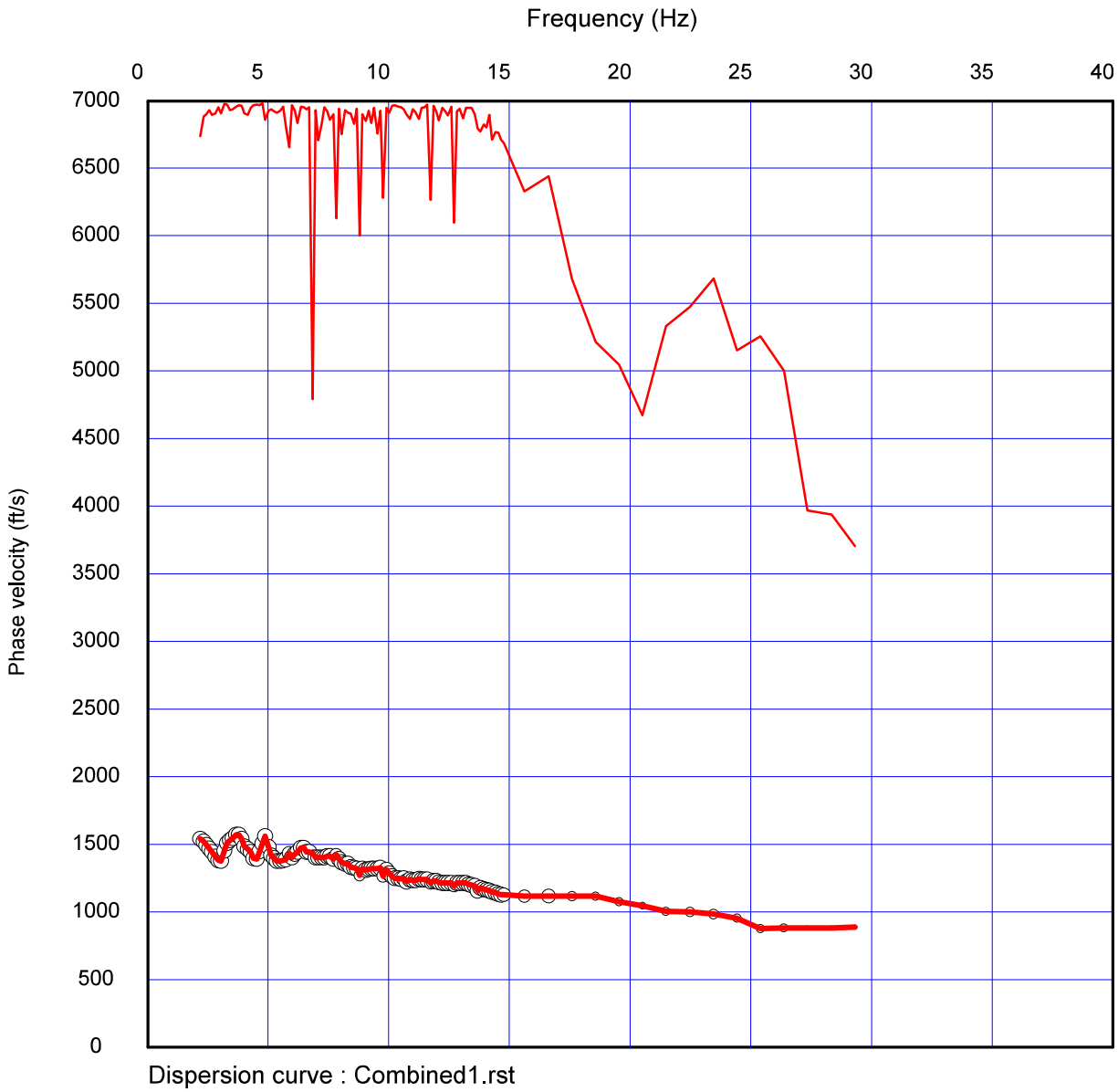
SEISMIC LINE SW-1

SHEAR-WAVE MODEL



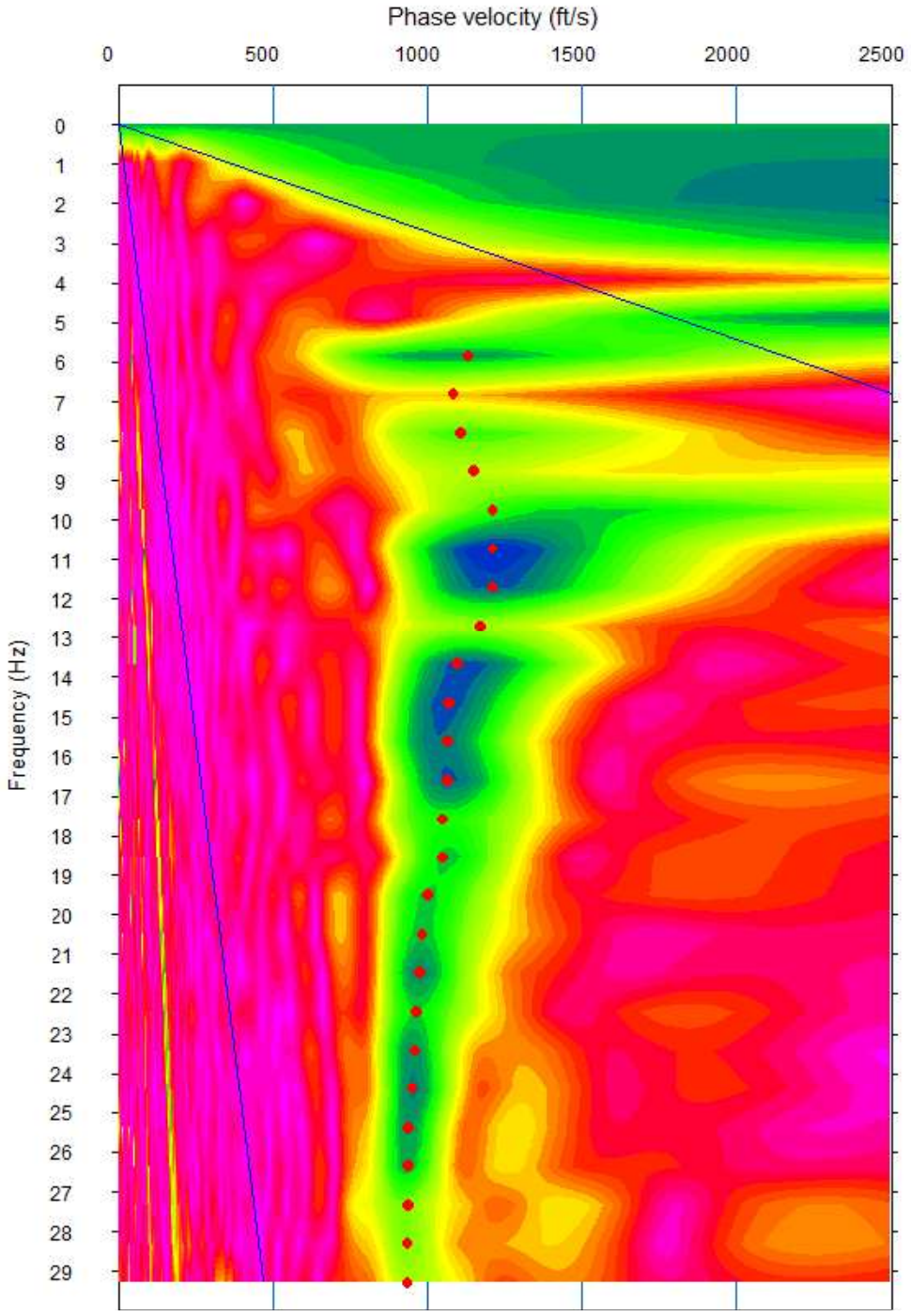
S-wave velocity model (inverted): Final1.rst **Average Vs 100ft = 1253.5 ft/sec**

SEISMIC LINE SW-1



COMBINED DISPERSION CURVE

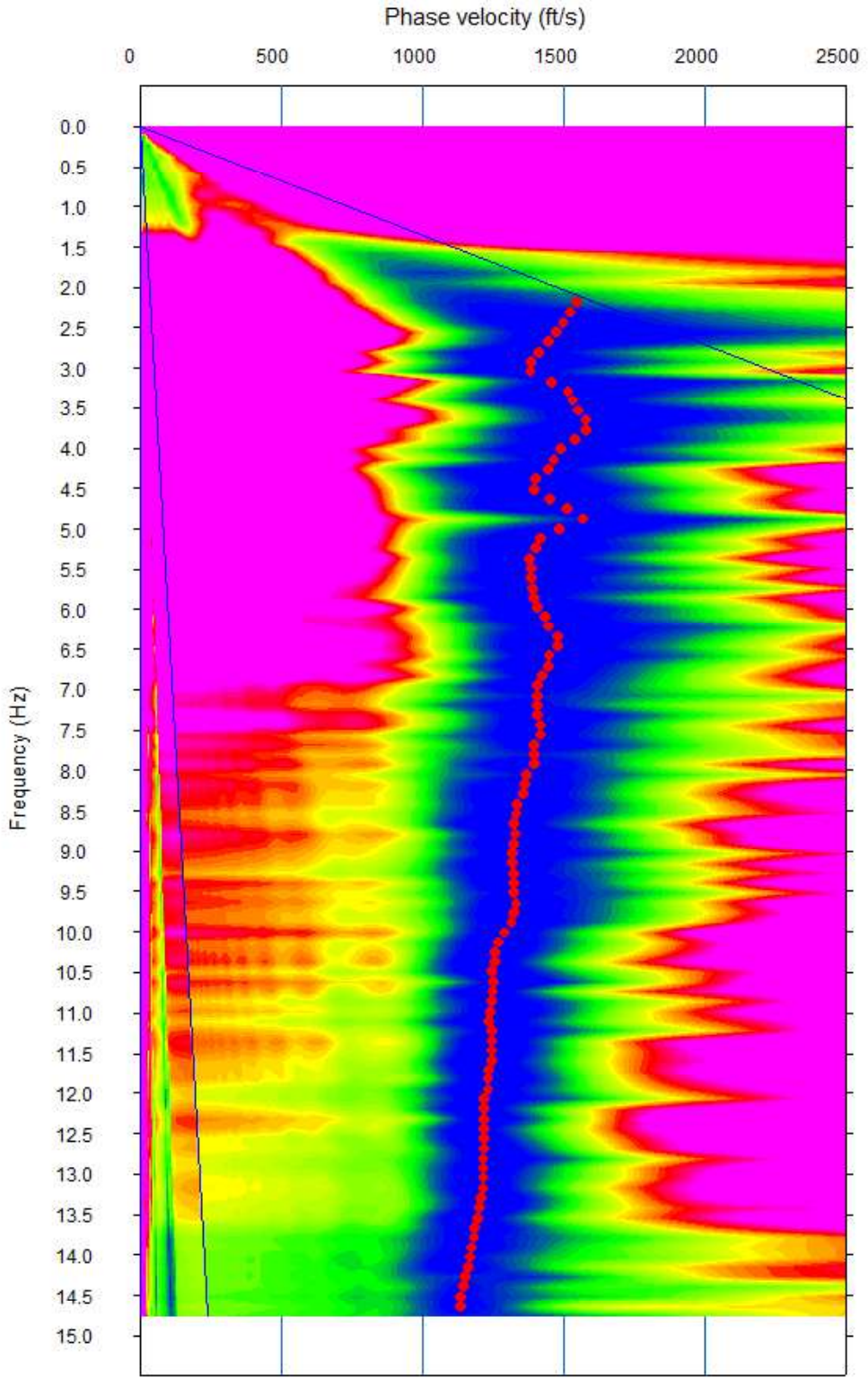
SEISMIC LINE SW-1



Dispersion Cure: Active.dat

ACTIVE DISPERSION CURVE

SEISMIC LINE SW-1

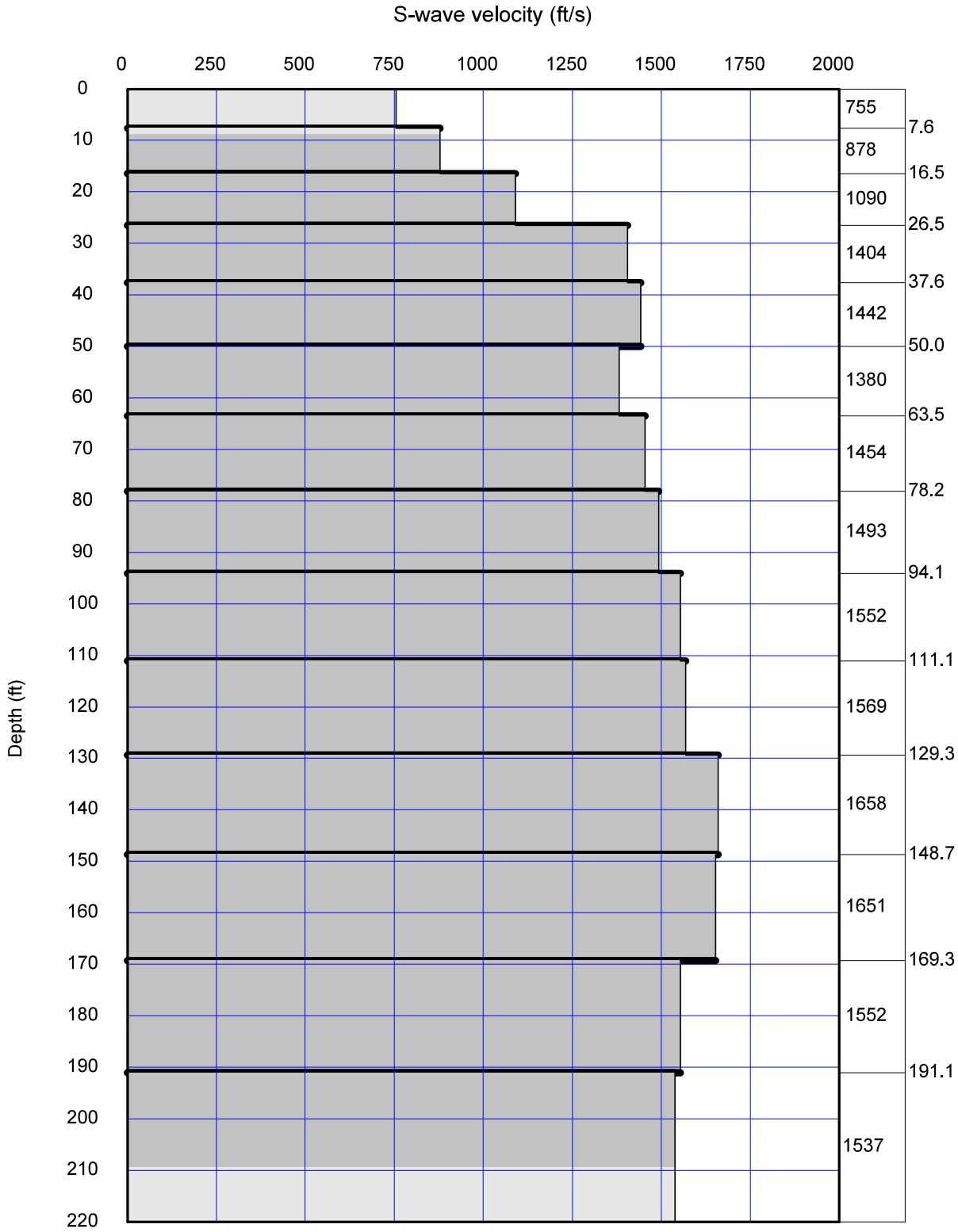


Dispersion Curve: Passive.dat

PASSIVE DISPERSION CURVE

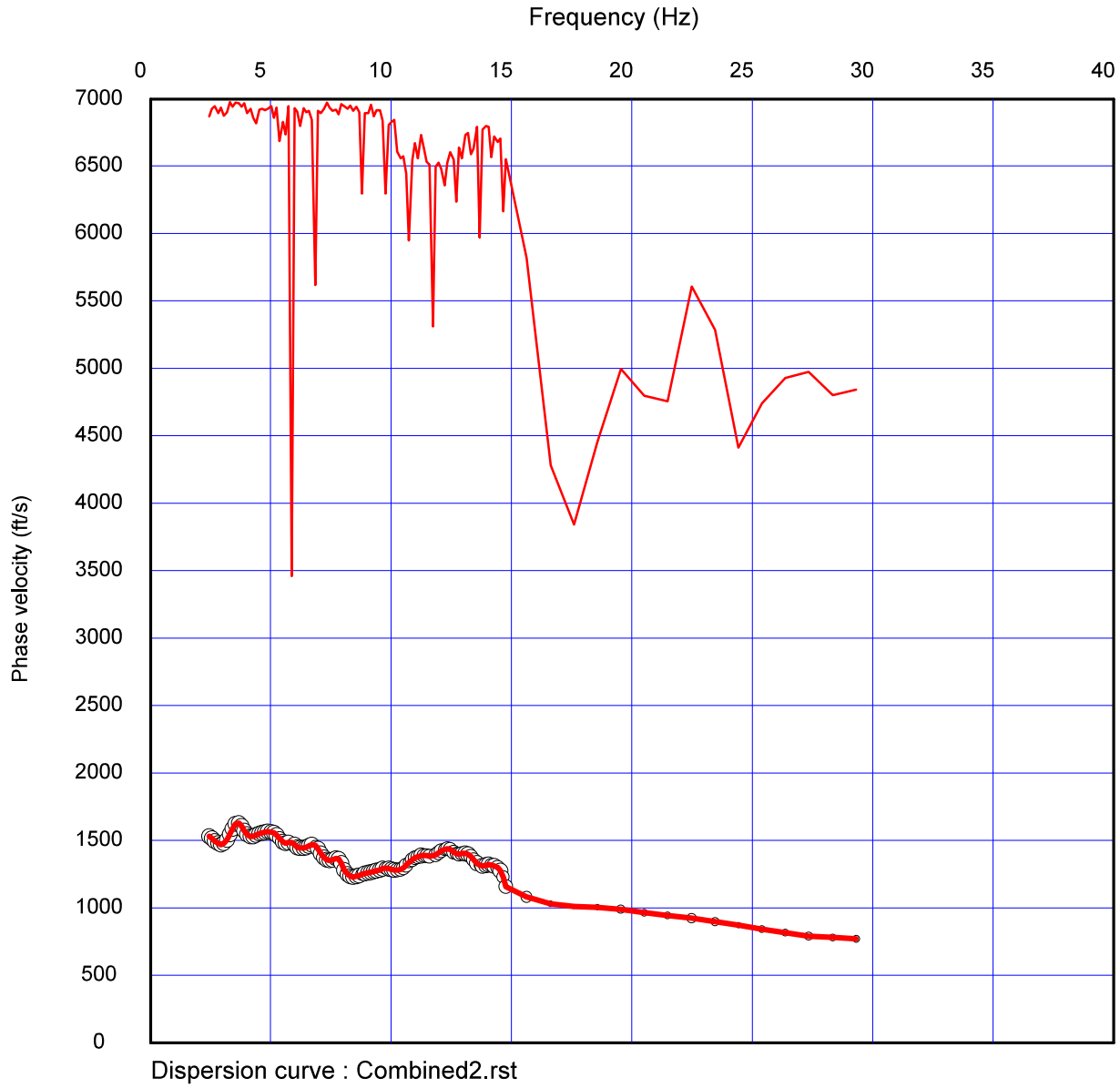
SEISMIC LINE SW-2

SHEAR-WAVE MODEL



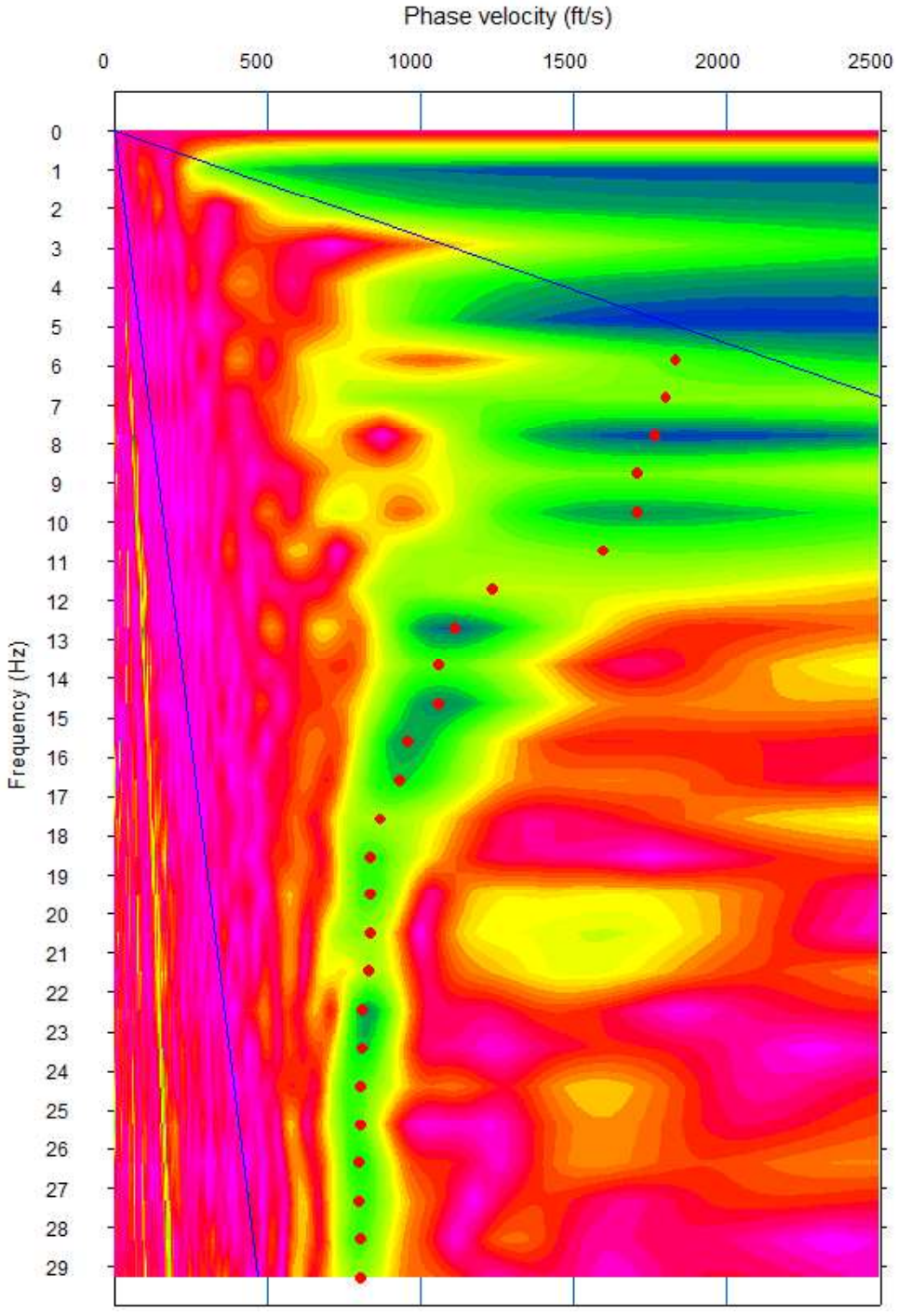
S-wave velocity model (inverted): Final2.rst **Average Vs 100ft = 1247.1 ft/sec**

SEISMIC LINE SW-2



COMBINED DISPERSION CURVE

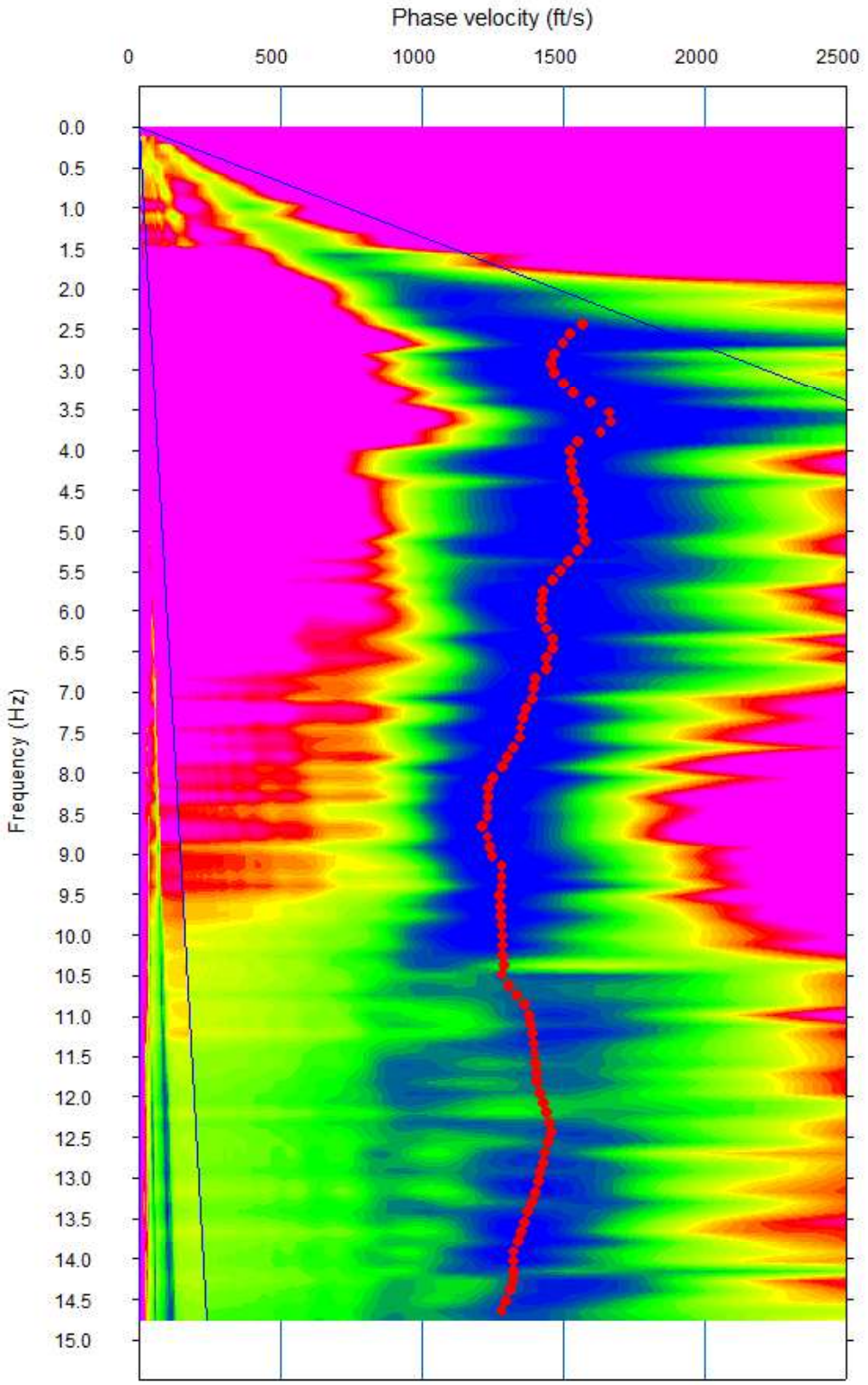
SEISMIC LINE SW-2



Dispersion Cure: Active.dat

ACTIVE DISPERSION CURVE

SEISMIC LINE SW-2



Dispersion Curve: Passive.dat

PASSIVE DISPERSION CURVE

APPENDIX B

REFERENCES

REFERENCES

American Society of Civil Engineers (ASCE), 2017, Minimum Design Loads and Associated Criteria for Buildings and other Structures, ASCE Standard 7-16, 889pp.

American Society for Testing and Materials, Intl. (ASTM), 2000, Standard Guide for Using the Seismic Refraction Method for Subsurface Investigation, Designation D 5777-00, 13 pp.

California Building Standards Commission (CBSC), 2019, 2019 California Building Code, California Code of Regulations, Title 24, Part 2, Volume 2 of 2.

California State Board for Geologists and Geophysicists, Department of Consumer Affairs, 1998, Guidelines for Geophysical Reports for Environmental and Engineering Geology, 5 pp.

Crice, Douglas B., undated, Shear Waves, Techniques and Systems, Reprinted by Geometrics, Sunnyvale, California.

Geometrics, Inc., 2012, StrataVisor™ NZXP Operation and Reference Manual, Revision M, Software Version 9.3, San Jose, California, 226 pp.

Geometrics, Inc., 2021, SeisImager/SW Analysis of Surface Waves, Pickwin Version 6.0.2.1. and WaveEq Version 5.2.6.0.

Google™ Earth, 2022, http://earth.google.com/, Version 7.3.4.8642 (64-bit).

Louie, J.N., 2001, Faster, Better: Shear-Wave Velocity to 100 Meters Depth rom Refraction Microtremor Arrays, *in,* Bulletin of the Seismological Society of America, Volume 91, pp. 347-364.

Okada, H., 2003, The Microtremor Survey Method, Society of Exploration Geophysicists, Geophysical Monograph Series Number 12, 135 pp.

Park, C.B, Milner, R.D., Rynden, N., Xia, J., and Ivanov, J., 2005, Combined use of Active and Passive Surface Waves, *in*, Journal of Environmental and Engineering Geophysics, Volume 10, Issue 3, pp. 323-334.

Yerkes, R.F., 1997, Preliminary Geologic Map of the Los Angeles 7.5' Quadrangle, Southern California, U.S.G.S. Open-File Report 97-254, Scale 1: 24,000.

APPENDIX F.6

GEOTECHNICAL ENGINEERING ASSESSMENT, LOS ANGELES COUNTY GENERAL HOSPITAL REDEVELOPMENT: PROPOSED STABILIZATION PROJECT, 1200 NORTH STATE STREET, LOS ANGELES, CALIFORNIA

Geotechnologies, Inc.
Consulting Geotechnical Engineers
439 Western Avenue
Glendale, California 91201-2837
818.240.9600 • Fax 818.240.9675

August 29, 2025
File Number 22662

Centennial GH Partners
9950 Jefferson Boulevard, Building 2
Culver City, California 90232

Attention: David Abasta

Subject: Geotechnical Engineering Assessment
Los Angeles County General Hospital Redevelopment
Proposed Stabilization Project
Los Angeles County General Hospital Campus
1200 North State Street, Los Angeles, California

Dear Mr. Abasta:

This letter transmits the Geotechnical Engineering Assessment for the project site prepared by Geotechnologies, Inc. This report provides general geotechnical recommendations for the proposed project. Engineering for the proposed project should not begin until approval of the geotechnical assessment is granted by the local building official. Significant changes in the geotechnical recommendations may result due to the building department review process.

The validity of the recommendations presented herein is dependent upon review of the geotechnical aspects of the project during construction by this firm. The subsurface conditions described herein have been projected from limited subsurface exploration and laboratory testing. The exploration and testing presented in this report should in no way be construed to reflect any variations which may occur between the exploration locations, or which may result from changes in subsurface conditions.

Should you have any questions please contact this office.

Respectfully submitted,
GEOTECHNOLOGIES, INC.

REGISTERED PROFESSIONAL ENGINEER GREGORIO VARELA No. 81201 Exp. 9/30/27 CIVIL STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER STANLEY SIU-KEI TANG R.C.E. 56178 EXP. 12/31/26 CIVIL STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER REINARD T. KNUR G.E. 2755 EXP. 12/31/26 GEOTECHNICAL STATE OF CALIFORNIA

PROFESSIONAL GEOLOGIST REINARD T. KNUR NO. 1547 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

GREGORIO VARELA R.C.E. 81201 | STANLEY S. TANG R.C.E. 56178 | REINARD T. KNUR G.E. 2755, C.E.G. 1547

GV/SST/RTK:km

Email to: [Lena@thecooperativela.com]

TABLE OF CONTENTS

SECTION **PAGE**

INTRODUCTION 1
SITE CONDITIONS 1
PROPOSED PROJECT 2
LOCAL GEOLOGY 3
GEOTECHNICAL EXPLORATION 4
PREVIOUS SITE EXPLORATIONS 4
FIELD EXPLORATION 5
Geologic Materials 5
Fill Materials 5
Alluvium 6
Bedrock 6
Groundwater 7
Caving 8
OIL FIELDS AND OIL WELLS 8
METHANE ZONES 8
SEISMIC EVALUATION 9
REGIONAL GEOLOGIC SETTING 9
REGIONAL FAULTING 9
SEISMIC HAZARDS AND DESIGN CONSIDERATIONS 10
Surface Rupture 10
Liquefaction 11
Dynamic Dry Settlement 13
Tsunamis, Seiches and Flooding 13
Landsliding 14
CONCLUSIONS AND RECOMMENDATIONS 14
Los Angeles County Code Sections 110 and 111 17
EXPANSIVE SOILS 17
SOIL CORROSION POTENTIAL 17
GRADING GUIDELINES 18
Site Preparation 18
Compaction 19
Acceptable Materials 19
Utility Trench Backfill 20
Wet Soils 20
Shrinkage 21
Weather Related Grading Considerations 21
Geotechnical Observations and Testing During Grading 22
LEED Considerations 22
PERMANENT CUTS 23
TEMPORARY EXCAVATIONS 24
Slot Cutting 25
Excavation Observations 25
SHORING DESIGN 26

TABLE OF CONTENTS

SECTION **PAGE**

Soldier Piles – Drilled and Poured 26
Soldier Piles – Vibrated 28
Lagging 30
Lateral Pressures 30
Tied-Back Anchors 33
Anchor Installation 34
Deflection 34
Monitoring 35
Shoring Observations 35
SLABS ON GRADE 36
Outdoor Flatwork 36
Concrete Crack Control 36
PAVEMENTS 36
SITE DRAINAGE 37
STORMWATER DISPOSAL 38
DESIGN REVIEW 39
CONSTRUCTION MONITORING 39
EXCAVATION CHARACTERISTICS 40
CLOSURE AND LIMITATIONS 40
EXCLUSIONS 42
GEOTECHNICAL TESTING 42
Classification and Sampling 42
Moisture and Density Relationships 43
Direct Shear Testing 43
Consolidation Testing 44
Expansion Index Testing 44
Laboratory Compaction Characteristics 44
Grain Size Distribution 45
Atterberg Limits 45

ENCLOSURES
References
Vicinity Map
Geologic Map
Regional Geologic Map
Cross Sections (4 sheets)
Local Geologic Map (Lamar)
Local Geologic Map (Dibblee)
Fault Location Map (NavigateLA)
Historically Highest Groundwater Levels Map
Earthquake Zones of Required Investigation Map
Oil Well Map
Methane Map

TABLE OF CONTENTS

SECTION **PAGE**

ENCLOSURES - continued
Plates A-1 through A-13
Plates B-1 through B-6
Plates C-1 through C-4
Plate D
Plate E
Plate F
Calculation Sheets (14 pages)
Corrosion Study by HDR (12 pages)
Logs from Previous Explorations (117 pages):
Geotechnologies, Inc., File No. 22579 (4 Borings)
Geotechnologies, Inc., February 7, 2024, File No. 22453 (4 Borings)
Geotechnical Professionals, Inc., Project No. 3080.221 (3 Borings)
Geotechnologies, Inc., File No. 22146 (8 Borings)
AECOM, Job No. 60595699 (10 Borings)
Geocon West, Inc., Project No. A8559-06-74 (10 Borings)
Geocon West, Inc., Project No. A9733-06-01 (5 Borings)
RMA Group, Job No. 18-0056-01 (3 Borings)
Dames & Moore, Job No. 00499-038-015 (2 Borings)
LeRoy Crandall & Assoc., Job No. 60383 (11 Borings)
LeRoy Crandall & Assoc., Job No. 56356 (7 Borings)
L.T. Evans, September 8, 1950 (7 Test Holes)
L.T. Evans, August 2, 1950 (6 Test Holes)
L.T. Evans, July 27, 1948 (5 Test Holes)

GEOTECHNICAL ENGINEERING ASSESSMENT
PROPOSED LOS ANGELES COUNTY GENERAL HOSPITAL REDEVELOPMENT
PROPOSED STABILIZATION PROJECT
LOS ANGELES COUNTY GENERAL HOSPITAL CAMPUS
1200 NORTH STATE STREET, LOS ANGELES, CALIFORNIA

INTRODUCTION

This report presents the results of the geotechnical engineering assessment performed on the project site. The purpose of this assessment was to identify the distribution and engineering properties of the geologic materials underlying the project site, and to provide geotechnical recommendations to aid in the proposed project.

This assessment included the excavation of eleven borings and two test pits, collection of representative samples, laboratory testing, engineering analysis, review of published geologic data, review of previous geotechnical investigation conducted at the site, and the preparation of this report. The exploratory excavation locations are shown on the enclosed Geologic Map. The results of the exploration and the laboratory testing are presented in the Appendix of this report.

SITE CONDITIONS

The Project Site consists of the Los Angeles County General Hospital and its West Campus, located at 1200 North State Street, in the Boyle Heights area of the City of Los Angeles, California. The project site is reported to be 41.9 acres in area, and is bounded by North Mission Road to the northwest, Women's and Children's Road and Zonal Avenue to the northeast, a parking structure, the Intern and Resident Building and an Outpatient Clinic to the east, the new LAC+USC Medical Center to the southeast, Marengo Street to the south and southwest, and the L.A. County Coroner to the west. The project site limits are illustrated in the enclosed Geologic Map and Vicinity Map.

The project site is currently occupied by the historic hospital building, several parking lots, driveways, miscellaneous buildings and miscellaneous retaining walls. North State Street bisects the project site along the north-south direction. The enclosed Geologic Map identifies the overall current site elevations and existing development. This map is based on the Topographic Survey prepared by Adkan Engineers, dated December 4, 2023. The highest project site elevation is approximately 390 feet above mean sea level (AMSL), and is found within the eastern area, around the perimeter of the historic hospital building. Beyond the historic hospital building, the site grade descends to the north, west and south. The lowest site elevation is approximately 306 feet AMSL, observed within the western area of the site, adjacent to Marengo Street.

Vegetation at the project site is limited, and consists of mature trees, shrubs and bushes, contained in planter areas. Drainage across the project site appears to be by sheetflow to the city streets.

PROPOSED PROJECT

The Los Angeles County General Hospital Redevelopment Project is an ambitious initiative that aims to transform the historic Los Angeles County General Hospital and the West Campus into a vibrant, mixed-use district. Spanning 41.9 acres, the project will offer a dynamic mix of residential, commercial, and healthcare spaces.

The first construction phase is a comprehensive plan to maintain, remediate, and modernize the Los Angeles County General Hospital and West Campus, giving continuity to the 2014 LAC+USC Medical Center Master Plan execution. The primary goals of this initiative are to eliminate hazardous and unsafe conditions, upgrade historic structures, and modernize campus utilities for better efficiency, service delivery, and user safety and wellbeing. This project phase is structured to ensure compliance with regulatory frameworks while facilitating timely and safe implementation. In addition, this first construction phase is expected to include the demolition of several existing buildings, as well as the redefinition of some of the existing site grades.

The geotechnical engineering assessment presented herein provides recommendations to aid in the design and construction of the first construction phase, with the exception of the retrofit and repurposing of the historic hospital building. Additionally, this assessment provides a baseline report to aid the design team in establishing a framework for preliminary siting and conceptual design as part of the master-planning for the redevelopment project. Additional geotechnical explorations, laboratory testing and engineering analyses will be necessary in the future for design and permitting for each individual building.

Any changes in the design of the project or location of any structure, as outlined in this report, should be reviewed by this office. The recommendations contained in this report should not be considered valid until reviewed and modified or reaffirmed, in writing, subsequent to such review.

LOCAL GEOLOGY

The site is located in the Repetto Hills northeast of downtown Los Angeles, in the Los Angeles Basin. The Repetto Hills are composed of Upper Miocene-age, fine grained, sedimentary rock of the Puente Formation and uplifted alluvial deposits. Small, secondary folds on the Elysian Park Anticline have resulted in bedding dipping steeply to the northeast and southwest. The geology of the site vicinity is presented on the Local Geologic Map included in the Appendix of this report.

The Repetto Hills are located near the nose of the southeast-plunging Elysian Park Anticline. The Elysian Park Anticline is formed due to movement on the Elysian Park Fault. The Elysian Park Fault is mapped by Lamar (1970) to be located 4,500 feet northeast of the site. However, the NavigateLA Website identifies the Elysian Park Fault approximately 600 feet northeast of the project site. Both the Lamar and NavigateLA maps are included in the Appendix. No evidence has yet been cited that suggests recent surface activity of this mapped fault. Recent work by Shaw and Suppe (1996), suggests the Elysian Park Fault is an active blind thrust fault, which by definition, does not have a ground surface expression.

GEOTECHNICAL EXPLORATION

PREVIOUS SITE EXPLORATIONS

This firm has reviewed the exploration logs from several investigations previously conducted at the project site, and its immediate vicinity. These previous investigations are listed below in chronological order and include the number of excavations performed. The enclosed Geologic map provides the location of these previous explorations, as well as a summary of the subsurface conditions encountered at each location. Logs of these previous explorations are included in the Appendix. It should be noted that the site conditions encountered at the time of the older explorations should be expected to be significantly different from the current conditions. As such, the upper stratigraphy and elevations identified in the older exploration logs may not be reflective of current conditions. This would be expected for the explorations listed as Items #9 through #14 below.

1. Geotechnologies, Inc., November 5, 2024, File No. 22579 (4 Borings)
2. Geotechnologies, Inc., February 7, 2024, File No. 22453 (4 Borings)
3. Geotechnical Professionals, Inc., November 7, 2022, Project No. 3080.221 (3 Borings)
4. Geotechnologies, Inc., September 1, 2021, File No. 22146 (8 Borings)
5. AECOM, October 21, 2019, Job No. 60595699 (10 Borings)
6. Geocon West, Inc., May 20, 2019, Project No. A8559-06-74 (10 Borings)
7. Geocon West, Inc., April 9, 2018, Project No. A9733-06-01 (5 Borings)
8. RMA Group, March 30, 2018, Job No. 18-0056-01 (3 Borings)
9. Dames & Moore, December 13, 1993, Job No. 00499-038-015 (2 Borings)
10. LeRoy Crandall & Assoc., September 30, 1960, Job No. 60383 (11 Borings)
11. LeRoy Crandall & Assoc., February 11, 1957, Job No. 56356 (7 Borings)
12. L.T. Evans, September 8, 1950 (7 Test Holes)
13. L.T. Evans, August 2, 1950 (6 Test Holes)
14. L.T. Evans, July 27, 1948 (5 Test Holes)

FIELD EXPLORATION

The site was explored by our firm on July 14 through 17, 2025 by excavating eleven borings and two test pits. The borings were drilled to depths ranging between 20 and 50 feet below the existing grade, with the aid of a truck-mounted drilling machine and a limited-access drilling machine, both using 8-inch diameter hollowstem augers. The test pits were excavated to a depth of 13 and 20 feet below the existing grade, with the aid of hand tools and a 4-inch diameter hand auger. The exploration locations are shown on the Geologic Map and the geologic materials encountered are logged on Plates A-1 through A-13.

The location and elevation of the exploratory excavations was determined from the Topographic Survey prepared by Adkan Engineers, dated December 4, 2023.

Geologic Materials

Based on review of the exploratory excavations conducted by our firm, as well as previous geotechnical consultants, the project site is underlain by fill materials, natural alluvium, and bedrock of the Puente Formation. These geologic materials are addressed below:

Fill Materials

Fill materials were encountered in most of the exploratory excavations. The depth of fill varies widely throughout the project site. The deepest fill was observed along the southern and central portions of the site, where the fill was observed to extend up to a depth of 22½ feet below the existing grade. The fill depth observed at each exploratory excavation is provided in the enclosed Geologic Map.

The composition of the fill is widely variable, consisting of a mixture of clay, silt, sand, and recompacted siltstone and sandstone. The fill is generally moist, stiff to very stiff, or medium dense to very dense, and fine to medium grained, with occasional brick, concrete and rock fragments. A compaction report documenting the existing fill was not located. Therefore, the existing fill is considered to be uncertified.

Alluvium

Within most of the project site, particularly within the lower elevation areas, the fill was observed to be underlain by native alluvial soils. The alluvial soils consist of interlayered mixtures of sandy to silty clay, sandy to clayey silt, silty to clayey sand, and sand. The alluvial soils are moist to wet, stiff to very stiff, or medium dense to very dense, and fine to coarse grained, with occasional pebbles and gravel.

Bedrock

Bedrock of the Puente Formation was identified in some of the exploratory excavations conducted within the central area of the project site. At this area, the bedrock was observed at depths ranging between 1 and 24½ feet below the existing grade. Based on review of the exploration logs, the bedrock appears to be shallow within the higher site elevations, around the historic hospital building, and seems to deepen towards the north, south and west. This is consistent with the findings of Yerkes (1977). The enclosed Geologic Map provides the depth to bedrock when encountered, and the cross sections illustrate the approximate bedrock elevations.

The observed bedrock consists of siltstone and sandstone, which is moist, moderately hard to hard, occasionally weathered, and generally well bedded. From all the excavations conducted within the project site, bedrock bedding could only be recorded in Borings B1 and B2, performed by RMA Group. The bedding observed in these two borings dips between 43 and 64 degrees to the southwest. The observed bedding orientation is consistent with the geologic map by Dibblee

(1991) and Lamar (1970), which show that the bedding should be expected to dip to the southwest, at angles ranging between 11 and 84 degrees.

More detailed descriptions of the earth materials encountered may be obtained from individual logs of the subsurface excavations.

Groundwater

Groundwater was encountered in some of the exploratory excavations. The table below provides a summary of the groundwater levels encountered in some of the most recent borings:

Geotechnical Consultant	Boring No.	File No.	Date of Drilling	Depth to Water Below G.S (feet)	Groundwater Elevation AMSL (feet)
Geotechnologies, Inc.	B3	22662	7/15/25	26	306.5
Geotechnologies, Inc.	B6	22662	7/14/25	26.5	283.5
Geotechnologies, Inc.	B8	22662	7/17/25	43	288.5
Geotechnologies, Inc.	B9	22662	7/16/25	31	283.5
Geotechnologies, Inc.	B1	22662	10/09/24	29.5	283.3
Geotechnologies, Inc.	B2	22662	10/10/24	29.5	284.8
G.P.I.	B1	3080.221	9/22/22	54	270
G.P.I.	B2	3080.221	9/23/22	26	279
G.P.I.	B3	3080.221	9/23/22	35	271

According to the Seismic Hazard Zone Report of the Los Angeles 7½-Minute Quadrangle (CDMG, 1998, Revised 2006), the historically highest groundwater level is not well defined in the vicinity of the site. This is likely due to the presence of the shallow, poor water-bearing, bedrock materials. The nearest contour presented in the map indicates a historically highest groundwater level of 20 feet below the existing grade.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can result in changed conditions.

Caving

Based on the experience of this firm, large diameter excavations, excavations that encounter granular, cohesionless soils and excavations below the groundwater table will most likely experience caving.

OIL FIELDS AND OIL WELLS

Based on a review of the Interactive Well Finder Map Application, developed by the California Department of Conservation, the site is not located within the limits of an oil field. However, this website identifies an oil and gas well installed within the southern portion of the site, adjacent to Marengo Street. The location of this well is shown in the enclosed Oil Well Location Map. This well is labeled as "Idle". If grading is planned to be conducted in the vicinity of this well, it is recommended that additional information is obtained to determine the exact location of this well, and its current condition.

METHANE ZONES

Based on review of the NavigateLA Website, developed by the City of Los Angeles, Bureau of Engineering, Department of Public Works, the subject site is located within the limits of a City of Los Angeles Methane Zone. A copy of this map is attached. It is recommended that an expert in the field of methane analysis and mitigation should be consulted to determine what mitigating measures, if any, may be required.

SEISMIC EVALUATION

REGIONAL GEOLOGIC SETTING

The subject property is located within the Los Angeles Basin and Peninsular Ranges Geomorphic Province. The Peninsular Ranges are characterized by northwest-trending blocks of mountain ridges and sediment-floored valleys. The dominant geologic structural features are northwest trending fault zones that either die out to the northwest or terminate at east-west trending reverse faults that form the southern margin of the Transverse Ranges.

The Los Angeles Basin is located at the northern end of the Peninsular Ranges Geomorphic Province. The basin is bounded by the east and southeast by the Santa Ana Mountains and San Joaquin Hills, to the northwest by the Santa Monica Mountains. Over 22 million years ago the Los Angeles basin was a deep marine basin formed by tectonic forces between the North American and Pacific plates. Since that time, over 5 miles of marine and non-marine sedimentary rock as well as intrusive and extrusive igneous rocks have filled the basin. During the last 2 million years, defined by the Pleistocene and Holocene epochs, the Los Angeles basin and surrounding mountain ranges have been uplifted to form the present day landscape. Erosion of the surrounding mountains has resulted in deposition of unconsolidated sediments in low-lying areas by rivers such as the Los Angeles River. Areas that have experienced subtle uplift have been eroded with gullies (Yerkes, 1965).

REGIONAL FAULTING

Based on criteria established by the California Division of Mines and Geology (CDMG) now called California Geologic Survey (CGS), Faults may be categorized as Holocene-active, Pre-Holocene faults, and Age-undetermined faults. Holocene-active faults are those which show evidence of surface displacement within the last 11,700 years. Pre-Holocene faults are those that have not moved in the past 11,700 years. Age-undetermined faults are faults where the recency of fault movement has not been determined.

The Elysian Park blind thrust fault is mapped by Lamar (1970) to be located 4,500 feet northeast of the site, and by the NavigateLA Website to be approximately 600 feet northeast of the project site. Both of these maps are enclosed. Buried thrust faults are faults without a surface expression but are a significant source of seismic activity. They are typically broadly defined based on the analysis of seismic wave recordings of hundreds of small and large earthquakes in the southern California area. The Elysian Park blind thrust fault has been estimated to cause an earthquake every 500 to 1,300 years in the magnitude range 6.2 to 6.7. The risk for surface rupture potential of buried thrust faults is inferred to be low (Leighton, 1990).

SEISMIC HAZARDS AND DESIGN CONSIDERATIONS

The primary geologic hazard at the site is moderate to strong ground motion (acceleration) caused by an earthquake on any of the local or regional faults. The potential for other earthquake-induced hazards was also evaluated including surface rupture, liquefaction, dynamic settlement, inundation and landsliding.

Surface Rupture

In 1972, the Alquist-Priolo Special Studies Zones Act (now known as the Alquist-Priolo Earthquake Fault Zoning Act) was passed into law. As revised in 2018, The Act defines "Holocene-active" Faults utilizing the same aging criteria as that used by California Geological Survey (CGS). However, established state policy has been to zone only those faults which have direct evidence of movement within the last 11,700 years. It is this recency of fault movement that the CGS considers as a characteristic for faults that have a relatively high potential for surface rupture in the future.

CGS policy is to delineate a boundary from 200 to 500 feet wide on each side of the Holocene-Active fault trace based on the location precision, the complexity, or the regional significance of the fault. If a site lies within an Earthquake Fault Zone, a geologic fault rupture investigation must

be performed that demonstrates that the proposed building site is not threatened by surface displacement from the fault before development permits may be issued.

Surface rupture is defined as surface displacement which occurs along the surface trace of the causative fault during an earthquake. Based on research of available literature and results of site reconnaissance, no known Holocene-active or Pre-Holocene faults underlie the subject site. In addition, the subject site is not located within an Alquist-Priolo Earthquake Fault Zone. Based on these considerations, the potential for surface rupture at the subject site is considered low.

Liquefaction

Liquefaction is a phenomenon in which saturated silty to cohesionless soils below the groundwater table are subject to a temporary loss of strength due to the buildup of excess pore pressure during cyclic loading conditions such as those induced by an earthquake. Liquefaction-related effects include loss of bearing strength, amplified ground oscillations, lateral spreading, and flow failures.

The Seismic Hazards Maps for the Los Angeles Quadrangle of the State of California (CDMG, 1999), classify a small portion of the site as part of the potentially "Liquefiable" area. As shown in the enclosed Earthquake Zones of Required Investigation Map, this liquefiable area is located within the northwestern portion of the site. The "Liquefiable" area determination is based on groundwater depth records, soil type and distance to a fault capable of producing a substantial earthquake. A copy of this map is provided in the Appendix as the "Earthquake Zones of Required Investigation Map".

Borings B6 and B8 from this investigation, as well as Borings B1 and B2 from our previous investigation performed under File No. 22579, were drilled within the liquefiable area, or its immediate vicinity. Four site-specific liquefaction analyses, based on these four borings, were performed as part of this assessment following the Recommended Procedures for Implementation of the California Geologic Survey Special Publication 117A, Guidelines for Analyzing and

Mitigating Seismic Hazards in California (CGS, 2008), and the EERI Monograph (MNO-12) by Idriss and Boulanger (2008). This semi-empirical method is based on a correlation between measured values of Standard Penetration Test (SPT) resistance and field performance data.

According to the Seismic Hazard Zone Report of the Los Angeles 7½-Minute Quadrangle (CDMG, 1998, Revised 2006), the historically highest groundwater level for the site was 20 feet below the ground surface. The current groundwater level observed at each boring, as well as the historically highest groundwater level mapped for the site, were utilized for the enclosed liquefaction analyses.

Section 11.8.3 of ASCE 7-16 indicates that the potential for liquefaction shall be evaluated utilizing an acceleration consistent with the MCE_G PGA. Utilizing the OSHPD seismic utility program, this corresponds to a PGA_M of 0.94g. The USGS Probabilistic Seismic Hazard Deaggregation program (USGS, 2014) indicates a PGA of 0.88g (2 percent in 50 years ground motion) and a mean magnitude of 6.9 for the site. The liquefaction potential evaluations were performed by utilizing a magnitude 6.9 earthquake, and a peak horizontal acceleration of 0.94g.

The enclosed "Liquefaction Evaluation" analyses utilized the Standard Penetration Test data recorded at 5-foot intervals from the four borings listed above. Soil samples were collected and conveyed to the laboratory for testing and analysis. The percent passing Number 200 sieve, Atterberg Limits, and Plasticity Index (PI) of the representative soil samples collected during our field exploration are presented on Plates E and F.

The procedure presented in the SP117A guidelines was followed in analyzing the liquefaction potential of the subject site. The SP117A guidelines were developed based on a paper titled, "Assessment of the Liquefaction Susceptibility of Fine-Grained Soils", by Bray and Sancio (2006). According to the SP117A, soils having a Plastic Index greater than 18 exhibit clay-like behavior, and the liquefaction potential of these soils are considered to be low. Therefore, where the results of Atterberg Limits testing showed a Plastic Index greater than 18, the soils would be considered

non-liquefiable, and the analysis of these soil layers was turned off in the liquefaction susceptibility column.

Based on CGS Special Publication 117A (CDMG, 2008), a factor of safety against the occurrence of liquefaction greater than about 1.3 can be considered an acceptable level of risk where high-quality, site-specific penetration resistance and geotechnical laboratory data is collected. The four site-specific liquefaction analyses presented in the Appendix indicate that the site soils would not be susceptible to liquefaction during the ground motion expected during the design-based seismic event. Furthermore, the site is not expected to be affected by potential impacts related to liquefaction, such as lateral spreading and surface manifestation.

Dynamic Dry Settlement

Seismically-induced settlement or compaction of dry or moist, cohesionless soils can be an effect related to earthquake ground motion. Such settlements are typically most damaging when the settlements are differential in nature across the length of structures. Based on the cohesive nature of the majority of the underlying geologic materials, excessive differential settlements are not expected to occur.

Tsunamis, Seiches and Flooding

Tsunamis are large ocean waves generated by sudden water displacement caused by a submarine earthquake, landslide, or volcanic eruption. Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site does not lie within the mapped tsunami inundation boundaries.

Review of the County of Los Angeles Flood and Inundation Hazards Map, Leighton (1990), indicates the site does not lie within the potential mapped inundation boundaries of a dam or reservoir.

Based on review of the corresponding FEMA Flood Insurance Rate Map (06037C1637F, 2008), the site is located in an area of minimal flood hazard (Zone X).

Landsliding

No indications of landsliding were observed during the site investigation. The Geologic Map by Lamar (1970) does not show a landslide on, or adjacent to the site. Based on review of the Seismic Hazards Maps for the Los Angeles Quadrangle of the State of California (CDMG, 1999), an "Earthquake-Induced landslide Area" is not mapped within the project site or its immediate vicinity. The probability of seismically-induced landslides occurring on the site is considered to be remote.

CONCLUSIONS AND RECOMMENDATIONS

Based upon the exploration, laboratory testing, and research, it is the finding of Geotechnologies, Inc. that construction of the proposed project is considered feasible from a geotechnical engineering standpoint provided the advice and recommendations presented herein are followed and implemented during construction.

The primary intention of this geotechnical engineering assessment is to provide recommendations to aid in the demolition of several existing buildings, as well as the redefinition of some of the existing site grades, as proposed under the first phase of the project. Due to the conceptual phase of this project, there are no specific demolition or grading plans available at the time of preparation of this report. The proposed demolition and grading plans shall be provided to this firm for review so that the recommendations contained herein may be confirmed or modified as necessary.

Based on review of the exploratory excavations conducted by our firm, as well as previous geotechnical consultants, the project site is underlain by fill materials, natural alluvium, and bedrock of the Puente Formation. The depth of fill varies widely throughout the project site. The

deepest fill was observed along the southern and central portions of the site, where the fill was observed to extend up to a depth of 22½ feet below the existing grade. Bedrock of the Puente Formation was identified in some of the exploratory excavations conducted within the central area of the project site. At this area, the bedrock was observed to depth ranging between 1 and 24½ feet below the existing grade. The bedding observed in two of the borings performed by a previous geotechnical consultant dips between 43 and 64 degrees to the southwest. The observed bedding orientation is consistent with the geologic map by Dibblee (1991) and Lamar (1970), which show that the bedding should be expected to dip to the southwest, at angles ranging between 11 and 84 degrees. Based on this bedding orientation, southern or western-facing excavations conducted in bedrock are expected to expose daylighted bedrock. Northern or eastern-facing excavations conducted in bedrock are expected to expose favorable-oriented bedrock.

Groundwater was encountered in some of the borings performed by this firm, as well as previous geotechnical consultants. Based on review of the borings performed at the project site in the past 5 years, groundwater was encountered at depths ranging between 26 and 54 feet below the existing grade, which correspond to elevations ranging between 270 and 306.5 feet AMSL.

Temporary excavations are anticipated during the proposed building demolition and site grading. It is anticipated that the majority of these excavations will expose uncertified fill and native alluvial soils. However, if temporary excavations are performed near the central and eastern portion of the project site, these excavations may expose bedrock. All the on-site geologic materials, with the exception of daylighted bedrock, are suitable for vertical excavations up to five feet where not surcharged by adjacent traffic or structures. Vertical excavations performed in daylighted bedrock will require the use of temporary shoring.

Temporary vertical excavations conducted in uncertified fill materials, native alluvial soils, and favorable-oriented bedrock, which will be surcharged by traffic or the public right of way, may be conducted with the aid of slot cuts up to a maximum height of 6 feet. Vertical cuts in these

materials, which exceed a height of 6 feet, will require the installation of a temporary shoring system.

Temporary excavations which will expose fill materials, native alluvial soils, or favorable-oriented bedrock may be performed at a 1:1 slope gradient up to a heigh of 15 feet, and at a 1½:1 (horizontal:vertical) up to a height of 25 feet. Temporary excavations in daylighted bedrock may not be made steeper than the angle of bedding, which could vary between 11 and 84 degrees.

If miscellaneous permanent cuts are required as part of the proposed grading, it is recommended that these permanent cuts have a gradient which is not steeper than a 2:1 (horizontal: vertical). This recommendation is limited to cuts which do not exceed a vertical height of 10 feet, and which expose fill materials, native alluvial soils, or favorable-oriented bedrock. Permanent cuts in daylighted bedrock are not recommended. It is recommended that erosion protection is installed on the surface of these permanent cuts. Erosion protection could consist of select vegetation. Uncontrolled water runoff shall not be allowed over the surface of the sloped embankments.

Where a building is to be demolished, it is recommended that the entire structure be removed. This includes retaining walls, slabs-on-grade and foundation elements. It is our understanding that some of the existing buildings may be supported on deep foundation systems, such as piles, and removing the entire pile system may not be feasible. But as a minimum, it is recommended that all pile caps are removed. In addition, it is recommended that the piles are removed to a depth where they will not interfere with the proposed grading and/or new foundation systems. In areas where new buildings are proposed, the depth of removal should be analyzed on a case-by-case basis. For areas where new buildings are not anticipated, it is recommended that the piles are removed to a minimum depth of 3 feet below the proposed subgrade.

If miscellaneous outdoor flatwork and paving are required for the proposed project, it is recommended that the flatwork or paving sections be supported on competent alluvial soils, or bedrock. Where fill materials are be exposed at the subgrade of the flatwork or paving sections, it

is recommended that the existing fill be removed and recompacted to a minimum depth of 2 feet below the bottom of the flatwork or paving sections.

The validity of the conclusions and design recommendations presented herein is dependent upon review of the geotechnical aspects of the proposed construction by this firm. The subsurface conditions described herein have been projected from excavations on the site as indicated and should in no way be construed to reflect any variations which may occur between these excavations or which may result from changes in subsurface conditions. Any changes in the design, as outlined in this report, should be reviewed by this office. The recommendations contained herein should not be considered valid until reviewed and modified or reaffirmed subsequent to such review.

Los Angeles County Code Sections 110 and 111

The following statement is made in regard to Los Angeles County Code Sections 110 and 111. It is the opinion of the undersigned based on the findings of this assessment that provided the recommendations presented in this report are followed, the proposed development will be safe for its intended use against hazard from landsliding, settlement or slippage. The proposed development will have no adverse effect on the stability of the site of adjoining properties.

EXPANSIVE SOILS

Based on a review of the Expansion Index obtained from representative soil samples, the onsite geologic materials range from the moderate to the high expansion range. The Expansion Index was found to be between 50 and 99 for bulk samples representative of the site soils.

SOIL CORROSION POTENTIAL

The results of the soil corrosivity testing performed on samples representative of the upper onsite soils by HDR indicate that the electrical resistivities of the soils are corrosive to severely corrosive to general metals when saturated. The soil pH value of the samples was between 7.1 and 7.4. This

pH level was determined to be neutral to mildly alkaline. These values do not particularly increase soil corrosivity.

Soluble salt contents ranged from low to moderate. Nitrates concentrations were low, and Ammonium was not detected.

Per ACI 318-14, Table 19.3.1.1, sulfate levels in the samples are categorized as S0 and are negligible for corrosion of metals and cement. Per ACI 318-14 Table 19.3.2.1 any type of cement can be used.

In conclusion, the soil tested is classified as severely corrosive to ferrous metals and negligible for sulfate attack on concrete. Detailed results, discussion of results and recommended mitigating measures are provided within the enclosed Soils Corrosivity Study prepared by HDR, dated August 12, 2024.

GRADING GUIDELINES

The following guidelines are provided to aid in the miscellaneous site grading anticipated for this first phase of the project. Please note these guidelines are not intended for the preparation of future building pads.

Site Preparation

- A thorough search should be made for possible underground utilities and/or structures. Any existing or abandoned utilities or structures located within the footprint of the proposed grading should be removed or relocated as appropriate.

- All vegetation, and soft or disturbed geologic materials should be removed from the areas to receive controlled fill.

- Subsequent to the indicated removals, the exposed grade shall be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted in excess of the minimum required comparative density.

- The excavated areas shall be observed by the geotechnical engineer prior to placing compacted fill.

Compaction

All fill should be mechanically compacted in layers not more than 8 inches thick. Based on the moderate to high expansion potential of the on-site soils, the materials placed should be moisture conditioned to approximately 5 percent over the optimum moisture content of the particular material placed.

All fill shall be compacted to at least 90 percent of the maximum laboratory density for the materials used. The maximum density shall be determined by the laboratory operated by Geotechnologies, Inc. in general accordance with the most recent revision of ASTM D 1557.

Field observation and testing shall be performed by a representative of the geotechnical engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until a minimum of 90 percent compaction is obtained.

Acceptable Materials

The excavated onsite materials are considered satisfactory for reuse in the engineered fills as long as any deleterious debris (glass, metal, wood, etc.) and/or organic matter is removed. Non-deleterious debris, such as concrete, brick and asphalt, may be reused as controlled fill, as long as the particles are less than 2 inches in dimension, and their amount does not exceed 20 percent of the fill composition.

Any imported materials shall be observed and tested by the representative of the geotechnical engineer prior to use in fill areas. Imported materials should contain sufficient fines so as to be relatively impermeable and result in a stable subgrade when compacted. Any required import materials should consist of geologic materials with an expansion index of less than 50. The water-soluble sulfate content of the import materials should be less than 0.1% percentage by weight. Cobbles within the import materials should not exceed 6 inches in dimension.

Imported materials should be free from chemical or organic substances which could affect the proposed development. A competent professional should be retained in order to test imported materials and address environmental issues and organic substances which might affect the proposed development.

Utility Trench Backfill

Utility trenches should be backfilled with controlled fill. The utility should be bedded with clean sands at least one foot over the crown. The remainder of the backfill may be onsite soil compacted to 90 percent of the laboratory maximum density. Utility trench backfill should be tested by representatives of this firm in general accordance with the most recent revision of ASTM D 1557.

Wet Soils

At the time of exploration, some of the soils expected to be recompacted were locally above optimum moisture content. It is anticipated that some of the materials to be exposed at the bottom of excavated plane may require some level of drying and aeration prior to recompaction, particularly if the grading is conducted during the rainy season. Similarly, the excavated material to be placed as compacted fill may require blending and/or aeration to reduce its overall moisture content prior to recompaction.

If pumping (yielding or vertical deflection) is experienced during operation of heavy equipment, angular minimum ¾-inch gravel should be placed and worked into the subgrade. The exact thickness of the gravel would be a trial and error procedure, and would be determined in the field.

The gravel will help to densify the subgrade as well as function as a stabilization material upon which heavy equipment may operate. It is not recommended that rubber tire construction equipment attempt to operate directly on the pumping subgrade soils prior to placing the gravel. Direct operation of rubber tire equipment on the soft subgrade soils will likely result in excessive disturbance to the soils, which in turn will result in a delay to the construction schedule since those disturbed soils would then have to be removed and properly recompacted. Extreme care should be utilized to place gravel as the subgrade becomes exposed.

Shrinkage

Shrinkage results when a volume of soil removed at one density is compacted to a higher density. A shrinkage factor between 5 and 15 percent should be anticipated when excavating and recompacting the existing fill and underlying native geologic materials on the site to an average comparative compaction of 92 percent.

Weather Related Grading Considerations

When rain is forecast all fill that has been spread and awaits compaction shall be properly compacted prior to stopping work for the day or prior to stopping due to inclement weather. These fills, once compacted, shall have the surface sloped to drain to an area where water can be removed.

Temporary drainage devices should be installed to collect and transfer excess water to the street in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope.

Work may start again, after a period of rainfall, once the site has been reviewed by a representative of this office. Any soils saturated by the rain shall be removed and aerated so that the moisture content will fall within three percent of the optimum moisture content.

Surface materials previously compacted before the rain shall be scarified, brought to the proper moisture content and recompacted prior to placing additional fill, if considered necessary by a representative of this firm.

Geotechnical Observations and Testing During Grading

Geotechnical observations and testing during grading are considered to be a continuation of the geotechnical investigation. It is critical that the geotechnical aspects of the project be reviewed by representatives of Geotechnologies, Inc. during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise this office at least twenty-four hours prior to any required site visit.

Proper compaction is necessary to reduce settlement of overlying improvements. Some settlement of compacted fill should be anticipated. Any utilities supported therein should be designed to accept differential settlement.

LEED Considerations

The Leadership in Energy and Environmental Design (LEED) Green Building Rating System encourages adoption of sustainable green building and development practices. Credit for LEED Certification can be assigned for reuse of construction waste and diversion of materials from landfills in new construction.

In an effort to provide the design team with a viable option in this regard, demolition debris could be crushed onsite in order to use it in the ongoing grading operations. The environmental ramifications of this option, if any, should be considered by the team.

The demolition debris should be limited to concrete, asphalt and other non-deleterious materials. All deleterious materials should be removed including, but not limited to, paper, garbage, ceramic materials and wood.

For structural fill applications, the materials should be crushed to 2-inch in maximum dimension or smaller. The crushed materials should be thoroughly blended and mixed with onsite soils prior to placement as compacted fill. The amount of crushed material should not exceed 20 percent. The blended and mixed materials should be tested by this office prior to placement to ensure they are suitable for compaction purposes. The blended and mixed materials should be tested by Geotechnologies, Inc. during placement to ensure that they have been compacted in a suitable manner.

PERMANENT CUTS

If miscellaneous permanent cuts are required as part of the proposed grading, it is recommended that these permanent cuts have a gradient which is not steeper than a 2:1 (horizontal: vertical). This recommendation is limited to cuts which do not exceed a vertical height of 10 feet, and which expose fill materials, native alluvial soils, or favorable-oriented bedrock. Permanent cuts in daylighted bedrock are not recommended. It is recommended that erosion protection is installed on the surface of these permanent cuts. Erosion protection could consist of select vegetation. Uncontrolled water runoff shall not be allowed over the surface of the sloped embankments.

TEMPORARY EXCAVATIONS

It is anticipated that the majority of the anticipated temporary excavations will expose uncertified fill and native alluvial soils. However, if temporary excavations are performed near the central portion of the project site, these excavations may expose bedrock

The on-site uncertified fill materials, native alluvial soils, and favorable-oriented bedrock are suitable for vertical excavations up to 5 feet where not surcharged by adjacent traffic, structures, or the public right of way. Vertical excavations performed in daylighted bedrock will require the use of temporary shoring. As mentioned before in this report, daylighted bedrock is anticipated for excavations into bedrock which are southern or western-facing.

Temporary vertical excavations conducted in uncertified fill materials, native alluvial soils, and favorable-oriented bedrock, which will be surcharged by traffic or the public right of way, may be conducted with the aid of slot cuts up to a maximum height of 6 feet, as recommended in the following section. Vertical cuts in these materials, which exceed a height of 6 feet, will require the installation of temporary shoring.

Where sufficient space is available, temporary excavations which will expose fill materials, native alluvial soils, or favorable-oriented bedrock may be performed at a 1:1 slope gradient up to a heigh of 15 feet, and at a 1½:1 (horizontal:vertical) up to a height of 25 feet. Temporary excavations in daylighted bedrock may not be made steeper than the angle of bedding, which could vary between 11 and 84 degrees.

Where sloped embankments are utilized, the tops of the slopes should be barricaded to prevent vehicles and storage loads near the top of slope within a horizontal distance equal to the depth of the excavation. If the temporary construction embankments are to be maintained during the rainy season, berms are strongly recommended along the tops of the slopes to prevent runoff water from

entering the excavation and eroding the slope faces. Water should not be allowed to pond on top of the excavation nor to flow towards it.

Slot Cutting

In order to maintain a stable vertical excavation, the slot cutting method may be utilized where temporary excavations exposing uncertified fill, native alluvial soils, or favorable-oriented bedrock, will be surcharged by vehicular traffic or the public right of way. The "A-B-C" slot-cutting procedure is recommended. The maximum height of the slots is 6 feet.

The slot cutting method employs the earth as a buttress and allows the earth excavation to proceed in phases. The initial excavation consists of excavating the "A" slots. Alternate "A" slots of 8 feet may be worked. The remaining earth buttresses ("B" and "C" slots) should be 16 feet in width. The "A" slots should be properly backfilled, before the "B" slots are excavated. The height of the slots shall not exceed 6 feet. Calculations indicating that slots 8 feet in width will be stable for the maximum recommended height of 6 feet have been included in the appendix of this report. This calculation includes a conservative surcharge load to account for adjacent vehicular traffic.

Excavation Observations

It is critical that the soils exposed in the cut slopes are observed by a representative of Geotechnologies, Inc. during excavation so that modifications of the slopes can be made if variations in the geologic material conditions occur. Many building officials require that temporary excavations should be made during the continuous observations of the geotechnical engineer. All excavations should be stabilized within 30 days of initial excavation.

SHORING DESIGN

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

The recommended method of shoring consists of steel soldier piles, placed in drilled holes and backfilled with concrete. As discussed below vibrating methods of shoring installation may also be utilized. The soldier piles may be designed as cantilevers or laterally braced utilizing drilled tie-back anchors.

These recommendations are considered preliminary due to the variable nature of the project site subsurface conditions. These recommendations must be re-evaluated once the location of a temporary shoring system is determined.

Soldier Piles – Drilled and Poured

Drilled cast-in-place soldier piles should be placed no closer than two diameters on center. The minimum diameter of the piles is 18 inches. Structural concrete should be used for the soldier piles below the excavation; lean-mix concrete may be employed above that level. As an alternative, lean-mix concrete may be used throughout the pile where the reinforcing consists of a wideflange section. The slurry must be of sufficient strength to impart the lateral bearing pressure developed by the wideflange section to the geologic materials. For design purposes, an allowable passive value for the geologic materials below the bottom plane of excavation, may be assumed to be 400 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 geologic materials.

The frictional resistance between the soldier piles and retained geologic material may be used to resist the vertical component of the anchor load. The coefficient of friction may be taken as 0.3 based on uniform contact between the steel beam and lean-mix concrete and retained earth. The portion of soldier piles below the plane of excavation may also be employed to resist the downward loads. The downward capacity may be determined using a frictional resistance of 400 pounds per square foot. The minimum depth of embedment for shoring piles is 5 feet below the bottom of the footing excavation or 7 feet below the bottom of excavated plane whichever is deeper.

Groundwater was encountered during exploration at depths between 26 and 54 feet below the existing grade. If the piles will extend into the groundwater level, caving of the saturated sandy materials below the groundwater level may occur during drilling of piles. Casing or polymer drilling fluid will most likely be required during drilling in order to maintain open shafts. 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.

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

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

Soldier Piles – Vibrated

The vibration method of shoring pile installation is acceptable to this firm from a geotechnical standpoint, provided the recommendations presented herein are implemented. When using the vibration method of installing the soldier beams, the minimum embedment depth shall be 10 feet below the lowest excavated plane. The available passive resistance of the pile may be determined using the diagonal length from the outer edges of opposite flange sections.

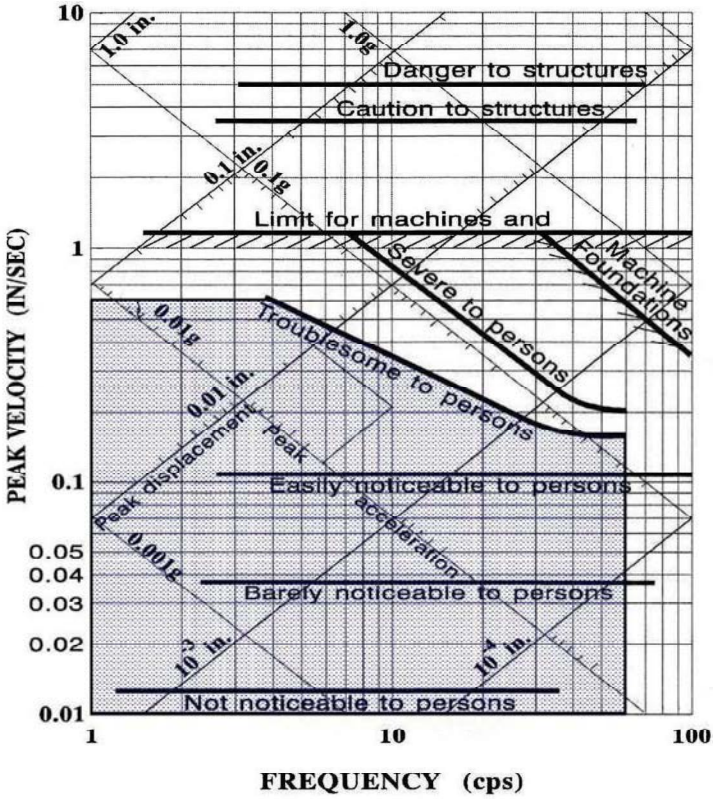
If predrilling is required, it is recommended that the diameter of the predrilled holes should not exceed 75 percent of the depth of the web of the I-beam. The depth of the predrilled holes should not exceed the planned excavation depth. In addition, when predrilling, the auger shall be backspun out of the pilot holes, leaving the soils in place. All shoring (predrilling, installation of shoring piles, tieback installation and testing, and lagging) shall be performed under the continuous inspections by a deputy grading inspector of this firm.

It should be noted that Bedrock of the Puente Formation was encountered at some areas of the site. The shoring designer and contractor should be aware that, where the soldier beams will extend into this moderately hard to hard layer, predrilling will not be permitted at this depth. Vibrating soldier beams into the undisturbed bedrock may be challenging.

The allowable level of vibration that results from the installation of the piles should not exceed a threshold where occupants of the nearby structures are disturbed, despite higher vibration tolerances that a building may endure without deformation. There is a relationship between particle

velocity and vibration frequency that will occur due to the installation. A range of tolerable particle peak velocity and frequency of vibration is provided below. The shaded area on the graph is considered within acceptable limits to avoid damage to nearby structures. The acceptable limits should be measured at the neighboring structures.

The vibrations should be monitored with a seismograph during pile installation to detect the magnitude of vibration and oscillation experienced by the adjacent structure. The results should be recorded and provided to the owner. If, during installation, the vibrations exceed the range shown on the graph below, the shoring contractor should modify the installation procedure to reduce the values to the acceptable range.



Given Velocity = 0.2 inch/sec.
Frequency = 10 cps
Then from Graph, Displacement = 0.003 inches
Acceleration = 0.03g
Motion is easy noticeable or troublesome to persons

NOTE: Shaded area considered below threshold for structure damage

REFERENCE: Department of Defense, 1997, Soil Dynamics and Special Design Aspects, MIL-HDBK-1007/3

Lagging

Soldier piles and anchors should be designed for the full anticipated pressures. Due to arching in the geologic materials, the pressure on the lagging will be less. It is recommended that the lagging should be designed for the full design pressure but may be limited to a maximum of 400 pounds per square foot. It is recommended that a representative of this firm observe the installation of lagging to insure uniform support of the excavated embankment.

Lateral Pressures

It is anticipated that the majority of the temporary excavations will expose uncertified fill and native alluvial soils. However, if temporary excavations are performed near the central portion of the project site, these excavations may expose bedrock. Where bedrock is exposed, shoring wall loads will be affected by the orientation of bedrock bedding. The bedding is expected to dip to the southwest at angles ranging between 11 and 84 degrees. This daylighted bedding orientation is expected to occur where bedrock is exposed in excavations that will have a southern or western-facing component.

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 the diagram below. Equivalent fluid pressures for the design of cantilevered and restrained shoring are presented in the following table for all different materials expected at the project site:

LATERAL SHORING WALL PRESSURES FOR BEDROCK WITH DAYLIGHTED BEDDING ORIENTATION		
Height of Shoring Wall (feet)	Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
Up to 10	35	22H
10 to 15	47	30H
15 to 20	55	35H

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

LATERAL SHORING WALL PRESSURES FOR FILL MATERIALS		
Height of Shoring Wall (feet)	Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
Up to 10	28	18H
10 to 15	30	19H
15 to 20	40	25H

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

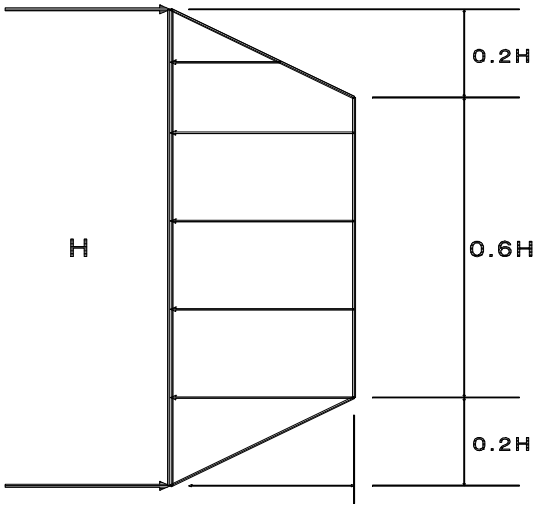
LATERAL SHORING WALL PRESSURES FOR ALLUVIAL SOILS		
Height of Shoring Wall (feet)	Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
Up to 15	28	18H
15 to 20	34	22H

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

LATERAL SHORING WALL PRESSURES FOR FAVORABLE-ORIENTED BEDROCK		
Height of Shoring Wall (feet)	Cantilever Shoring System Equivalent Fluid Pressure (pcf) Triangular Distribution of Pressure	Restrained Shoring System Lateral Earth Pressure (psf)* Trapezoidal Distribution of Pressure
Up to 20	28	18H

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

TRAPEZOIDAL DISTRIBUTION OF PRESSURE



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.

The upper ten feet of the shoring wall adjacent to streets, driveways or parking areas should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the walls due to normal street traffic. If the traffic is kept back at least ten feet from the retaining walls, the traffic surcharge may be neglected.

Tied-Back Anchors

Tied-back anchors may be used to resist lateral loads. Friction anchors 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.

Drilled friction anchors may be designed for a skin friction of 400 pounds per square foot. Pressure grouted anchors 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 applying the skin friction over the surface area of the bonded anchor shaft. The diameter of the bell may be utilized as the diameter of the bonded anchor shaft when determining the surface area. This implies that in order for the belled anchor to fail, the entire parallel soil column must also fail. 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 this test should not exceed 12 inches. The anchor deflection should not exceed 0.75 inches during the 24 hour period, measured after the 200 percent load has been 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 installed until satisfactory test results are obtained. The installation and testing of the anchors should be observed by the geotechnical engineer. Minor caving during drilling of the anchors should be anticipated.

Anchor Installation

Tied-back anchors may be installed between 20 and 45 degrees below the horizontal. Caving of the anchor shafts, particularly within sand deposits, should be anticipated and the following provisions should be implemented in order to minimize such caving. The anchor shafts should be filled with concrete by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. In order to minimize the chances of caving, it is recommended that the portion of the anchor shaft within the active wedge be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

Deflection

It is difficult to accurately predict the amount of deflection of a shored embankment. It should be realized that some deflection will occur. It is estimated that the deflection could be on the order of one 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 street 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.

It is recommended that the shoring deflection be limited to ½-inch at the top of the shored embankment where a structure is within a 1:1 (h:v) plane projected up from the base of the excavation. A maximum deflection of 1-inch has been allowed provided there are no structures within a 1:1 (h:v) plane drawn upward from the base of the excavation.

Monitoring

Because of the depth of the excavation, some means 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.

Some movement of the shored embankments should be anticipated as a result of the relatively deep excavation. It is recommended that photographs of the existing buildings on the adjacent properties be made during construction to record any movements for use in the event of a dispute.

Shoring Observations

It is critical that the installation of shoring is observed by a representative of Geotechnologies, Inc. Many building officials require that shoring installation should be performed during continuous observation of a representative of the geotechnical engineer. The observations insure that the recommendations of the geotechnical report are implemented and so that modifications of the recommendations can be made if variations in the geologic material or groundwater conditions warrant. The observations will allow for a report to be prepared on the installation of shoring for the use of the local building official, where necessary.

SLABS ON GRADE

Outdoor Flatwork

Outdoor concrete flatwork, not subject to vehicle traffic, should be a minimum of 5 inches in thickness. It is recommended that the outdoor flatwork sections are supported on a stable subgrade, consisting of recompacted fill materials, competent native alluvial soils, or bedrock. Where fill materials are exposed at the subgrade, it is recommended that the subgrade is removed and recompacted to a minimum depth of 2 feet below the bottom of the proposed flatwork section.

Outdoor flatwork should be reinforced with a minimum of #4 steel bars on 16-inch centers each way.

Concrete Crack Control

For standard control of concrete cracking, a maximum crack control joint spacing of 8 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. The crack control joints should be installed as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by a structural engineer.

PAVEMENTS

As a minimum, the subgrade of the proposed paving sections should consist of 2 feet of engineered fill materials. The materials should be recompacted to a minimum of 90 percent of the maximum dry density, as determined by the most recent revision of ASTM D 1557. The client should be aware that removal of all existing uncertified fill in the area of new paving is not required, however, pavement constructed in this manner will most likely have a shorter design life and increased maintenance costs. The following pavement sections are recommended:

Service	Asphalt Pavement Thickness Inches	Base Course Inches
Passenger Car Traffic	3	7
Moderate Truck Traffic	4	9
Heavy Truck Traffic	5	12

Concrete paving may also be used on the project. For passenger cars and moderate truck traffic, concrete paving should be 6 inches of concrete over 4 inches of compacted base. For heavy truck traffic, concrete paving should be 7½ inches of concrete over 4 inches of compacted base. For standard crack control maximum expansion joint spacing of 8 feet should not be exceeded. Lesser spacings would provide greater crack control. Joints at curves and angle points are recommended. Concrete paving should be reinforced with a minimum of #4 steel bars on 16-inch centers each way.

Aggregate base should be compacted to a minimum of 95 percent of the most recent revision of ASTM D 1557 laboratory maximum dry density. Base materials should conform to Sections 200-2.2 or 200-2.4 of the "Standard Specifications for Public Works Construction", (Green Book), latest edition.

The performance of pavement is highly dependent upon providing positive surface drainage away from the edges. Ponding of water on or adjacent to pavement can result in saturation of the subgrade materials and subsequent pavement distress. If planter islands are planned, the perimeter curb should extend a minimum of 12 inches below the bottom of the aggregate base.

SITE DRAINAGE

Proper surface drainage is critical to the future performance of the project. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the designed engineering properties. Proper site drainage should be maintained at all times.

All site drainage, with the exception of any required to disposed of onsite by stormwater regulations, should be collected and transferred to the street in non-erosive drainage devices. Discharge from downspouts, roof drains and scuppers should not be permitted on unprotected soils within five feet of a building perimeter. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters which are located within a distance equal to the depth of a retaining wall should be sealed to prevent moisture adversely affecting the wall. Planters which are located within five feet of a foundation should be sealed to prevent moisture affecting the earth materials supporting the foundation.

STORMWATER DISPOSAL

It is the opinion of this firm that on-site stormwater infiltration is not feasible at the project site. Deep uncertified fill materials were encountered throughout the site. Infiltrating into fill materials is not acceptable. Additionally, the native alluvial soils consist predominantly of silts and clays, which are considered to be relatively impervious. Similarly, the underlying Puente Formation Bedrock encountered within the central portion of the site is considered to be relatively impervious. Allowing the infiltration of stormwater into these geologic materials would most likely result in a perched water condition. Additionally, the upper on-site materials have a moderate to high expansion index.

Based on the above considerations, it is the opinion of Geotechnologies, Inc, that the disposal of stormwater by infiltration into the site soils is not recommended at the project site. Where percolation of stormwater into the subgrade soils is not advisable, most Building Officials have allowed the stormwater to be filtered through soils in planter areas. Once the water has been filtered through a planter it may be released into the storm drain system. It is recommended that overflow pipes are incorporated into the design of the discharge system in the planters to prevent flooding. In addition, the planters shall be sealed and waterproofed to prevent leakage. Please be advised that adverse impact to landscaping and periodic maintenance may result due to excessive water and contaminants discharged into the planters.

It is recommended that the design team (including the structural engineer, waterproofing consultant, plumbing engineer, and landscape architect) be consulted in regards to the design and construction of filtration systems.

DESIGN REVIEW

Engineering of the proposed project should not begin until approval of the geotechnical report by the Building Official is obtained in writing. Significant changes in the geotechnical recommendations may result during the building department review process.

It is recommended that the geotechnical aspects of the project be reviewed by this firm during the design process. This review provides assistance to the design team by providing specific recommendations for particular cases, as well as review of the proposed construction to evaluate whether the intent of the recommendations presented herein are satisfied.

CONSTRUCTION MONITORING

Geotechnical observations and testing during construction are considered to be a continuation of the geotechnical investigation. It is critical that this firm review the geotechnical aspects of the project during the construction process. Compliance with the design concepts, specifications or recommendations during construction requires review by this firm during the course of construction. All foundations should be observed by a representative of this firm prior to placing concrete or steel. Any fill which is placed should be observed, tested, and verified if used for engineered purposes. Please advise Geotechnologies, Inc. at least twenty-four hours prior to any required site visit.

If conditions encountered during construction appear to differ from those disclosed herein, notify Geotechnologies, Inc. immediately so the need for modifications may be considered in a timely manner.

It is the responsibility of the contractor to ensure that all excavations and trenches are properly sloped or shored. All temporary excavations should be cut and maintained in accordance with applicable OSHA rules and regulations.

EXCAVATION CHARACTERISTICS

The exploration performed for this investigation is limited to the geotechnical excavations described. Direct exploration of the entire site would not be economically feasible. The owner, design team and contractor must understand that differing excavation and drilling conditions may be encountered based on boulders, gravel, oversize materials, groundwater and many other conditions. Fill materials, especially when they were placed without benefit of modern grading codes, regularly contain materials which could impede efficient grading and drilling. Southern California sedimentary bedrock is known to contain variable layers which reflect differences in depositional environment. Such layers may include abundant gravel, cobbles and boulders. Similarly bedrock can contain concretions. Concretions are typically lenticular and follow the bedding. They are formed by mineral deposits. Concretions can be very hard. Excavation and drilling in these areas may require full size equipment and coring capability. The contractor should be familiar with the site and the geologic materials in the vicinity.

CLOSURE AND LIMITATIONS

The purpose of this report is to aid in the design and completion of the described project. Implementation of the advice presented in this report is intended to reduce certain risks associated with construction projects. The professional opinions and geotechnical advice contained in this report are sought because of special skill in engineering and geology and were prepared in accordance with generally accepted geotechnical engineering practice. Geotechnologies, Inc. has a duty to exercise the ordinary skill and competence of members of the engineering profession. Those who hire Geotechnologies, Inc. are not justified in expecting infallibility, but can expect reasonable professional care and competence.

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the geologic conditions do not deviate from those disclosed in the investigation. If any variations are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geotechnologies, Inc. should be notified so that supplemental recommendations can be prepared.

This report is issued with the understanding that it is the responsibility of the owner, or the owner's representatives, to ensure that the information and recommendations contained herein are brought to the attention of the project architect and engineer and are incorporated into the plans. The owner is also responsible to see that the contractor and subcontractors carry out the geotechnical recommendations during construction.

The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside control of this firm. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Geotechnical observations and testing during construction is considered to be a continuation of the geotechnical investigation. It is, therefore, most prudent to employ the consultant performing the initial investigative work to provide observation and testing services during construction. This practice enables the project to flow smoothly from the planning stages through to completion.

Should another geotechnical firm be selected to provide the testing and observation services during construction, that firm should prepare a letter indicating their assumption of the responsibilities of geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for review. The letter should acknowledge the concurrence of the new geotechnical engineer with the recommendations presented in this report.



EXCLUSIONS

Geotechnologies, Inc. does not practice in the fields of methane gas, radon gas, environmental engineering, waterproofing, dewatering, organic substances or the presence of corrosive soils or wetlands which could affect the proposed development including mold and toxic mold. Nothing in this report is intended to address these issues and/or their potential effect on the proposed development. A competent professional consultant should be retained in order to address environmental issues, waterproofing, organic substances and wetlands which might affect the proposed development.

GEOTECHNICAL TESTING

Classification and Sampling

The soil is continuously logged by a representative of this firm and classified by visual examination in accordance with the Unified Soil Classification system. The field classification is verified in the laboratory, also in accordance with the Unified Soil Classification System. Laboratory classification may include visual examination, Atterberg Limit Tests and grain size distribution. The final classification is shown on the excavation logs.

Samples of the geologic materials encountered in the exploratory excavations were collected and transported to the laboratory. Undisturbed samples of soil are obtained at frequent intervals. Unless noted on the excavation logs as an SPT sample, samples acquired while utilizing a hollow-stem auger drill rig are obtained by driving a thin-walled, California Modified Sampler with successive 30-inch drops of a 140-pound automatic-trip hammer. The soil is retained in brass rings of 2.50 inches outside diameter and 1.00 inch in height. The central portion of the samples are stored in close fitting, waterproof containers for transportation to the laboratory. Samples noted on the excavation logs as SPT samples are obtained in general accordance with the most recent revision of ASTM D 1586. Samples are retained for 30 days after the date of the geotechnical report.

Moisture and Density Relationships

The field moisture content and dry unit weight are determined for each of the undisturbed soil samples, and the moisture content is determined for SPT samples in general accordance with the most recent revision of ASTM D 4959 or ASTM D 4643. This information is useful in providing a gross picture of the soil consistency between exploration locations and any local variations. The dry unit weight is determined in pounds per cubic foot and shown on the “Excavation Logs”, A-Plates. The field moisture content is determined as a percentage of the dry unit weight.

Direct Shear Testing

Shear tests are performed in general accordance with the most recent revision of ASTM D 3080 with a strain controlled, direct shear machine manufactured by Soil Test, Inc. or a Direct Shear Apparatus manufactured by GeoMatic, Inc. The rate of deformation ranges between approximately 0.005 and 0.025 inches per minute. Each sample is sheared under varying confining pressures in order to determine the Mohr-Coulomb shear strength parameters of the cohesion intercept and the angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagram," B-Plates.

The most recent revision of ASTM 3080 limits the particle size to 10 percent of the diameter of the direct shear test specimen. The sheared sample is inspected by the laboratory technician running the test. The inspection is performed by splitting the sample along the sheared plane and observing the soils exposed on both sides. Where oversize particles are observed in the shear plane, the results are discarded and the test run again with a fresh sample.

Consolidation Testing

Settlement predictions of the soil's behavior under load are made on the basis of the consolidation tests in general accordance with the most recent revision of ASTM D 2435. The consolidation apparatus is designed to receive a single one-inch high ring. Loads are applied in several increments in a geometric progression, and the resulting deformations are recorded at selected time intervals. Porous stones are placed in contact with the top and bottom of each specimen to permit addition and release of pore fluid. Samples are generally tested at increased moisture content to determine the effects of water on the bearing soil. The normal pressure at which the water is added is noted on the drawing. Results are plotted on the "Consolidation Test," C-Plates.

Expansion Index Testing

The expansion tests performed on the remolded samples are in accordance with the Expansion Index testing procedures, as described in the most recent revision of ASTM D 4829. The soil sample is compacted into a metal ring at a saturation degree of 50 percent. The ring sample is then placed in a consolidometer, under a vertical confining pressure of 1 lbf/square inch and inundated with distilled water. The deformation of the specimen is recorded for a period of 24 hour or until the rate of deformation becomes less than 0.0002 inches/hour, whichever occurs first. The expansion index, EI, is determined by dividing the difference between final and initial height of the ring sample by the initial height, and multiplied by 1,000. Results are presented on Plate D of this report.

Laboratory Compaction Characteristics

The maximum dry unit weight and optimum moisture content of a soil are determined in general accordance with the most recent revision of ASTM D 1557. A soil at a selected moisture content is placed in five layers into a mold of given dimensions, with each layer compacted by 25 blows of a 10 pound hammer dropped from a distance of 18 inches subjecting the soil to a total

compactive effort of about 56,000 pounds per cubic foot. The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of moisture contents to establish a relationship between the dry unit weight and the water content of the soil. The data when plotted represent a curvilinear relationship known as the compaction curve. The values of optimum moisture content and modified maximum dry unit weight are determined from the compaction curve. Results are presented on Plate D of this report.

Grain Size Distribution

Sieve analysis, ASTM D6913, is used to determine the grain size distribution of the soil larger than the Number 200 sieve. The grain size distributions are plotted on the E-Plates presented in the Appendix of this report.

Atterberg Limits

Depending on their moisture content, cohesive soils can be solid, plastic, or liquid. The water contents corresponding to the transitions from solid to plastic or plastic to liquid are known as the Atterberg Limits. The transitions are called the plastic limit and liquid limit. The difference between the liquid and plastic limits is known as the plasticity index. ASTM D 4318 is utilized to determine the Atterberg Limits. The results are shown on the enclosed F-Plate.

REFERENCES

AECOM, October 21, 2019, Geotechnical Investigation and Bulk Factor, Proposed Women's and Children Hospital Demolition, LAC+USC Medical Center, 1240 North Mission Road, Los Angeles California, Project No. 60595699.

California Division of Mines and Geology, 1998, Seismic Hazard Evaluation Report for the Los Angeles 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 029.

California Division of Mines and Geology, 1999, Seismic Hazard Zone Map for the Los Angeles 7.5-Minute quadrangle, map scale 1:24,000.

Dames & Moore, December 13, 1993, Geotechnical Investigation Report for LAC+USC Medical Center Replacement Project and Marengo Parking Structure, Job No. 00499-038-015.

Dibblee, T.W., 1989, Geologic Map of the Los Angeles Hollywood and Burbank (South ½) 7.5-Minute Quadrangles, Map No DF-22, map scale 1:24,000.

Dolan, J.F., Sieh, K., Rockwell, T.K., Guptill, P., and Miller, G., 1997, Active Tectonics, Paleoseismology, and Seismic Hazards of the Hollywood Fault, Northern Los Angeles Basin, California, GSA Bulletin, v. 109: no 12, p1595-1616.

Geocon West, Inc., April 9, 2018, Geotechnical Investigation, Proposed Health Agency Recuperative Campus, 1774 Zonal Avenue, Los Angeles, California, Project No. A9733-06-01.

Geocon West, Inc., May 20, 2019, Preliminary Geotechnical Investigation, Proposed Health Agency Recuperative Campus, 1774 Zonal Avenue, Los Angeles, California, Project No. A8559-06-74.

Geotechnical Professionals, Inc., November 7, 2022, Report of Geotechnical Investigation for Structural Condition Assessment and Seismic Evaluation LAC+USC Medical Center - Old Pharmacy Warehouse Project, 1100 N. Mission Road, Los Angeles, California, Project No. 3080.221.

Geotechnologies, Inc., November 5, 2024, Geotechnical Engineering Investigation, Retrofit of the Existing Los Angeles County General Hospital Tunnel Structure, 1100 N. Mission Road, Los Angeles, California, File No. 22579.

Geotechnologies, Inc., February 7, 2024, Geotechnical Engineering Investigation, Proposed L.A. County Mental Health Urgent Care Clinic & Residential Withdrawal Management Facility, 1240 North Mission Road, Los Angeles, California, File No. 22453.

REFERENCES - continued

Geotechnologies, Inc., September 1, 2021, Proposed LAC + USC Steam Line and Boiler, North Mission Road and Zonal Avenue, Los Angeles, California, File No. 22146.

GPI, October 21, 2021, Revised October 20, 2021, Report of Geotechnical Observations and Testing, Proposed Women's and Children Hospital Demolition, LAC+USC Medical Center, 1240 North Mission Road, Los Angeles California, Project No. 3062.C.

Hart, E.W. and Bryant, W.A., 2007, Fault Rupture Zones in California, Division of Mines and Geology, Special Publication 42, 25pp.

Lamar, D.L. 1970, Geology of the Elysian Park-Repetto Hills Area, Los Angeles Count, California, California division of Mines and Geology Special Report 101, 45pp, map scale 1:24,000.

Leighton and Associates, Inc. (1990), Technical Appendix to the Safety Element of the Los Angeles County General Plan: Hazard Reduction in Los Angeles County.

LeRoy Crandall & Assoc., September 30, 1960, Report of Foundation Investigation, Proposed Refrigeration Plant and Tunnel, Marengo and Lord Streets, Job No. 60383.

LeRoy Crandall & Assoc., February 11, 1957, Report of Foundation Investigation, Proposed Nurses Residence Wing, Brittania Street near Marengo Street, Los Angeles, California, Job No. 56356.

L.T. Evans, September 8, 1950, Report of Foundation Investigation for Nurses Residence, Los Angeles General Hospital, Los Angeles, California.

L.T. Evans, August 2, 1950, Report of Foundation Investigation for Communicable Diseases Building, Los Angeles General Hospital, Los Angeles, California.

L.T. Evans, July 27, 1948, Report of Foundation Investigation for Psychopathic Building, Los Angeles General Hospital, Los Angeles, California.

RMA Group, March 30, 2018, Geotechnical Investigation for Proposed L.A. Co. - USC Medical Center Child Care Project, N. State Street between Zonal Avenue and Marengo Street, Los Angeles, Ca., Job No. 18-0056-01.

SEAOC/OSHPD U.S. Seismic Design Maps tool.

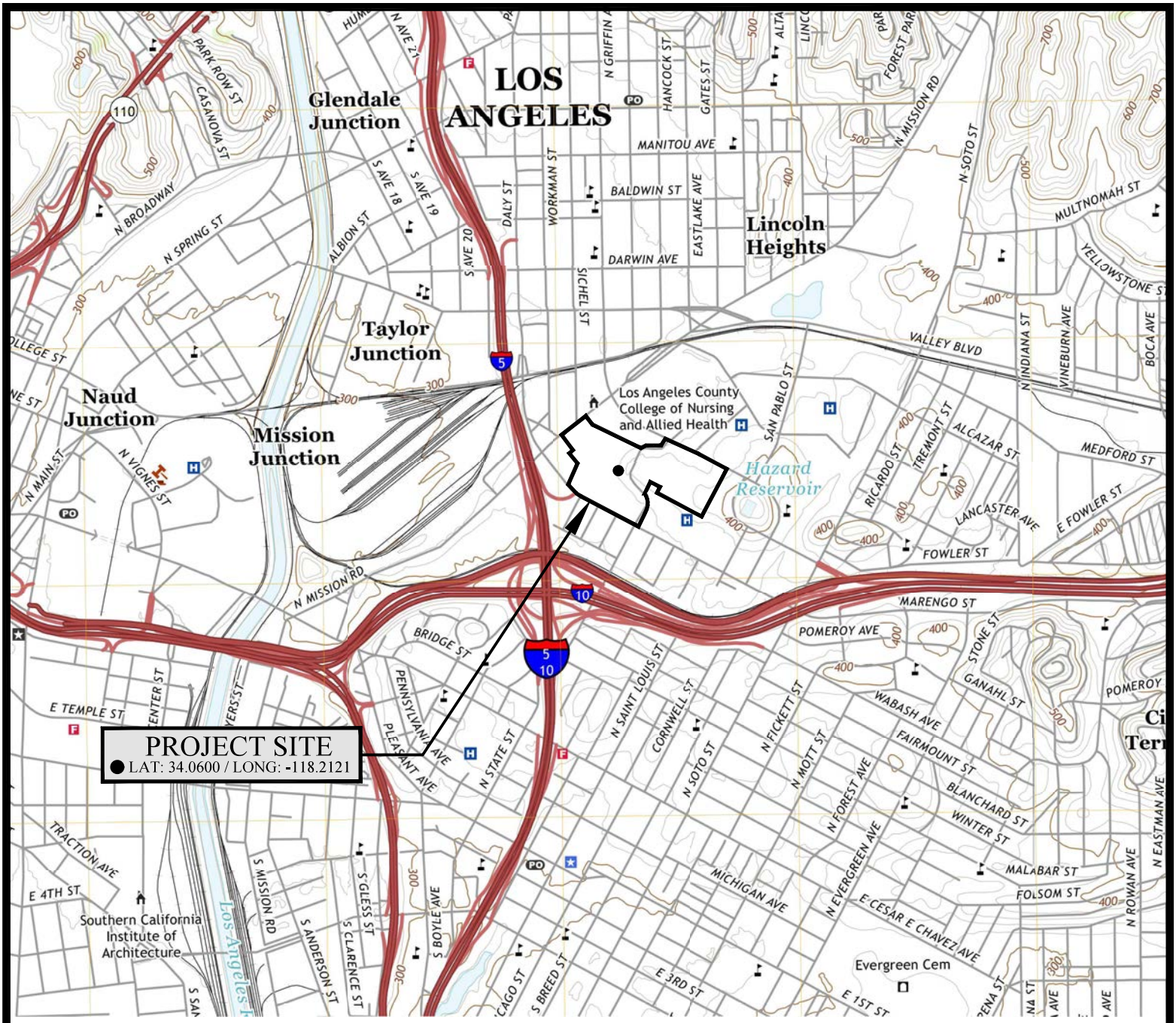
Shaw, J.H. and Suppe, J., 1996, Earthquake Hazards of Active Blind Thrust Fault Under the Central Los Angeles Basin, California Journal of Geophysical Research, Vol. 101, No. 4, pages 8623-8642.

REFERENCES - continued

Tokimatsu, K. and Seed, H. B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8.

Yerkes, R.F., et al. 1977, Geologic Aspects of Tunneling in the Los Angeles Area, U.S. Geological Survey.

Yerkes, R.F., et al. 1965, Geology of the Los Angeles, Basin, California - An Introduction, U.S. Geological Survey Professional Paper 420-A.



1 0.5 0 KILOMETERS 1 2

1000 500 0 METERS 1000 2000

1 0.5 0 1

MILES

1000 0 1000 2000 3000 4000 5000 6000 7000 8000 9000 10000

FEET

N

REFERENCE: U.S.G.S. TOPOGRAPHIC MAPS, 7.5 - MINUTE SERIES LOS ANGELES, CA QUADRANGLE 2022

VICINITY MAP

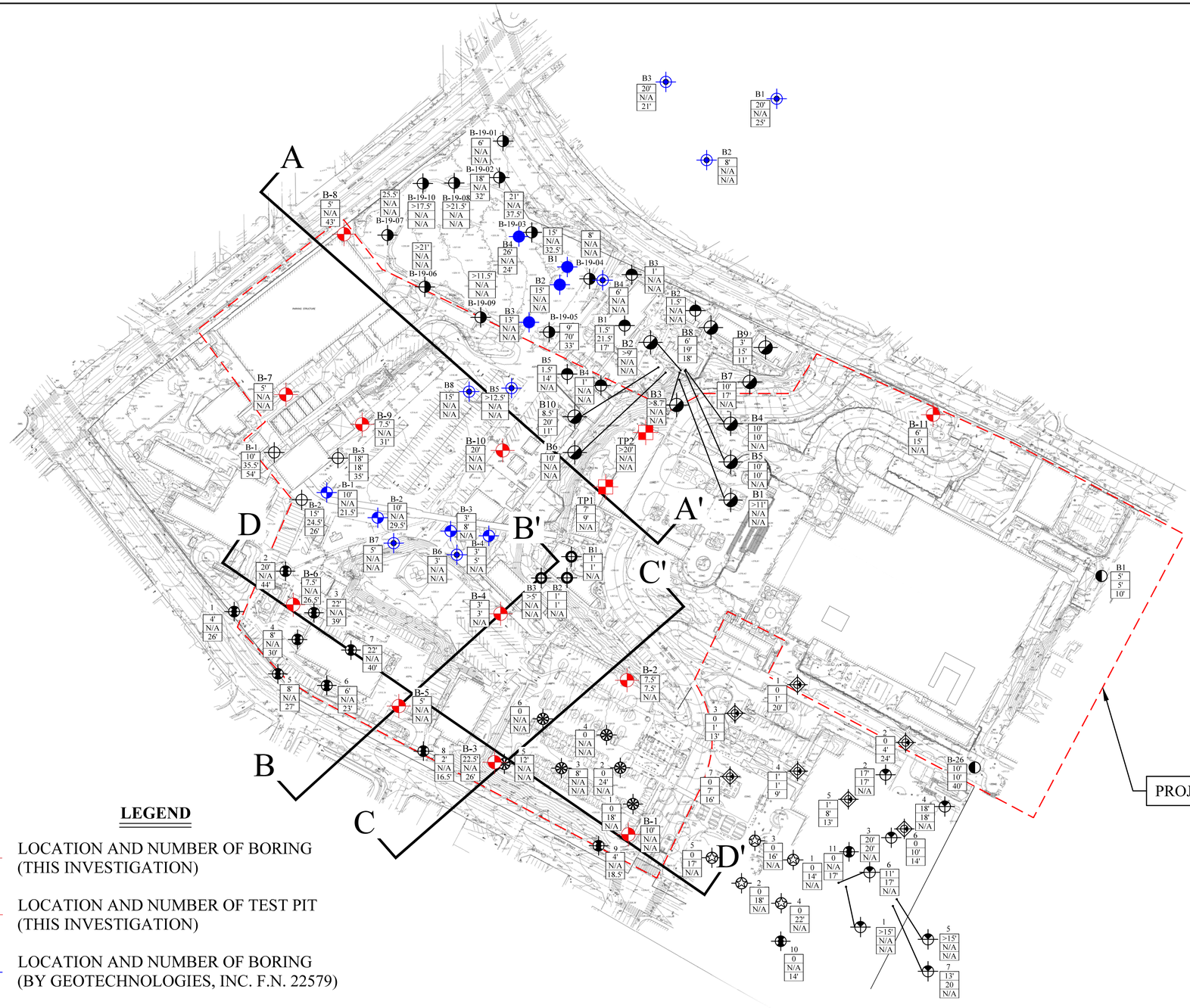
Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTENNIAL GH PARTNERS

FILE NO: 22662

8/28/2025

z:\shared\design cad_old\autocad\2025 projects\22662\22662 geologic map.dwg



PROJECT SITE LIMITS

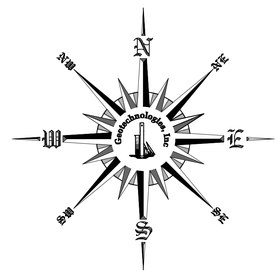
LEGEND

- B-19-10 — LOCATION AND NUMBER OF BORING (BY AECOM JOB NO. 60595699)
- B10 — LOCATION AND NUMBER OF BORING (BY GEOCON WEST, INC. PROJECT NO. A8559-06-74)
- B5 — LOCATION AND NUMBER OF BORING (BY GEOCON PROJECT NO. A9733-06-01)
- B-3 — LOCATION AND NUMBER OF BORING (BY GEOTECHNICAL PROFESSIONALS INC. PROJECT NO. 3080.221)
- B3 — LOCATION AND NUMBER OF BORING (BY RMA GROUP JOB NO. 18-0056-01)
- B-26 — LOCATION AND NUMBER OF BORING (BY DAMES & MOORE JOB NO. 00499-038-015)
- 11 — LOCATION AND NUMBER OF BORING (BY LEROY CRANDALL & ASSOCIATES JOB NO. 60383)
- 7 — LOCATION AND NUMBER OF BORING (BY LEROY CRANDALL & ASSOCIATES JOB NO. 56356)
- 7 — LOCATION AND NUMBER OF BORING (BY L.T. EVANS SEPTEMBER 1950)
- 6 — LOCATION AND NUMBER OF BORING (BY L.T. EVANS AUGUST 1950)
- 5 — LOCATION AND NUMBER OF BORING (BY L.T. EVANS JULY 1948)

LEGEND

- B-11 — LOCATION AND NUMBER OF BORING (THIS INVESTIGATION)
- TP2 — LOCATION AND NUMBER OF TEST PIT (THIS INVESTIGATION)
- B-4 — LOCATION AND NUMBER OF BORING (BY GEOTECHNOLOGIES, INC. F.N. 22579)
- B4 — LOCATION AND NUMBER OF BORING (BY GEOTECHNOLOGIES, INC. F.N. 22473)
- B8 — LOCATION AND NUMBER OF BORING (BY GEOTECHNOLOGIES, INC. F.N. 22146)
- 25' / 35.5' / 54' — DEPTH OF FILL / DEPTH TO BEDROCK / DEPTH TO GROUNDWATER
- D — D' — CROSS SECTIONS

REFERENCE: TOPOGRAPHIC SURVEY PREPARED BY ADKAN ENGINEERS DATED 12/4/2023



SCALE IN FEET
0 60 120 240

GEOLOGIC MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

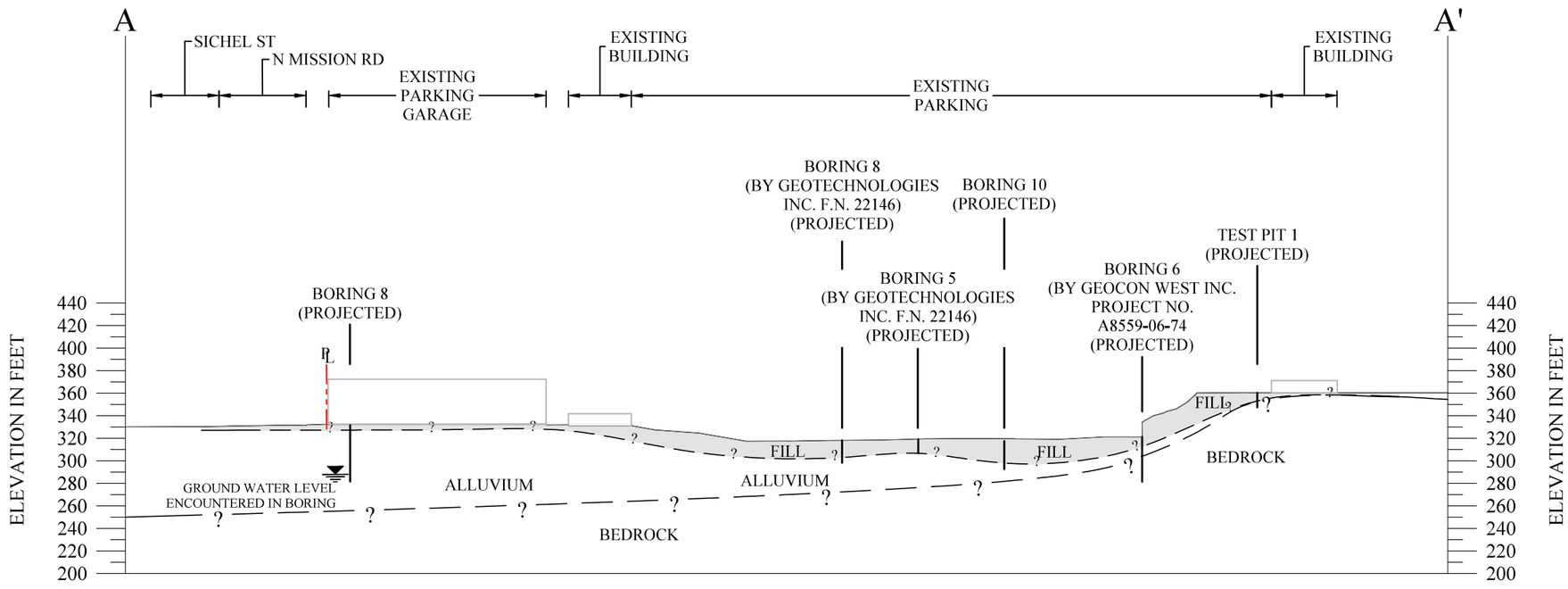
CENTENNIAL GH PARTNERS

Drawn by: JD | File No. 22662

Date: August 2025

Plot Date: Aug. 28, 2025
z:\shared\design cad_old\autocad\2025 projects\22662\22662 cross sections.dwg

N42W\



SCALE IN FEET

0 60 120 240

CROSS SECTION A-A'

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTENNIAL GH PARTNERS

Drawn by: JD | File No.: 22662

Date: August 2025

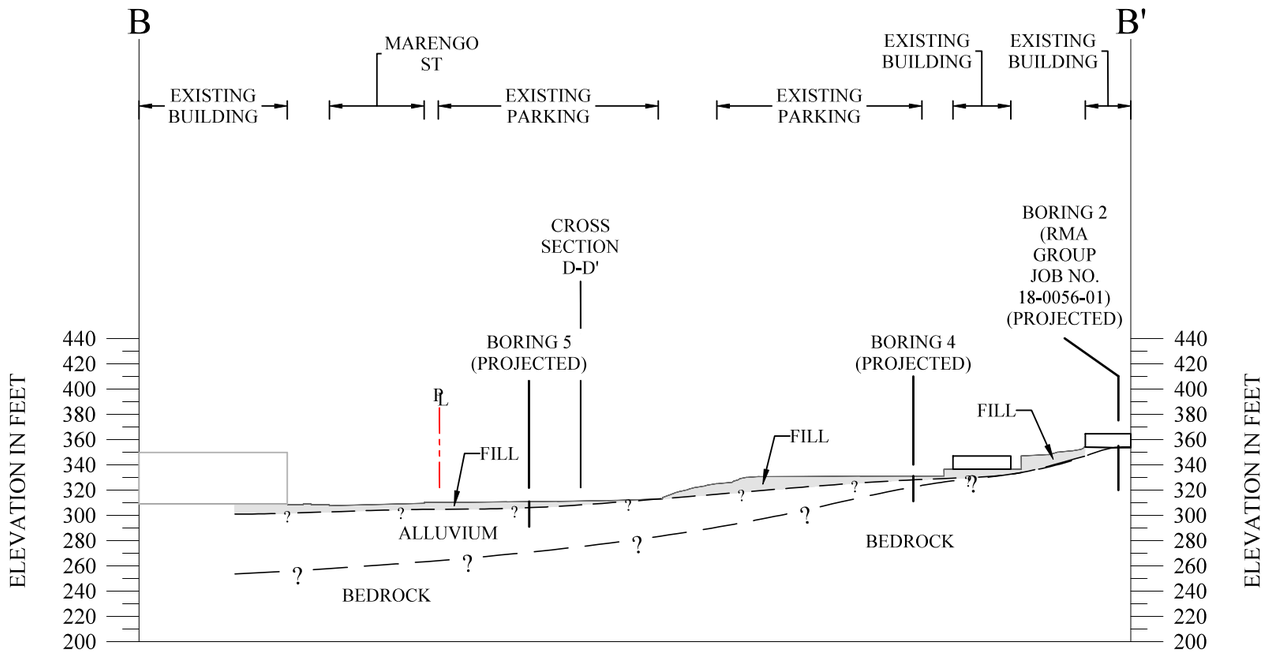
REFERENCE: A.L.T.A. SURVEY PROVIDED BY ADKAN ENGINEERS DATED 12/4/2023

REUSE OF DOCUMENTS: This document and the ideas and design incorporated herein, as an instrument of professional service, is the property of Geotechnologies, Inc. and shall not be reused in whole or part for any other project without Geotechnologies, Inc. express written authorization.

Plot Date: Aug. 28, 2025

z:\shared\design cad_old\autocad\2025 projects\22662\22662 cross sections.dwg

N42E\



SCALE IN FEET

0 60 120 240

CROSS SECTION B-B'

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTENNIAL GH PARTNERS

Drawn by: JD | File No.: 22662

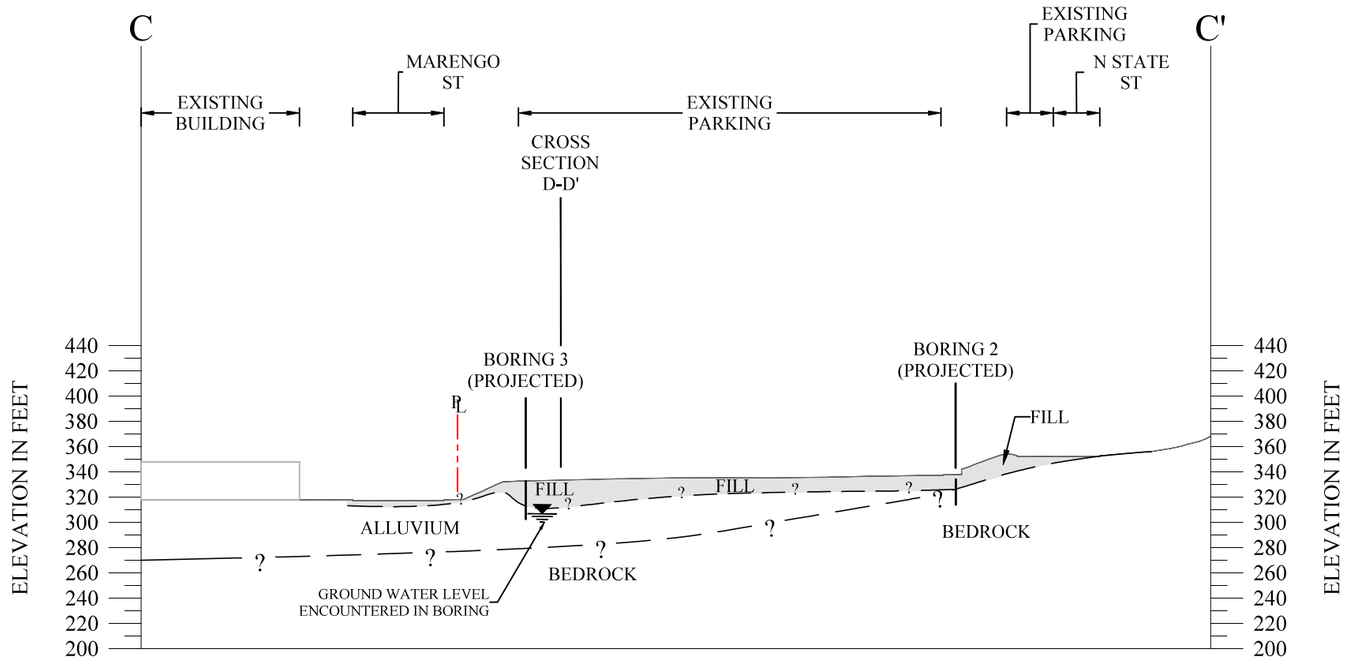
Date: August 2025

REFERENCE: A.L.T.A. SURVEY PROVIDED BY ADKAN ENGINEERS DATED 12/4/2023

REUSE OF DOCUMENTS: This document and the ideas and design incorporated herein, as an instrument of professional service, is the property of Geotechnologies, Inc. and shall not be reused in whole or part for any other project without Geotechnologies, Inc. express written authorization

Plot Date: Aug. 28, 2025
z:\shared\design cad_old\autocad\2025 projects\22662\22662 cross sections.dwg

N42E\



SCALE IN FEET

0 60 120 240

REFERENCE: A.L.T.A. SURVEY PROVIDED BY ADKAN ENGINEERS DATED 12/4/2023

CROSS SECTION C-C'

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTENNIAL GH PARTNERS

Drawn by: JD | File No.: 22662

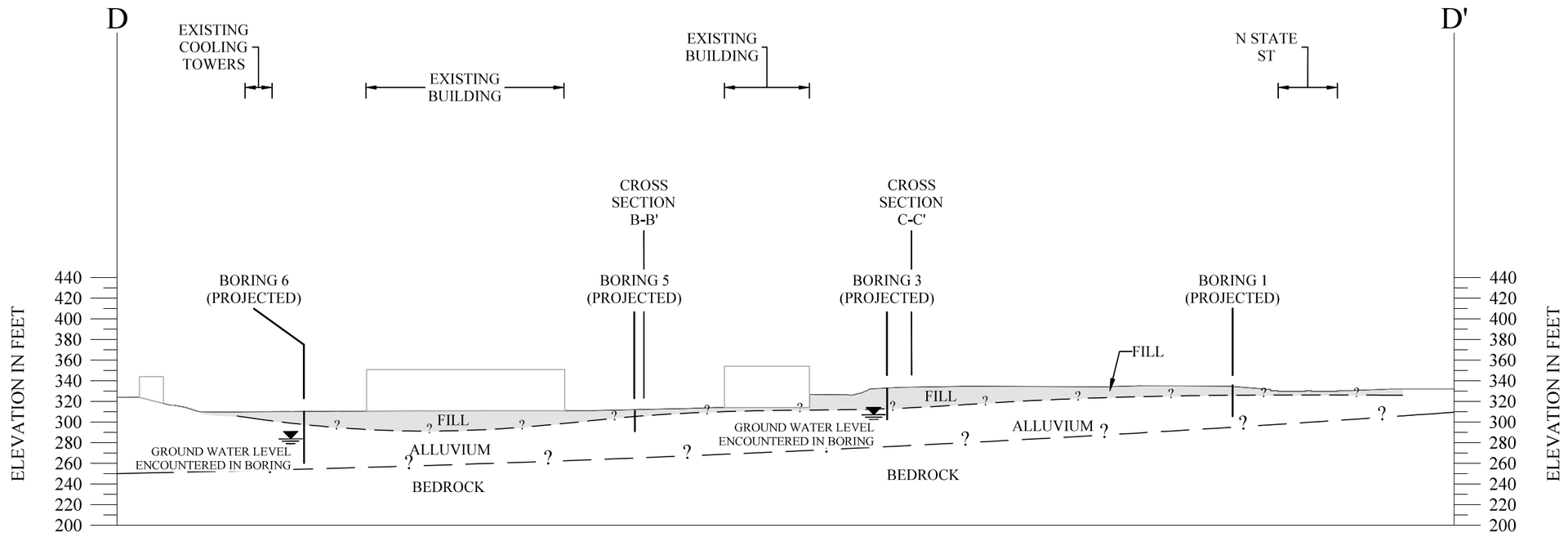
Date: August 2025

REUSE OF DOCUMENTS: This document and the ideas and design incorporated herein, as an instrument of professional service, is the property of Geotechnologies, Inc. and shall not be reused in whole or part for any other project without Geotechnologies, Inc. express written authorization.

Plot Date: Aug. 28, 2025

z:\shared\design cad_old\autocad\2025 projects\22662\22662 cross sections.dwg

N35W\



SCALE IN FEET

0 60 120 240

REFERENCE: A.L.T.A. SURVEY PROVIDED BY ADKAN ENGINEERS DATED 12/4/2023

CROSS SECTION D-D'

Geotechnologies, Inc.
Consulting Geotechnical Engineers

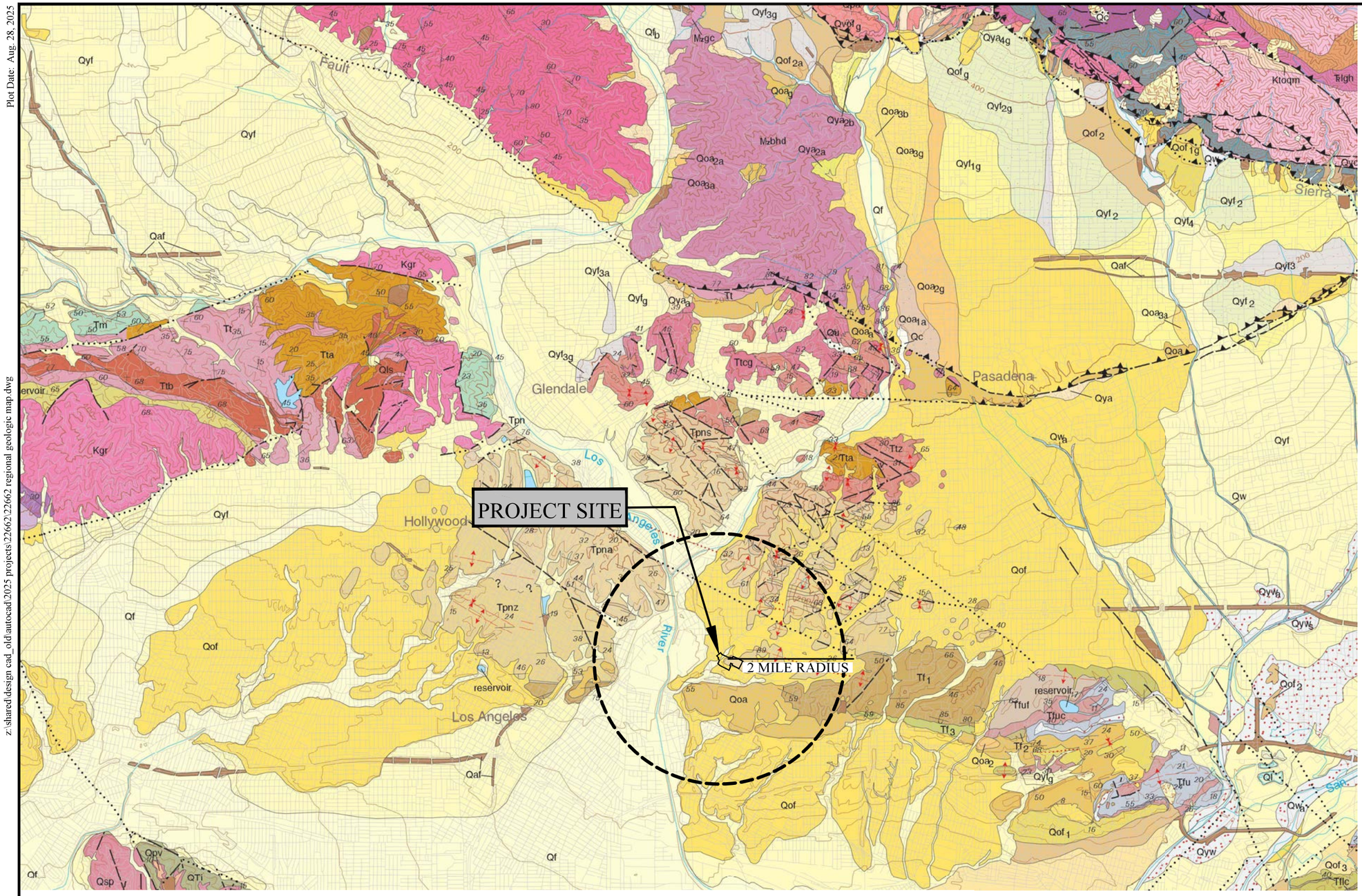
CENTENNIAL GH PARTNERS

Drawn by: JD | File No.: 22662

Date: August 2025

REUSE OF DOCUMENTS: This document and the ideas and design incorporated herein, as an instrument of professional service, is the property of Geotechnologies, Inc. and shall not be reused in whole or part for any other project without Geotechnologies, Inc. express written authorization

Plot Date: Aug. 28, 2025
z:\shared\design cad_old\autocad\2025 projects\22662\22662 regional geologic map.dwg



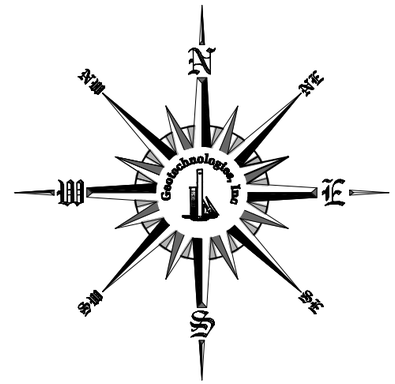
Scale 1:100,000

1 .5 0 1 2 3 4 5 6 7 MILES

1 .5 0 1 2 3 4 5 6 7 KILOMETERS

Contour Interval 40m

REFERENCE: U.S. DEPARTMENT OF THE INTERIOR, U.S. GEOLOGICAL SURVEY, PRELIMINARY GEOLOGIC MAP OF THE LOS ANGELES 30' X 60' QUADRANGLE, SOUTHERN CALIFORNIA,VERSION 1.0, 2005, COMPILED BY ROBERT F. YERKES AND RUSSELL H. CAMPBELL



LEGEND

Qaf: Artificial Fill
Qa: Alluvium
Qf: Alluvial-Fan Deposits
Qof: Old Alluvial-Fan Deposits
Qoa: Old Alluvium
Tm: Modelo Formation
Tt: Topanga Group
TKb: Sediment Rock in the Beverly Hills Area
Kt: Tuna Canyon Formation
Jsm: Santa Monica Slate

Fault: Solid where accurately located, dashed where approximately located, dotted where concealed, quieried where location or existence uncertain. Includes strike slip, normal, reverse, oblique, and unspecified slip.

REGIONAL GEOLOGIC MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

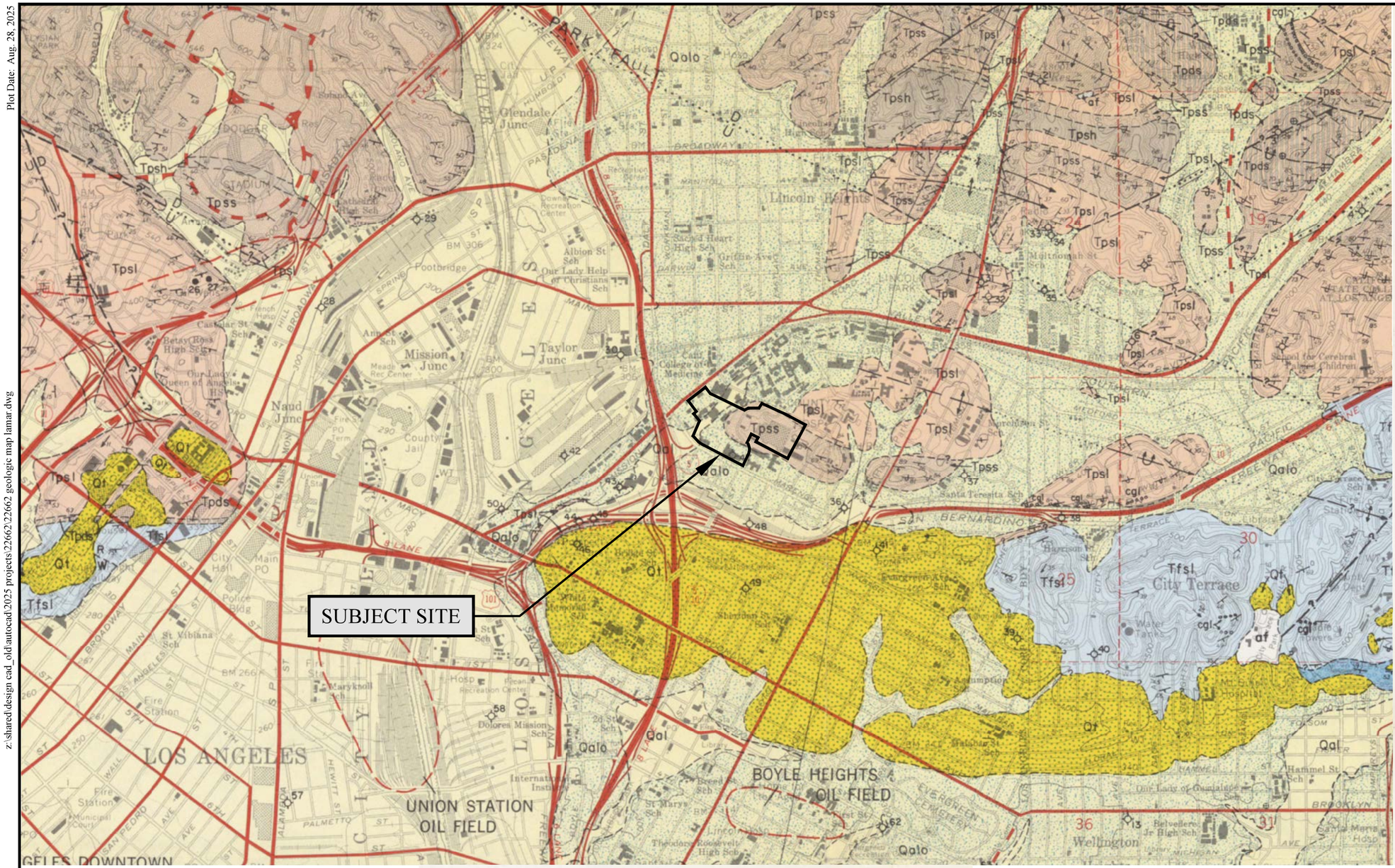
CENTENNIAL GH PARTNERS

Drawn by: JD | File No.: 22662

Date: August 2025

REUSE OF DOCUMENTS: This document and the ideas and design incorporated herein, as an instrument of professional service, is the property of Geotechnologies, Inc. and shall not be reused in whole or part for any other project without Geotechnologies, Inc. express written authorization

Plot Date: Aug. 28, 2025
z:\shared\design cad_old\autocad\2025 projects\22662\22662 geologic map lamar.dwg



EXPLANATION

RECENT
- af — Artificial fill
- Qal — Alluvium. Silt, sand and gravel.
- Qls — Landslide debris

PLEISTOCENE
- Qaloa — Old alluvium. Silt, sand and gravel forming dissected alluvial plain and alluvial terrace deposits.
- Qt — Terrace Deposits. Silt, sand and gravel forming alluvial terrace and dissected alluvial plain deposits.

PLIOCENE
- Tfcg / Tfss / Tfsl — Fernando Formation. Tfcg: conglomerate, light to red brown; Tfss: sandstone, fine- to medium-grained, light tan and brown; Tfsl: siltstone, massive, light gray; R/W: Repettian-Wheelerian Stage boundary

UPPER MIOCENE
- Tpsl / Tpsh / Tpds / Tpss / Tpun — Puente Formation. Tpsl: siltstone, well bedded, light brown and light gray; Tpsh: shale, well bedded, light gray, siliceous; Tpds: diatomaceous shale, punky, dull white; Tpss: sandstone, well bedded, medium- to coarse-grained, light brown to gray; Tpun: undifferentiated siltstone, shale, sandstone, and conglomerate.

MIDDLE MIOCENE
- Ttss / Ttcg / Ttsl — Topanga Formation. Ttss: sandstone, medium- to coarse-grained, well bedded, brown; conglomeratic north of York Boulevard fault; Ttcg: conglomerate, massive to well bedded, light brown; breccia at base north of York Boulevard fault; Ttsl: siltstone, well bedded, medium to dark brown, with interbedded sandstone, shale and chert.
- Td — Diabase. Dark red brown, deeply weathered.

CRETACEOUS (?)
- wqm / wgg — Wilson Quartz Diorite. wqm: massive, medium- to coarse-grained, light brown; wgg: gneissic, minor amounts of schist. Includes unmapped granodiorite dikes.
- vqd — Vermont Quartz Diorite. Fine- to medium-grained, massive, brown.
- tgd — Feliz Granodiorite. Fine- to medium-grained, massive, light gray to light brown.

Lithologic symbols for individual beds and small areas of exposure: cgl: conglomerate; ss: sandstone; t: tuff

QUATERNARY / TERTIARY / CRETACEOUS (?)
- Contact. Dashed where approximately located, querried where inferred.
- Fault, showing dip. Dashed where approximately located, querried where inferred or hypothetical, dotted where concealed. U: up-thrown side; D: down-thrown side; arrows indicate relative movement. Only horizontal separation demonstrated by offset of structures within map area are indicated. See text for discussion of possible magnitude and direction of horizontal movement on major faults based on regional distribution of rocks.
- Anticline / Syncline. Axial planes of folds; arrows indicate dip direction on flanks and plunge; dashed where approximately located.
- Strike and dip of beds
- Strike and dip of beds inferred to be overturned.
- Horizontal beds
- Vertical beds
- Component of dip, strike unknown.
- Strike and dip of beds with grading or other sedimentary structures indicating tops.
- Strike and dip of foliation
- Dry hole / Producer. Numbers refer to list of wells drilled for oil in text. Circle indicates surface location of directed wells. Not all old wells and wells in oil fields are shown.
- Limits of oil field
- Xf — Fossil locality

1000 0 2000 4000 6000 8000 FEET

CONTOUR INTERVAL 5 AND 25 FEET

REFERENCE: LAMAR, D.L., 1970, GEOLOGY OF THE ELYSIAN PARK-REPETTO HILLS AREA, LOS ANGELES COUNTY, CALIFORNIA, CALFORNIA DIVISION OF MINES AND GEOLOGY, SPECIAL REPORT SR-101

LOCAL GEOLOGIC MAP (LAMAR)

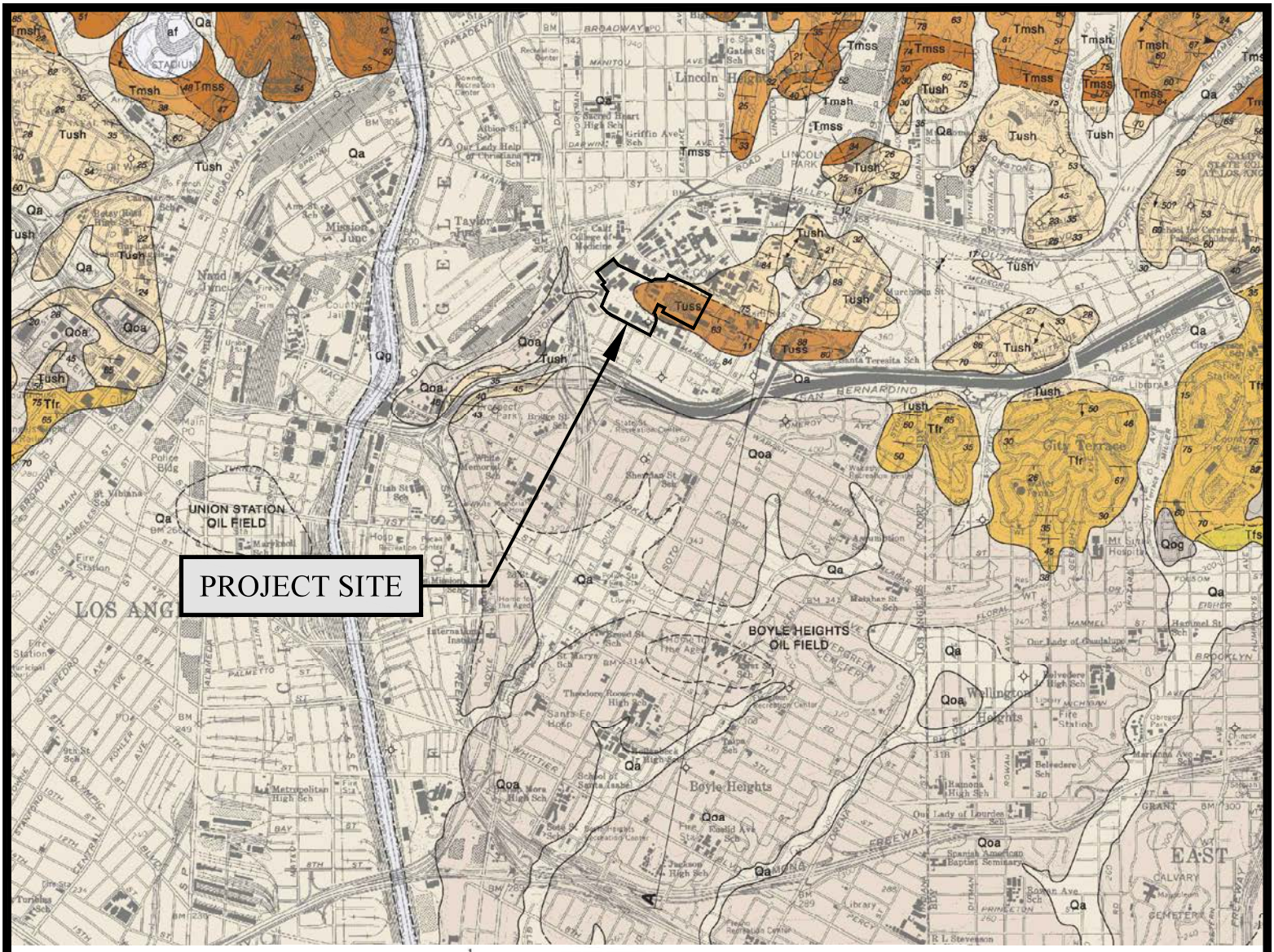
Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTENNIAL GH PARTNERS

Drawn by: JD | File No.: 22662

Date: August 2025

REUSE OF DOCUMENTS. This document and the ideas and design incorporated herein, as an instrument of professional service, is the property of Geotechnologies, Inc. and shall not be reused in whole or part for any other project without Geotechnologies, Inc. express written authorization.



LEGEND

af: Surficial Sediments - Artificial fill
Qg: Alluvial clay and sand of valley areas
Qa: Alluvium: clay, sand & gravel
Qoa: Older Surficial Sediments - Remnants of older weakly consolidated alluvial deposits of gravel, sand and silt
Tfr: Marine Claystone - Gray, soft, vaguely bedded claystone
Tush: Unnamed Shale - Gray to light brown, thin bedded, silty clay shale, locally contains scattered large calcareous nodules
Tuss: Unnamed Shale - Light gray to tan semi-friable sandstone, with thin interbeds of silty shale
Tmsh: Monterey Formation - White-weathering, thin bedded, platy, siliceous shale, locally porcelaneous and silty
Tmss: Tan to light gray, semi-friable arkosic sandstone; includes some interbedded silty shale

Folds - arrow on axial trace of fold indicates direction of plunge

Fault - dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful

REFERENCE: T.W. DIBBLEE (EDITED 2006) GEOLOGIC MAP OF LOS ANGELES QUADRANGLES (#DF-22)

GEOLOGIC MAP (DIBBLEE)

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTENNIAL GH PARTNERS

FILE NO: 22662



SCALE IN FEET

0 300 600

N

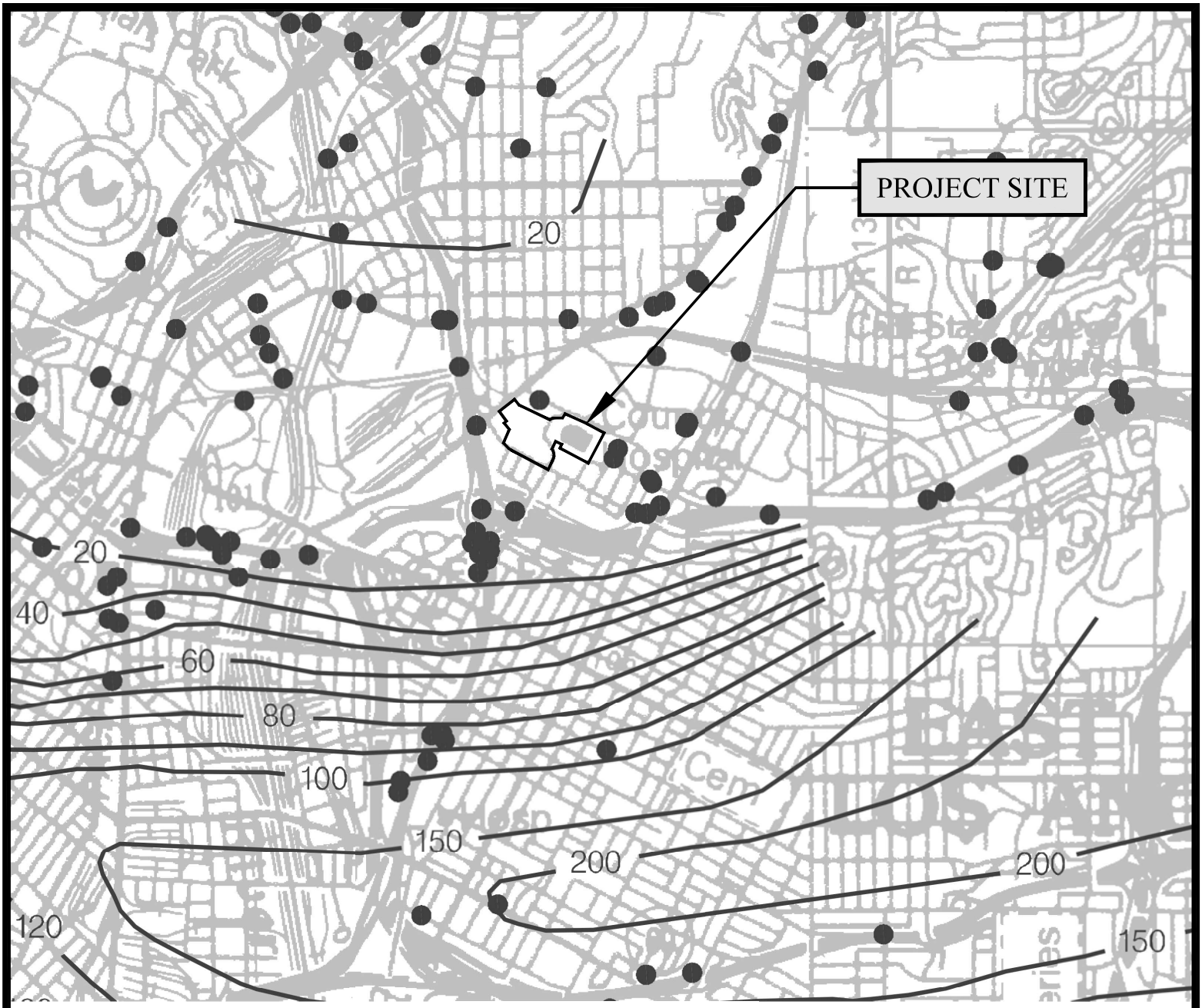
REFERENCE: Navigate LA Website [https://navigatela.lacity.org/navigatela]

FAULT LOCATION MAP (NAVIGATELA)

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTENNIAL GH PARTNERS

FILE NO. 22662



ONE MILE

SCALE

N

LEGEND

- Borehole Site
- 30 — Depth to ground water in feet

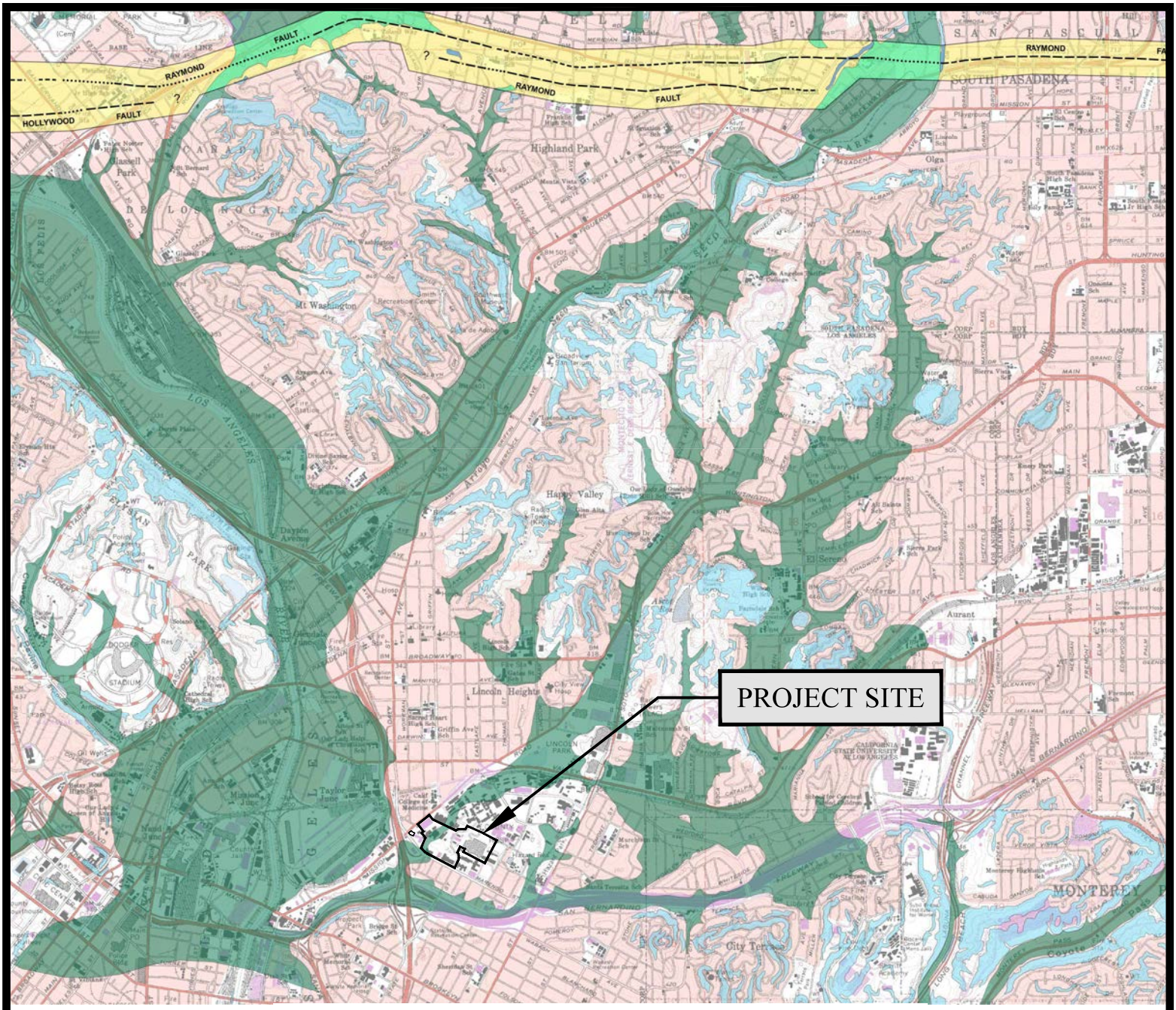
REFERENCE: CDMG, SEISMIC HAZARD ZONE REPORT, 029 LOS ANGELES, 7.5 - MINUTE QUADRANGLE, LOS ANGELES COUNTY, CALIFORNIA 1998, (REV 2006)

HISTORICALLY HIGHEST GROUNDWATER LEVELS MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTENNIAL GH PARTNERS

FILE NO: 22662



1 0.5 0 1 2 Miles

1,000 500 0 1,000 2,000 3,000 4,000 5,000 6,000 7,000 8,000 9,000 10,000 Feet

1 0.5 0 1 2 Kilometers

1,000 500 0 1,000 2,000 Meters

LEGEND

- LIQUEFACTION ZONES
- EARTHQUAKE-INDUCED LANDSLIDE ZONES
- ACTIVE FAULT TRACES
- EARTHQUAKE FAULT ZONES

N

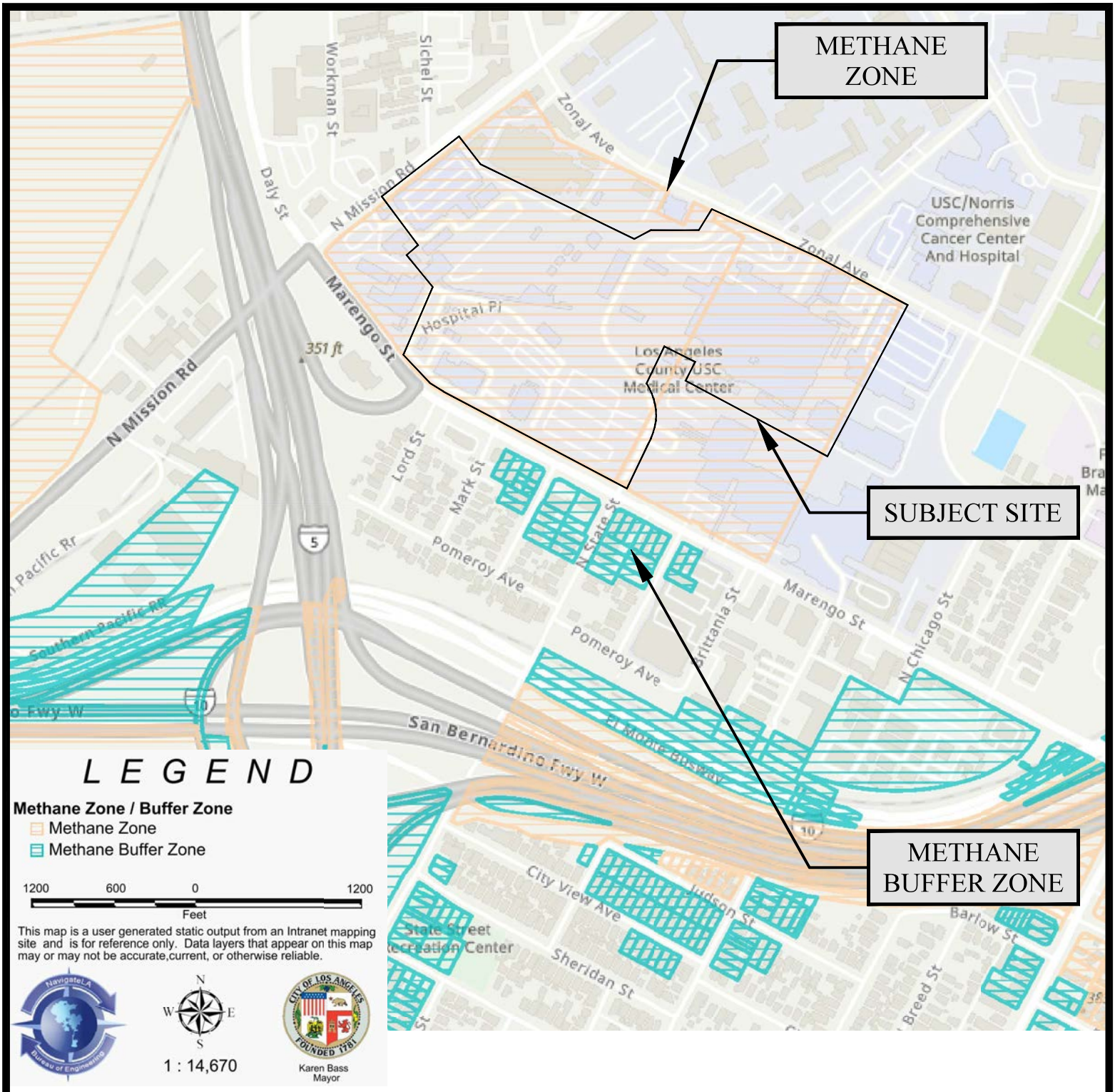
REFERENCE: EARTHQUAKE ZONES OF REQUIRED INVESTIGATION, LOS ANGELES QUADRANGLE (CGS, 2017)

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTENNIAL GH PARTNERS

FILE NO: 22662



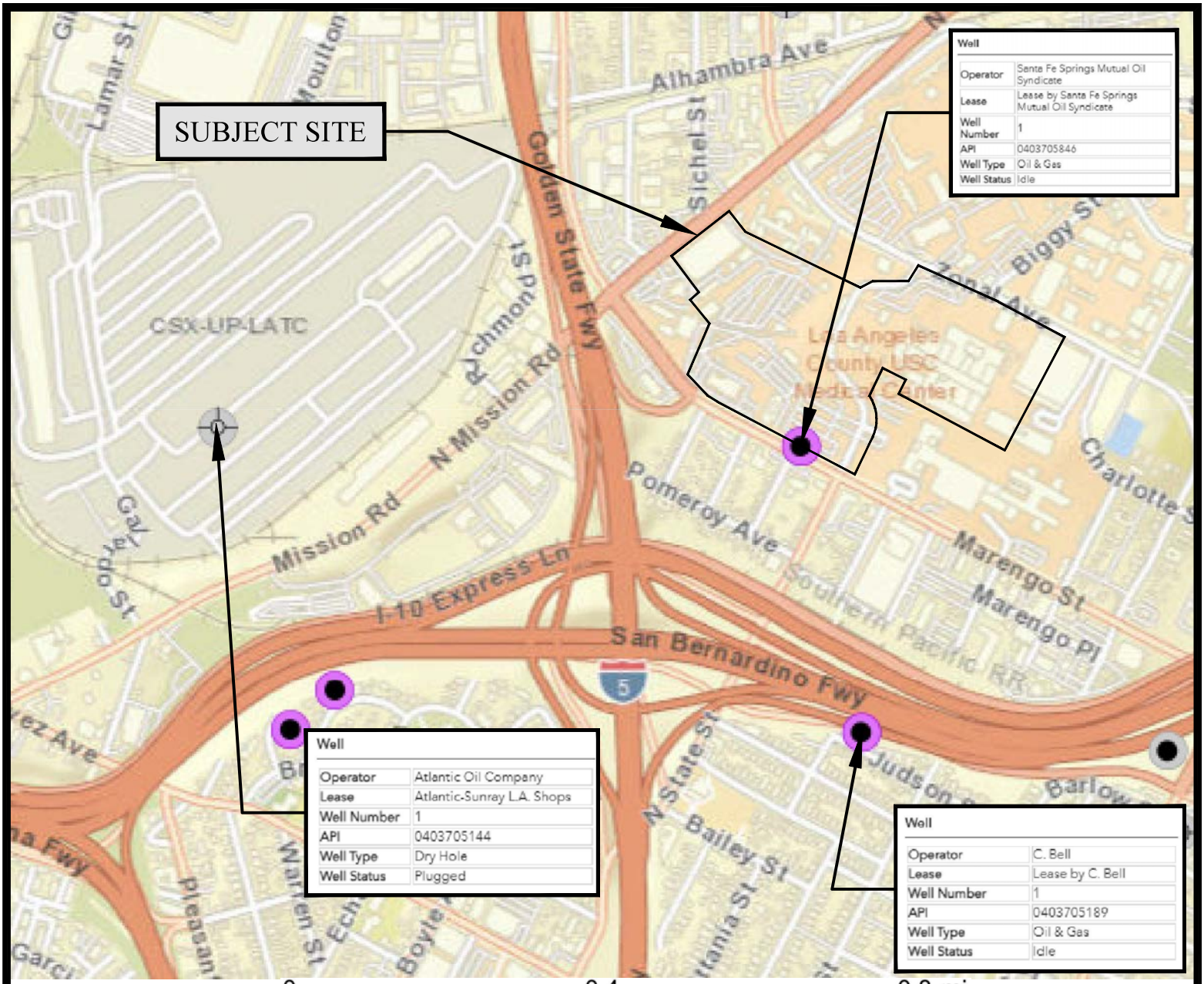
REFERENCE: https://navigatela.lacity.org/

METHANE ZONE RISK MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTENNIAL GH PARTNERS

FILE NO: 22662



0 0.4 0.8 mi

0 0.55 1.1 km

N

LEGEND

OIL WELLS

REFERENCE: CDEMD, Well Finder (https://maps.conservation.ca.gov/doggr/wellfinder/)

OIL WELL LOCATION MAP

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTENNIAL GH PARTNERS

FILE NO: 22662

BORING LOG NUMBER 1

Centennial GH Partners **Date: 07/14/25** **Elevation: 335.5'**

File No. 22662 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4-inch Asphalt over 11-inch Base
				-		
				1 --		
				-		FILL: Sandy Clay, dark and yellowish brown, moist, stiff
				2 --		
2.5	41	21.5	106.1	-		
				3 --		rock fragments
				-		
				4 --		
				-		
5	66	20.5	104.6	5 --		
				-		Sandy to Silty Clay, yellowish brown, moist, very stiff, fine grained, rock fragments
				6 --		
				-		
				7 --		
7.5	63	16.0	110.5	-		
				8 --		dark brown
				-		
				9 --		
				-		
10	75	17.3	113.1	10 --		
				-	CL	ALLUVIUM: Sandy Clay, dark and yellowish brown, moist, very stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	58	17.0	113.1	15 --		
	50/5"			-	CL/SC	Sandy Clay to Clayey Sand, medium brown, moist, very stiff, medium dense, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	80	16.2	115.4	20 --		
				-	CL	Sandy Clay, medium brown, moist, very stiff
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	64	15.9	115.2	25 --		
	50/5"			-		grayish brown

BORING LOG NUMBER 1

Centennial GH Partners

File No. 22662

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 -- - 28 -- - 29 -- -		
30	86	18.7	111.2	30 -- - 31 -- - 32 -- - 33 -- - 34 -- - 35 -- - 36 -- - 37 -- - 38 -- - 39 -- - 40 -- - 41 -- - 42 -- - 43 -- - 44 -- - 45 -- - 46 -- - 47 -- - 48 -- - 49 -- - 50 -- -	CL/SC	Sandy Clay to Clayey Sand, grayish brown, moist, very stiff, very dense, fine grained Total Depth 30 feet No Water Fill to 10 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 2

Centennial GH Partners **Date: 07/16/25** **Elevation: 340.0'**

File No. 22662 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		3½-inch Asphalt over 4-inch Base
				1 --		FILL: Sandy Clay, dark brown, moist, stiff
				2 --		
2.5	52	13.7	100.4	-		
				3 --		concrete fragments
				4 --		
5	52	16.0	115.6	5 --		
				6 --		Sandy Clay to Clayey Sand, dark brown, moist, stiff, dense, occasional concrete fragments
				7 --		
7.5	31	12.2	101.9	-		
	50/4"			8 --		BEDROCK (PUENTE FORMATION): Siltstone to Sandstone, medium to yellowish brown, moist, moderately hard, bedded
				9 --		
10	39	22.3	99.5	10 --		
	50/4"			-		grayish brown
				11 --		
				12 --		
				13 --		
				14 --		
15	34	21.0	103.2	15 --		
	50/5"			-		Sandstone, orange and grayish brown
				16 --		
				17 --		
				18 --		
				19 --		
				-		Sandstone to Siltstone
20	35	21.0	100.1	20 --		
	50/3"			-		Total Depth 20 feet No Water Fill to 7½ feet
				21 --		
				22 --		
				23 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 3

Centennial GH Partners **Date: 07/15/25** **Elevation: 332.5'**

File No. 22662 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4-inch Asphalt over 9-inch Base
				1 --		FILL: Clayey Sand, dark brown, moist, medium dense, fine grained
2.5	30 50/5"	11.7	104.5	2 --		rock fragments
				3 --		
				4 --		
5	40 50/5"	10.7	118.1	5 --		Clayey Sand, dark and yellowish brown, moist, very dense, very stiff, fine grained, with rock fragments
				6 --		
7.5	45 50/44"	12.1	115.5	7 --		Sandy Clay, dark brown, moist, very stiff, few gravel
				8 --		
				9 --		
10	88	12.4	114.6	10 --		Silty Sand to Sandy Silt, dark brown, moist, very dense, very stiff, fine grained, brick fragments
				11 --		
12.5	80	12.2	107.1	12 --		dark and medium brown, occasional rock fragments
				13 --		
				14 --		
15	83	10.7	106.7	15 --		Sandy Silt, medium brown, moist, very stiff, few rock fragments
				16 --		
17.5	45 50/2"	22.9	98.5	17 --		Sandy to Silty Clay, dark brown, wet, stiff
				18 --		
				19 --		
20	100/8"	26.6	91.7	20 --		
				21 --		
22.5	38 50/4"	12.0	121.6	22 --		
				23 --	CL	ALLUVIUM: Sandy Clay, dark and yellowish brown, moist, very stiff
				24 --		
25	95	11.3	120.6	25 --		
					SC	Clayey Sand, medium brown, moist, very dense, fine grained

BORING LOG NUMBER 3

Centennial GH Partners

File No. 22662

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 -- - 28 -- - 29 -- -		dark and grayish brown
30	100/8"	13.6	113.0	30 -- - 31 -- - 32 -- - 33 -- - 34 -- - 35 -- - 36 -- - 37 -- - 38 -- - 39 -- - 40 -- - 41 -- - 42 -- - 43 -- - 44 -- - 45 -- - 46 -- - 47 -- - 48 -- - 49 -- - 50 -- -		Total Depth 30 feet Water at 26 feet Fill to 22½ feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 4

Centennial GH Partners **Date: 07/15/25** **Elevation: 331.0'**

File No. 22662 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		3-inch Asphalt over 4-inch Base
				1 --		FILL: Sandy Clay, dark brown and yellowish brown, moist, stiff
2.5	40 50/5"	26.0	97.2	2 --		
				3 --		BEDROCK (PUENTE FORMATION): Siltstone, yellowish brown, moist, moderately hard, weathered
5	45 50/4"	26.0	98.6	4 --		less weathered, yellow and grayish brown
				5 --		
				6 --		
				7 --		
				8 --		
				9 --		
10	85	30.3	92.6	10 --		Siltstone to Sandstone, yellow and grayish brown, moist, moderately hard, bedding
				11 --		
				12 --		
				13 --		
				14 --		
15	40 50/5"	27.9	93.9	15 --		
				16 --		
				17 --		
				18 --		
				19 --		Sandstone
20	35 50/4"	28.5	90.0	20 --		Total Depth 20 feet No Water Fill to 3 feet
				21 --		
				22 --		
				23 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				24 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				25 --		

BORING LOG NUMBER 5

Centennial GH Partners **Date: 07/15/25** **Elevation: 311.0'**

File No. 22662 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		3-inch Asphalt over 5-inch Base
				1 --		FILL: Clayey Sand, dark brown, moist, medium dense, fine grained
				2 --		
2.5	44	18.3	105.0	-		
				3 --		dark and grayish brown, few pebbles
				4 --		
5	38	18.4	106.2	5 --		
				-	CL	ALLUVIUM: Silty Clay, dark brown and gray, moist, stiff
				6 --		
				7 --		
7.5	43	19.3	110.0	-		
				8 --		
				9 --		
10	50	18.9	109.8	10 --		
				11 --		
				12 --		
12.5	44	19.6	111.8	-		
				13 --	CL	Sandy Clay, dark brown, moist, stiff
				14 --		
15	84	19.6	110.5	15 --		
				-		very stiff
				16 --		
				17 --		
				18 --		
				19 --		
				-	SC	Clayey Sand, gray, moist to very moist, dense, fine grained
20	76	19.6	104.6	20 --		
				-		Total Depth 20 feet No Water Fill to 5 feet
				21 --		
				22 --		
				23 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				24 --		
				25 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 6

Centennial GH Partners **Date: 07/14/25** **Elevation: 310.0'**

File No. 22662 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4½-inch Asphalt over 4½-inch Base
				1 --		FILL: Silty Clay, dark and yellowish brown, moist, stiff
2.5	37	19.9	106.8	2 -- 3 --		
				4 --		
5	14	20.4	SPT	5 --		Sandy Clay, dark brown and gray, moist, stiff, minor brick fragments
				6 --		
7.5	38	20.4	99.9	7 -- 8 --	CL	ALLUVIUM: Sandy Clay, dark gray, moist, stiff
				9 --		
10	15	21.6	SPT	10 --		
				11 --		
12.5	72	24.9	101.6	12 -- 13 --	SM/CL	Silty Sand to Silty Clay, grayish brown and dark brown, moist, medium dense, stiff, fine grained
				14 --		
15	17	30.9	SPT	15 --	CL	Sandy Clay, grayish brown, moist, stiff
				16 --		
17.5	55	21.6	102.2	17 -- 18 --		
				19 --		
20	17	25.7	SPT	20 --		
				21 --		
22.5	57	21.0	109.4	22 -- 23 --	CL	Sandy Silty Clay, dark brown, moist, very stiff
				24 --		
25	73 50/4"	22.6	SPT	25 --		

BORING LOG NUMBER 6

Centennial GH Partners

File No. 22662

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 -- -		
27.5	36	20.9	106.1	- 28 -- - 29 -- -	SM/SP	Silty Sand to Sand, gray to dark gray, moist, medium dense, fine grained
30	69	23.4	SPT	30 -- - 31 -- - 32 -- -		moist to wet, very dense
32.5	32 50/4"	23.7	96.8	33 -- - 34 -- -		
35	85	13.9	SPT	35 -- - 36 -- - 37 -- -	SP/SW	Sand to Gravelly Sand, gray, wet, very dense, fine to medium, grained
37.5	45 50/5"	18.2	111.9	38 -- - 39 -- -	SP	Sand, dark gray, wet, very dense, fine to medium grained
40	69 50/5"	22.5	SPT	40 -- - 41 -- - 42 -- -	SP/SC	Sand to Clayey Sand, gray to dark gray, moist to wet, very dense, fine to coarse grained
42.5	48 50/3"	11.7	121.0	43 -- - 44 -- -	SP	Sand, gray to dark gray, wet, very dense, fine to coarse grained
45	39 50/4"	15.9	SPT	45 -- - 46 -- -	SP/SW	Sand to Gravelly Sand, gray to dark gray, wet, very dense, coarse grained
47.5	40 50/3"	13.4	119.0	47 -- - 48 -- - 49 -- -		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted SPT=Standard Penetration Test
50	38 50/4"	10.6	SPT	50 -- -		Total Depth 50 feet Water at 26½ feet Fill to 7½ feet

BORING LOG NUMBER 7

Centennial GH Partners **Date: 07/17/25** **Elevation: 334.0'**

File No. 22662 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		3-inch Asphalt, No Base
				-		
				1 --		FILL: Clayey Sand, yellowish brown, moist, medium dense, fine grained, occasional rock fragments
				-		
				2 --		
2.5	25	17.3	113.0	-		
				3 --		dark brown
				-		
				4 --		
				-		
5	21	13.4	122.4	5 --		
	50/5"			-	SM/ML	ALLUVIUM: Clayey Sand, dark brown, moist, very dense, fine grained
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	56	26.7	96.9	10 --		
				-	CL	Sandy Clay, medium brown, moist, stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	76	14.1	121.8	15 --		
				-	SC/CL	Clayey Sand to Sandy Clay, dark brown, moist, dense, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-	CL	Sandy Clay, dark brown and gray, moist, very stiff
20	78	17.9	113.9	20 --		
				-		Total Depth 20 feet No Water Fill to 5 feet
				21 --		
				-		
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				-		
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 8

Centennial GH Partners **Date: 07/17/25** **Elevation: 331.5'**

File No. 22662 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		6-inch Asphalt over 4-inch Base
				-		
				1 --		
				-		FILL: Clayey Sand, dark brown, moist, medium dense, fine grained
				2 --		
				-		
				3 --		
				-		
				4 --		
				-		
5	21	16.0	SPT	5 --		
				-	ML/CL	ALLUVIUM: Sandy Clay, dark brown, moist, stiff
				6 --		
				-		
				7 --		
7.5	42	14.6	112.4	-		
				8 --		
				-		
				9 --		
				-		
10	24	21.5	SPT	10 --		
				-	ML	Sandy Silt, yellowish brown, moist, stiff
				11 --		
				-		
				12 --		
12.5	29	23.6	100.6	-		
				13 --		
				-		
				14 --		
				-		
15	20	16.2	SPT	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	50	21.4	106.3	-		
				18 --	CL	Sandy Clay, dark brown, moist, stiff
				-		
				19 --		
				-		
20	14	17.4	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
22.5	32	13.9	118.9	-		
	50/5"			23 --	SC	Clayey Sand, grayish brown, moist, very dense, fine grained
				-		
				24 --		
				-		
25	32	8.2	SPT	25 --		
				-	SM/SP	Silty Sand to Sand, yellow and dark brown, moist, medium dense, fine grained

BORING LOG NUMBER 8

Centennial GH Partners

File No. 22662

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
27.5	53	28.2	95.9	28 --	ML	Sandy Silt, grayish brown, moist, stiff
				29 --		
30	25	24.6	SPT	30 --		
				31 --		
				32 --		
32.5	66	23.8	102.1	33 --		
				34 --		
35	28	22.2	SPT	35 --	CL	Sandy Clay, grayish brown, moist, stiff
				36 --		
				37 --		
37.5	69	25.1	102.2	38 --		
				39 --		
40	19	24.8	SPT	40 --	CH	Silty Clay, medium brown, moist, stiff
				41 --		
				42 --		
42.5	47 50/5"	20.0	108.3	43 --		
				44 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
45	17	15.5	SPT	45 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted SPT=Standard Penetration Test
				46 --		
				47 --		
47.5	81	8.3	126.5	48 --	SC	Clayey Sand, yellowish brown, moist, very dense, stiff, fine to coarse grained
				49 --		
50	34	16.4	SPT	50 --		Total Depth 50 feet Water at 43 feet Fill to 5 feet

BORING LOG NUMBER 9

Centennial GH Partners **Date: 07/16/25** **Elevation: 314.5'**

File No. 22662 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4-inch Asphalt over 8-inch Base
				1 --		
				2 --		FILL: Clayey Sand to Sandy Clay, dark and yellowish brown, moist, medium dense, stiff, fine grained
2.5	33	15.1	107.4	3 --		
				4 --		
5	35	14.1	SPT	5 --		occasional concrete fragments
				6 --		
				7 --		
7.5	33	14.5	106.9	8 --	CL	ALLUVIUM: Sandy Clay, dark brown, moist, stiff
				9 --		
10	25	12.9	SPT	10 --		
				11 --		
				12 --		
12.5	72	15.9	115.3	13 --		medium brown
				14 --		
15	37	24.9	SPT	15 --	ML	Sandy Silt, yellow and grayish brown, moist, stiff
				16 --		
				17 --		
17.5	42 50/3"	14.6	110.0	18 --	SM	Silty Sand, grayish brown, moist, very dense, fine grained
				19 --		
20	43 50/3"	14.9	SPT	20 --		
				21 --		
				22 --		
22.5	23 50/5"	19.0	111.5	23 --	SM/ML	Silty Sand to Sandy Silt, yellow and grayish brown, moist, very stiff, very dense, fine grained
				24 --		
25	78	8.4	SPT	25 --		

BORING LOG NUMBER 9

Centennial GH Partners

File No. 22662

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 -- - 28 -- - 29 -- -		
30	100/9"	20.3	103.8	30 -- - 31 -- - 32 -- - 33 -- - 34 -- -	SM	Silty Sand, gray, moist, very dense, fine grained
35	100/8"	24.4	103.4	35 -- - 36 -- - 37 -- - 38 -- - 39 -- -		
40	100/8"	21.2	107.2	40 -- - 41 -- - 42 -- - 43 -- - 44 -- -		
45	100/9"	14.1	120.4	45 -- - 46 -- - 47 -- - 48 -- - 49 -- -	SP	Sand, gray, wet, very dense, medium grained NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted SPT=Standard Penetration Test
50	100/8"	14.0	112.8	50 -- -		Total Depth 50 feet Water at 31 feet Fill to 7½ feet

BORING LOG NUMBER 10

Centennial GH Partners **Date: 07/14/25** **Elevation: 317.5'**

File No. 22662 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		3-inch Asphalt, No Base
				-		
				1 --		FILL: Sandy Clay, dark and yellowish brown, moist, stiff
				-		
				2 --		
2.5	49	20.8	103.7	-		
				3 --		
				-		
				4 --		
				-		
5	38	7.6	120.1	5 --		
	50/5"			-		Sand to Silty Sand, grayish brown, moist, very dense, fine to
				6 --		coarse grained, pebbles
				-		
				7 --		
7.5	100/8"	14.6	90.5	-		
				8 --		Silty Sand, dark and grayish brown, moist, very dense, fine
				-		to coarse grained, brick and concrete fragments
				9 --		
				-		
10	100/8"	13.0	83.1	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	100/8"	14.9	111.7	-		
				13 --		Clayey Sand, medium to dark brown, moist, very dense, fine to
				-		medium grained, pebbles and brick fragments
				14 --		
				-		
15	88	17.1	102.2	15 --		
				-		Clayey Sand to Sandy Clay, dark and yellowish brown, moist,
				16 --		very dense, very stiff, fine grained, rock fragments
				-		
				17 --		NOTE: The stratification lines represent the approximate
17.5	72	15.0	107.2	-		boundary between earth types; the transition may be gradual.
				18 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				19 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
20	62	11.8	108.1	20 --		
				-	SM/ML	ALLUVIUM: Silty Sand to Sandy Silt, yellow and grayish brown,
				21 --		moist, dense, stiff, fine grained
				-		
				22 --		
22.5	40	19.5	111.0	-		
	50/4"			23 --	SM/SP	Sand to Silty Sand, yellow and grayish brown, moist, very
				-		dense, fine grained
				24 --		
				-		
25	100/8"	24.1	95.2	25 --		
				-		Total Depth 25 feet
						No Water
						Fill to 20 feet

BORING LOG NUMBER 11

Centennial GH Partners **Date: 07/15/25** **Elevation: 346.5'**

File No. 22662 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		4-inch Asphalt over 6-inch Base
				-		
				1 --		
				-		FILL: Sandy Clay, dark brown, moist, stiff
				2 --		
2.5	71	13.8	105.1	-		
				3 --		Sandy Clay to Clayey Sand, dark and yellowish brown, moist, very stiff, very dense, fine grained
				-		
				4 --		
				-		
5	80	15.8	115.2	5 --		
				-	ML	ALLUVIUN: Sandy Clay, dark and medium brown, moist, stiff
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	40	17.4	107.9	10 --		
	50/5"			-	CL	Sandy Clay, dark and yellowish brown, moist, very stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	42	24.6	101.0	15 --		
	50/5"			-		BEDROCK (PUENTE FORMATION): Siltstone to Sandstone, gray and orange brown, moist, moderately hard
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	46	21.4	103.2	20 --		
	50/3"			-		Total Depth 20 feet No Water Fill to 5 feet
				21 --		
				-		
				22 --		
				-		
				23 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				-		
				24 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				25 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted

LOG OF TEST PIT NUMBER 1

Centennial GH Partners **Drilling Date: 07/16/25** **Elevation: 360.0'**

File No. 22662 **Method: Hand Dig Auger**

km

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
			0 --		FILL: Sandy Silt to Silty Sand, dark and yellowish brown, moist, medium dense, stiff, fine grained
			1 --		
			2 --		
3	9.6	94.7	3 --		
			4 --		
5	14.7	89.9	5 --		Sandy Silt, yellowish brown, moist, stiff, occasional rock fragments
			6 --		
7	15.1	103.2	7 --		
			8 --	CL	ALLUVIUM: Sandy Clay, dark brown, moist, stiff
			9 --		
10	16.7	99.6	10 --		BEDROCK (PUENTE FORMATION): Siltstone, yellow and grayish brown, moist, moderately hard
			11 --		
			12 --		
12.5	17.4	97.3	13 --		
			14 --		Total Depth 13 feet No Water Fill to 7 feet
			15 --		
			16 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
			17 --		
			18 --		Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			19 --		
			20 --		
			21 --		
			22 --		
			23 --		
			24 --		
			25 --		

LOG OF TEST PIT NUMBER 2

Centennial GH Partners **Drilling Date: 07/16/25** **Elevation: 353.0'**

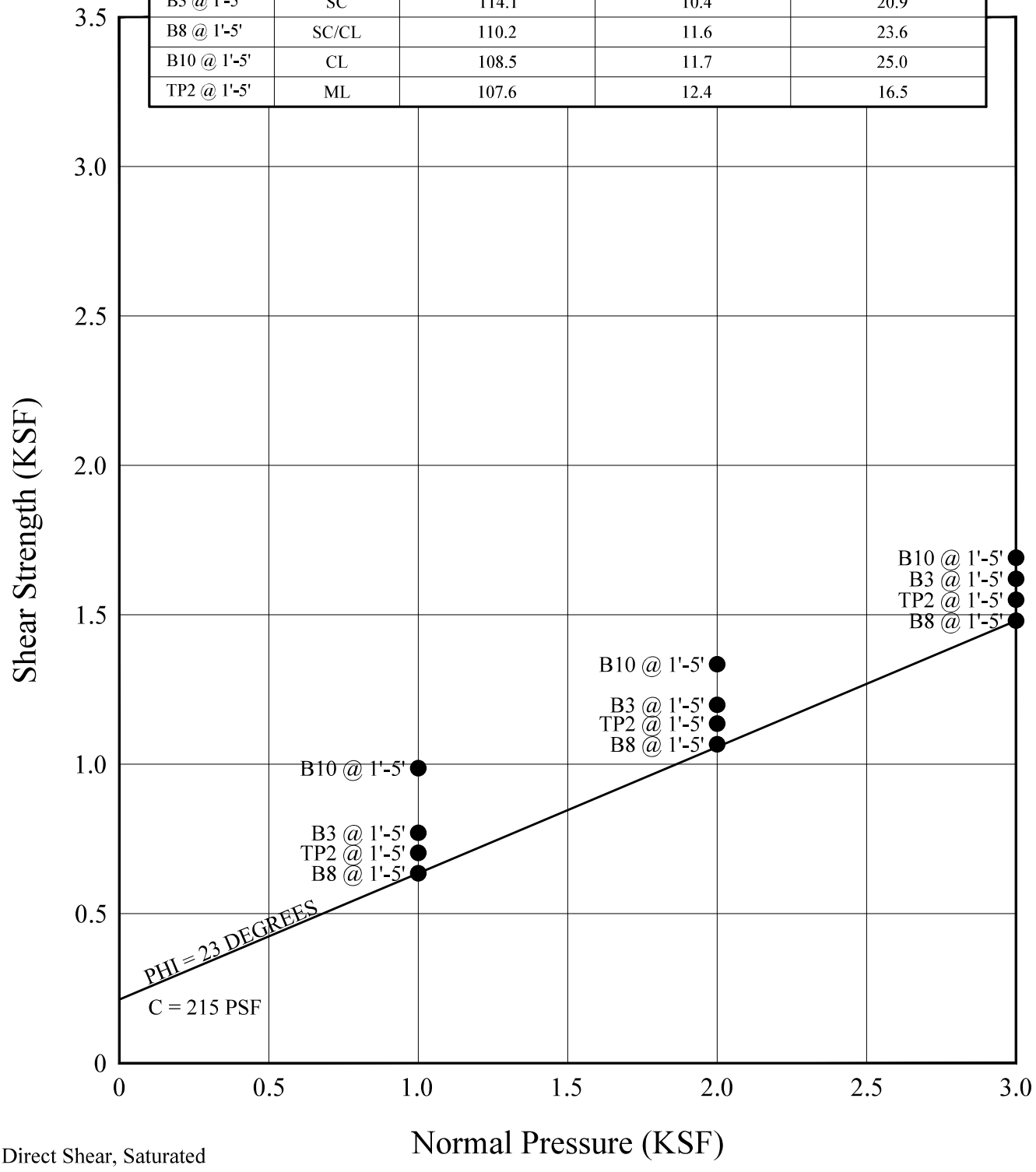
File No. 22662 **Method: Hand Dig Auger**

km

Sample Depth ft.	Moisture Content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
			0 --		FILL: Silty Sand, dark brown, moist, medium dense, fine grained
			-		
1	6.6	87.3	1 --		
			-		
			2 --		
			-		
3	11.3	101.0	3 --		
			-		Sandy Silt, dark and yellowish brown, moist, stiff, occasional rock fragments
			4 --		
			-		
5	11.3	Disturbed	5 --		
			-		Clayey Sand, dark and yellowish brown, moist, medium dense, fine grained
			6 --		
			-		
7	10.0	99.0	7 --		
			-		Sandy Silt, yellowish brown, moist, stiff, occasional brick fragments
			8 --		
			-		
			9 --		
			-		
10	13.6	97.1	10 --		
			-		
			11 --		
			-		
			12 --		
			-		
			13 --		
			-		
			14 --		
			-		
15	19.9	91.3	15 --		
			-		Sandy Silt, yellowish brown, moist, stiff, occasional rock fragments
			16 --		
			-		
			17 --		
			-		
			18 --		
			-		
			19 --		
			-		
20	15.7	99.6	20 --		
			-		Total Depth 20 feet No Water Fill to 20 feet
			21 --		
			-		
			22 --		
			-		
			23 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
			-		
			24 --		
			-		Used 4-inch diameter Hand-Augering Equipment; Hand Sampler
			25 --		
			-		

BULK SAMPLE REMOLDED TO 90 PERCENT OF THE MAXIMUM LABORATORY DENSITY

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B3 @ 1'-5'	SC	114.1	10.4	20.9
B8 @ 1'-5'	SC/CL	110.2	11.6	23.6
B10 @ 1'-5'	CL	108.5	11.7	25.0
TP2 @ 1'-5'	ML	107.6	12.4	16.5



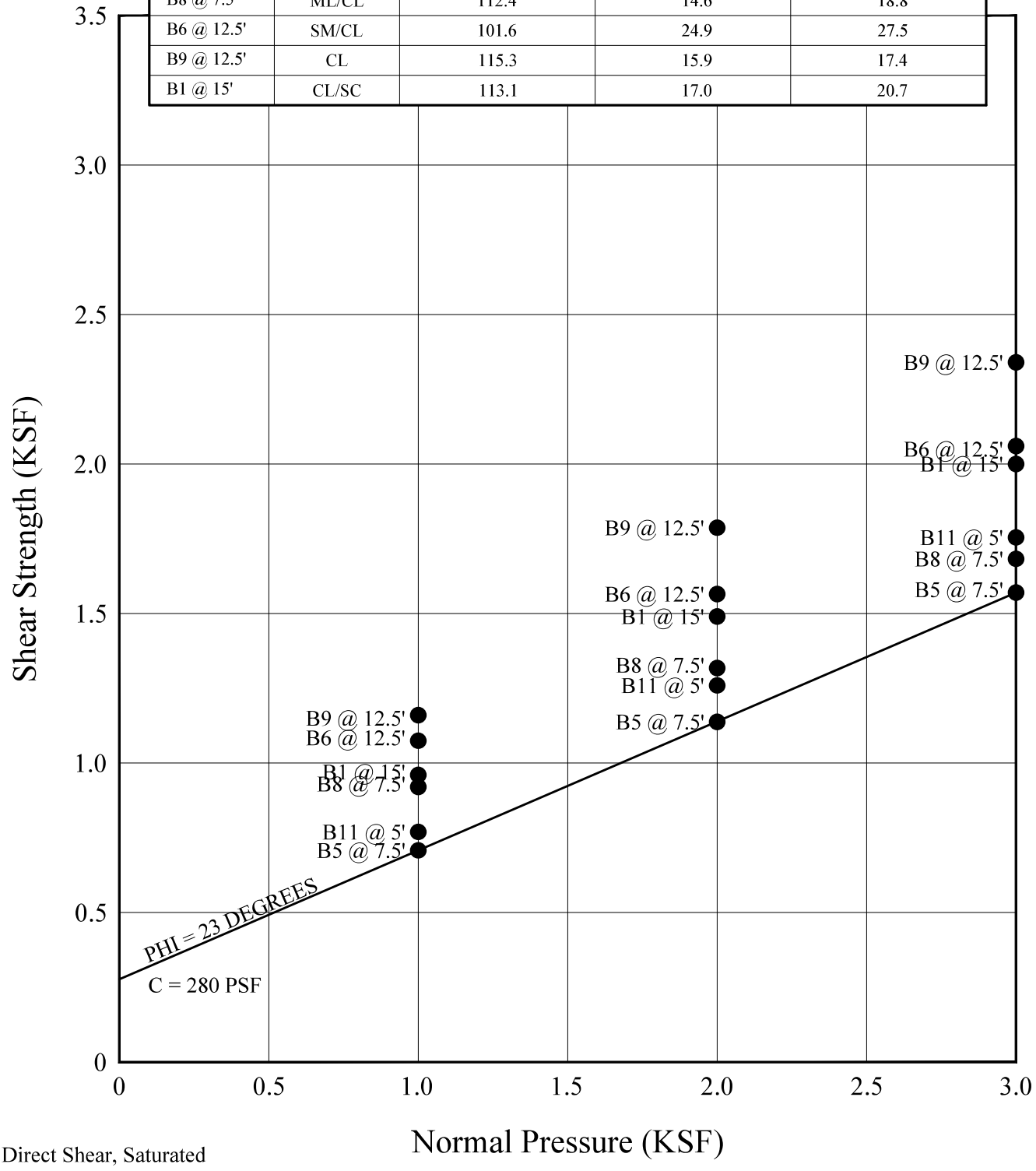
SHEAR TEST DIAGRAM (ASTM D3080)

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTENNIAL GH PARTNERS

FILE NO: 22662 | PLATE: B-1

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B11 @ 5'	ML	115.2	15.8	19.0
B5 @ 7.5'	CL	110.0	19.3	21.0
B8 @ 7.5'	ML/CL	112.4	14.6	18.8
B6 @ 12.5'	SM/CL	101.6	24.9	27.5
B9 @ 12.5'	CL	115.3	15.9	17.4
B1 @ 15'	CL/SC	113.1	17.0	20.7



SHEAR TEST DIAGRAM (ASTM D3080)

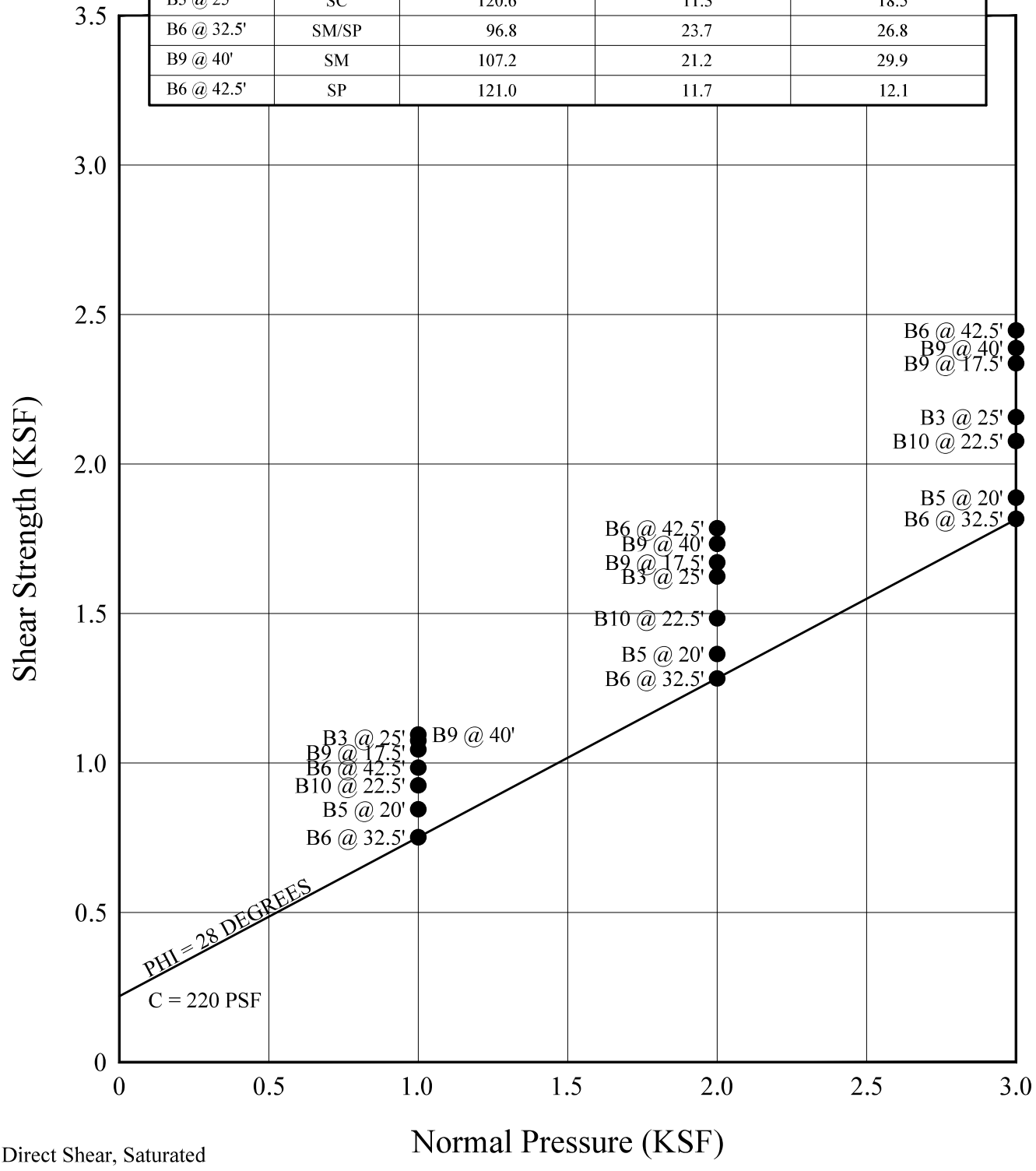
Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTENNIAL GH PARTNERS

FILE NO: 22662

PLATE: B-2

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B9 @ 17.5'	SM	110.0	14.6	16.4
B5 @ 20'	SC	104.6	19.6	18.7
B10 @ 22.5'	SM/SP	111.0	19.5	23.3
B3 @ 25'	SC	120.6	11.3	18.5
B6 @ 32.5'	SM/SP	96.8	23.7	26.8
B9 @ 40'	SM	107.2	21.2	29.9
B6 @ 42.5'	SP	121.0	11.7	12.1



SHEAR TEST DIAGRAM (ASTM D3080)

Geotechnologies, Inc.
Consulting Geotechnical Engineers

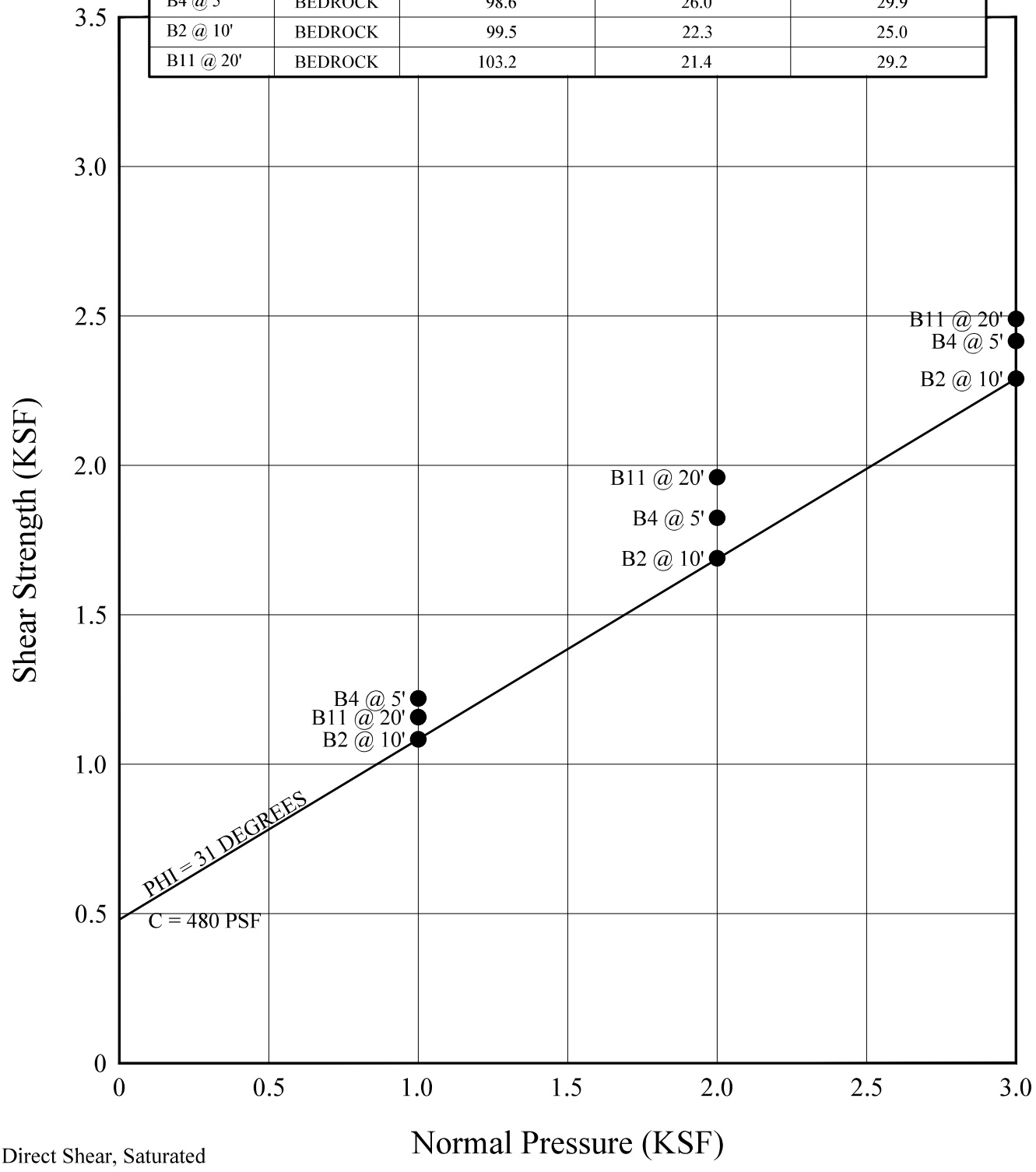
CENTENNIAL GH PARTNERS

FILE NO: 22662

PLATE: B-3

BEDROCK

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4 @ 5'	BEDROCK	98.6	26.0	29.9
B2 @ 10'	BEDROCK	99.5	22.3	25.0
B11 @ 20'	BEDROCK	103.2	21.4	29.2



SHEAR TEST DIAGRAM (ASTM D3080)

Geotechnologies, Inc.
Consulting Geotechnical Engineers

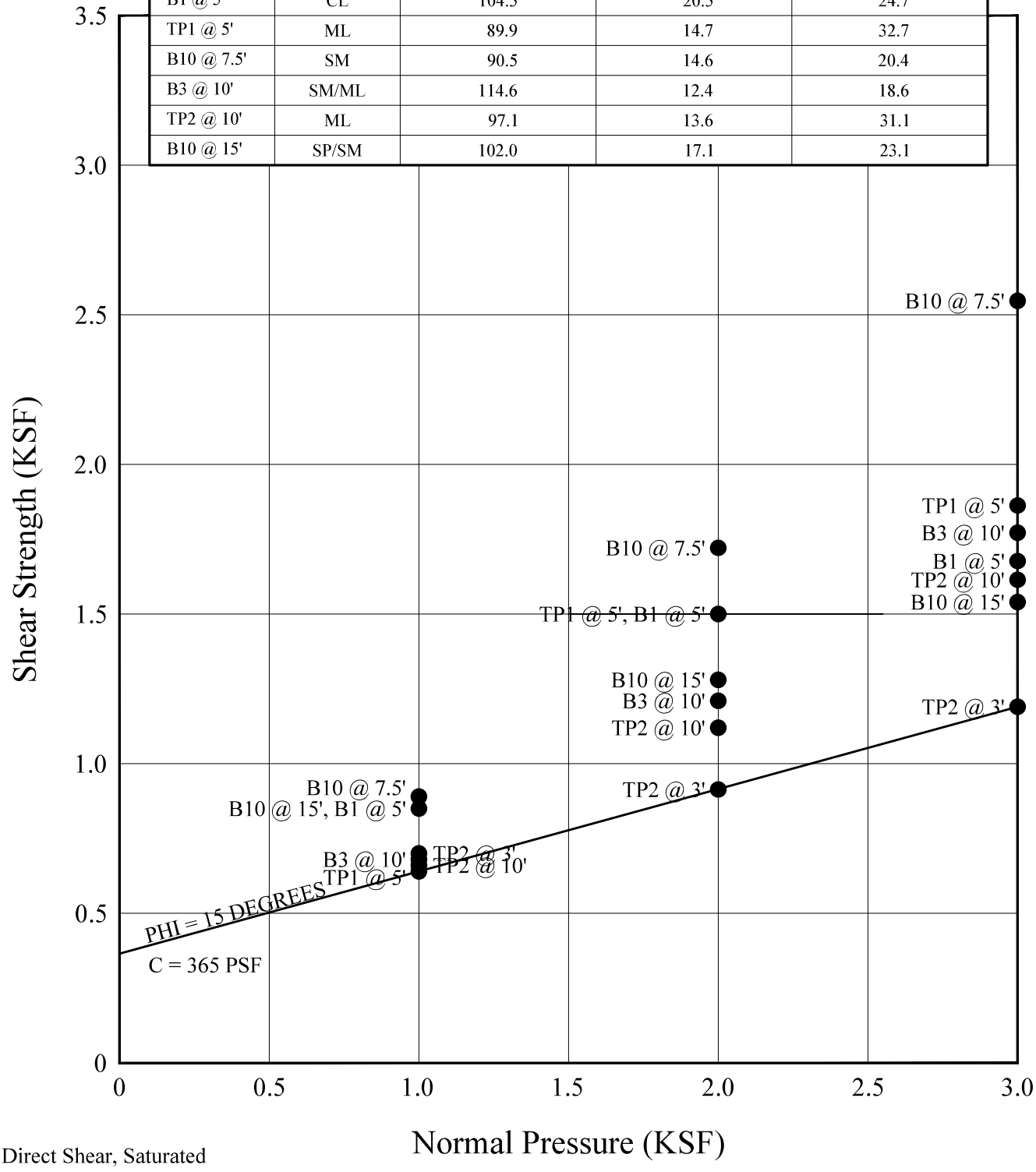
CENTENNIAL GH PARTNERS

FILE NO: 22662

PLATE: B-4

FILL

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
TP2 @ 3'	ML	101.0	11.3	27.1
B1 @ 5'	CL	104.5	20.5	24.7
TP1 @ 5'	ML	89.9	14.7	32.7
B10 @ 7.5'	SM	90.5	14.6	20.4
B3 @ 10'	SM/ML	114.6	12.4	18.6
TP2 @ 10'	ML	97.1	13.6	31.1
B10 @ 15'	SP/SM	102.0	17.1	23.1



SHEAR TEST DIAGRAM (ASTM D3080)

Geotechnologies, Inc.
Consulting Geotechnical Engineers

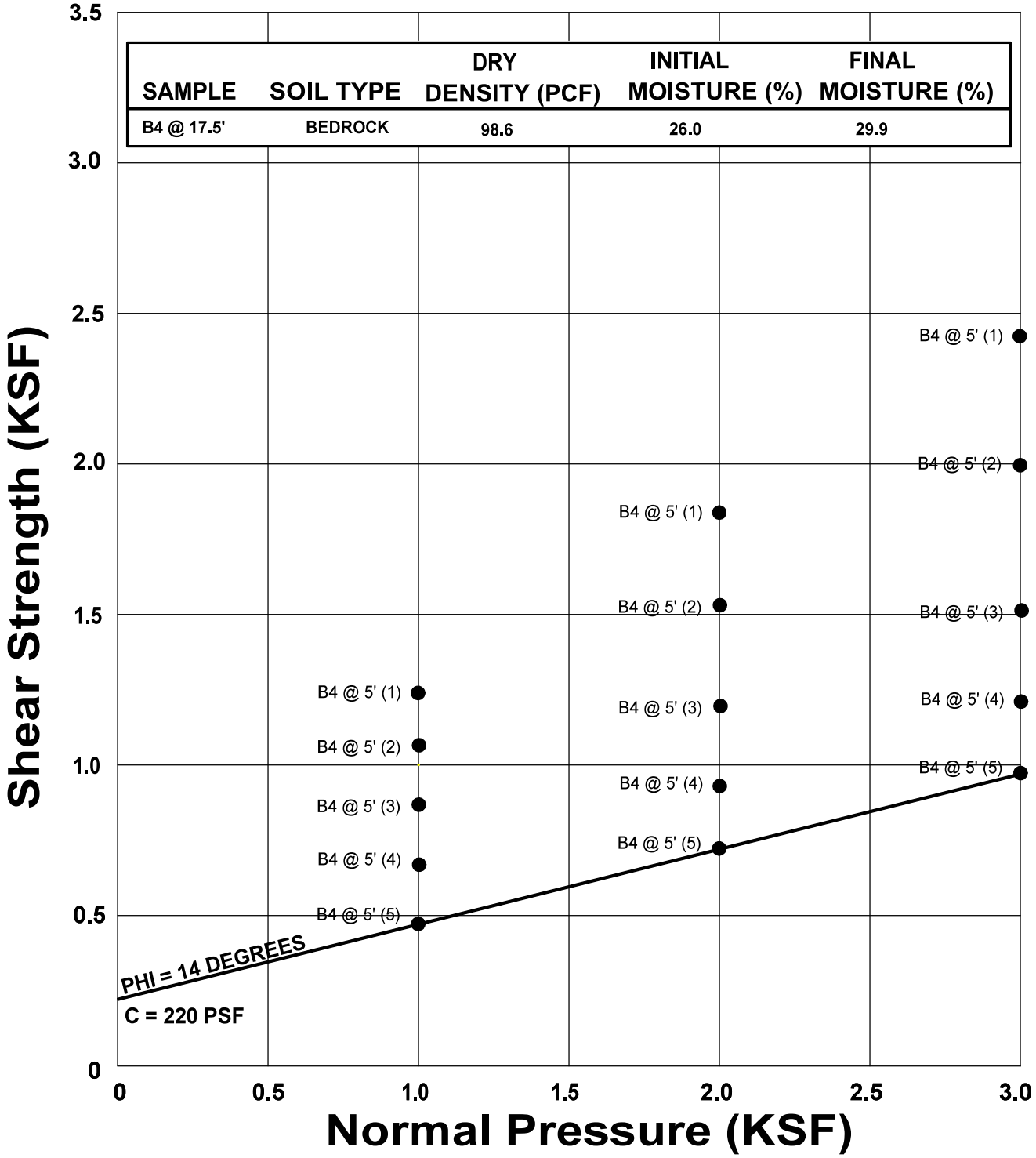
CENTENNIAL GH PARTNERS

FILE NO: 22662 | PLATE: B-5

BEDROCK (REPEAT SHEARING)

REMOLDED SAMPLE RESHEARED 5 TIMES

SAMPLE	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4 @ 17.5'	BEDROCK	98.6	26.0	29.9



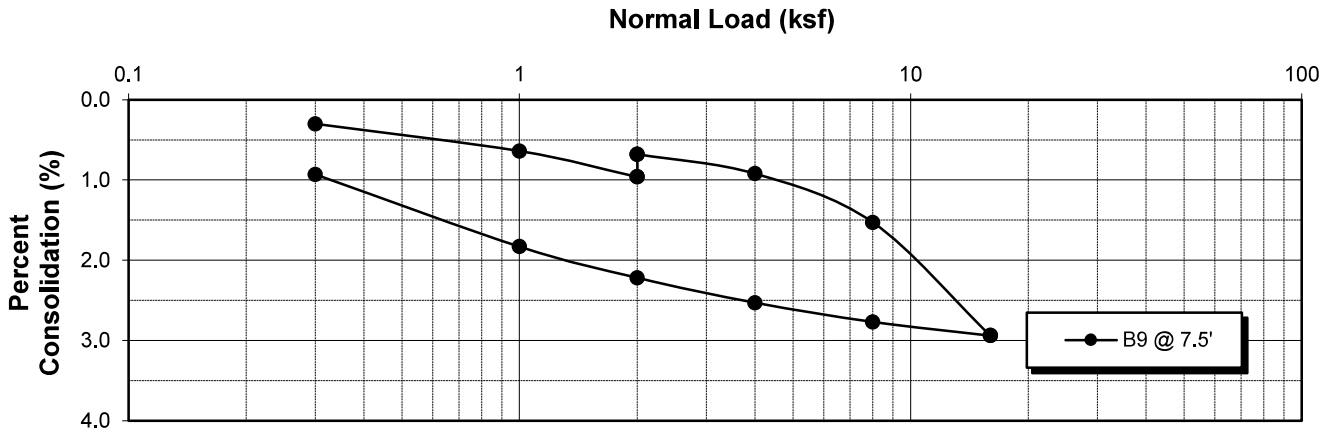
● Direct Shear, Saturated

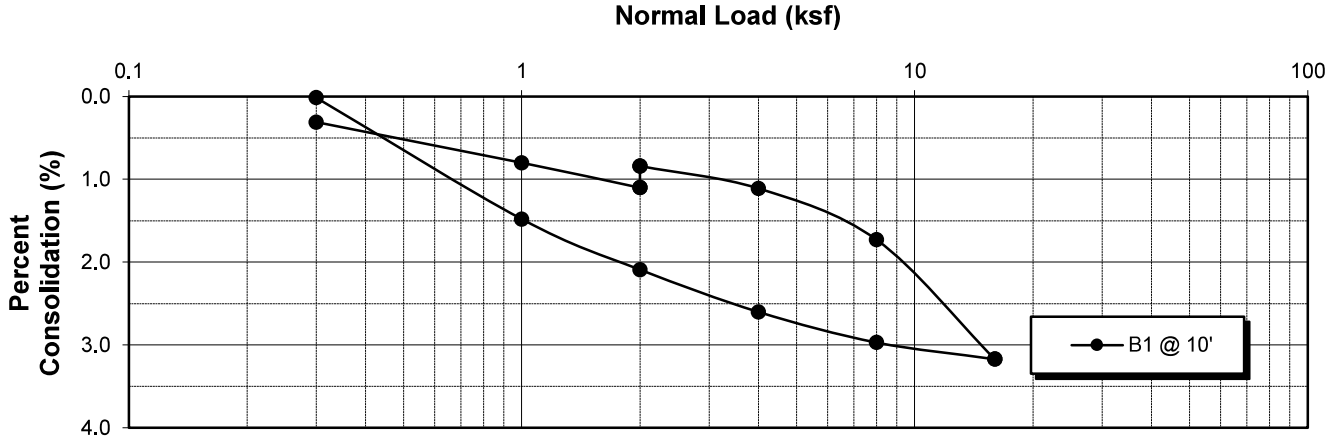
SHEAR TEST DIAGRAM

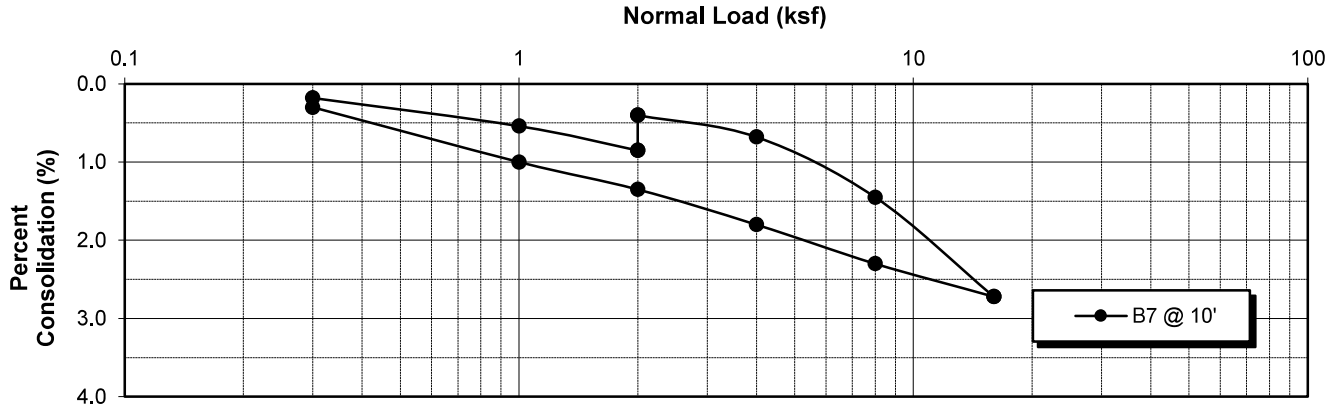
Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTENNIAL GH PARTNERS

FILE NO. 22662 | PLATE: B-6







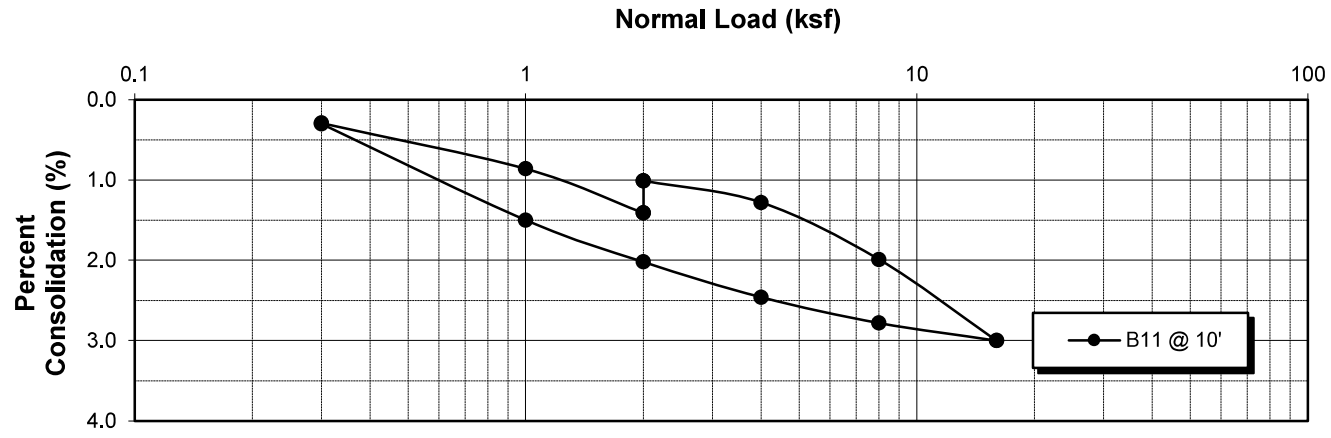
Water added at 2 KSF

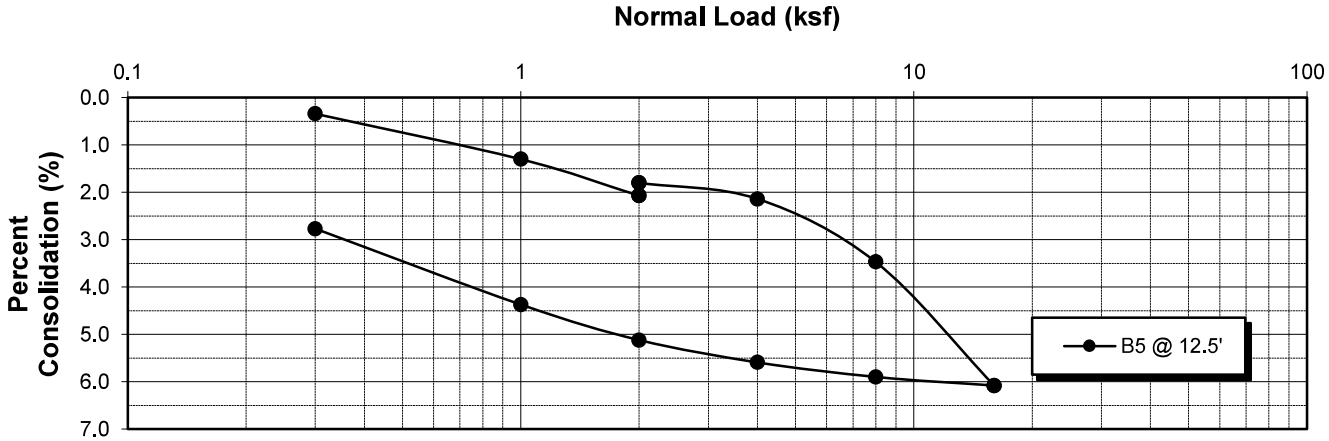
CONSOLIDATION (ASTM D2435)

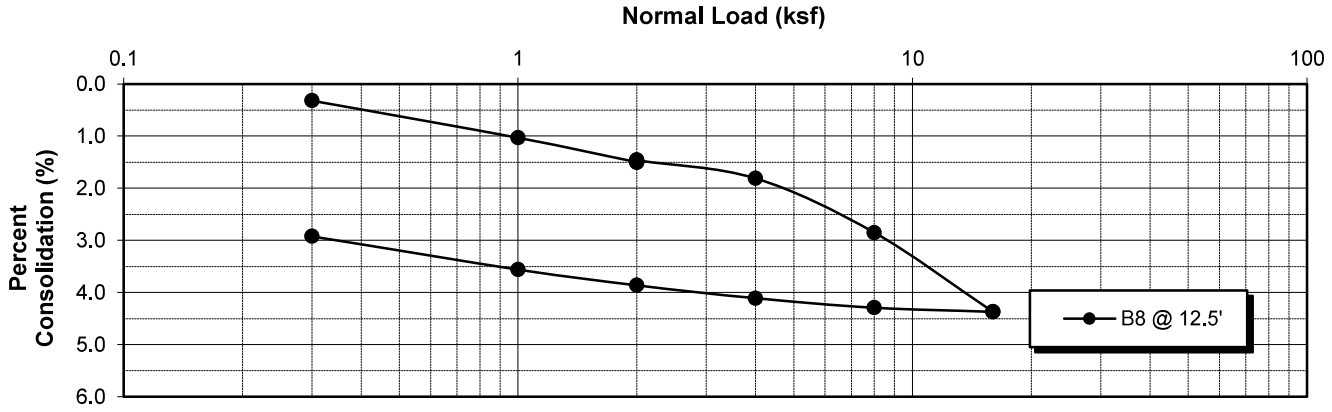
Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: CENTENNIAL GH PARTNERS

FILE NO.: 22662 | PLATE: C-1







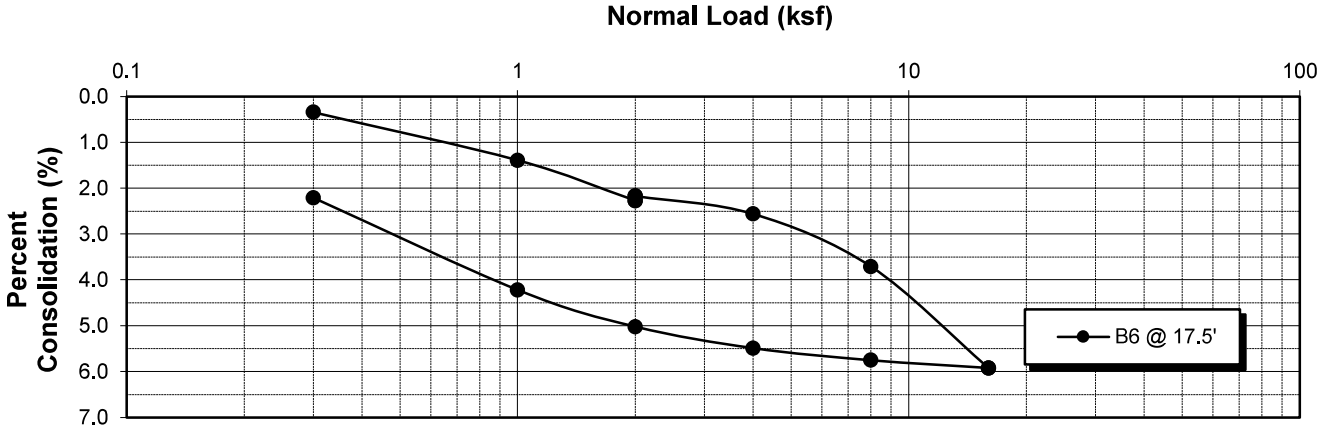
Water added at 2 KSF

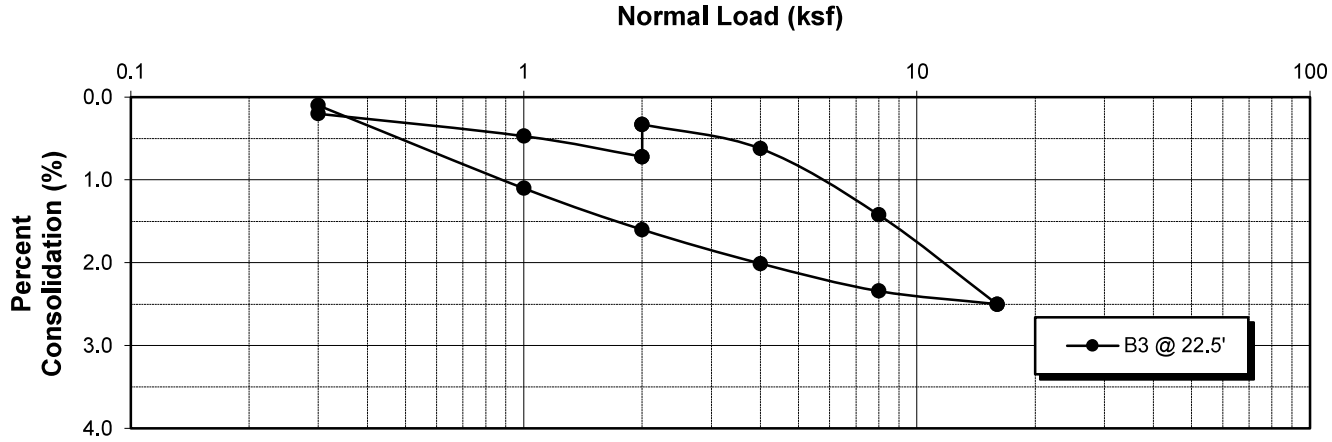
CONSOLIDATION(ASTM D2435)

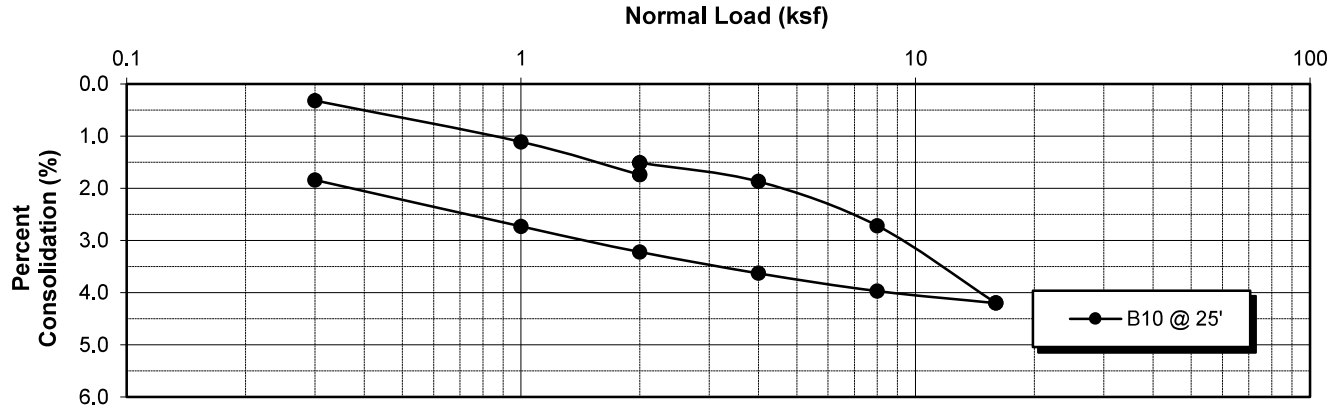
Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: CENTENNIAL GH PARTNERS

FILE NO.: 22662 | PLATE: C-2







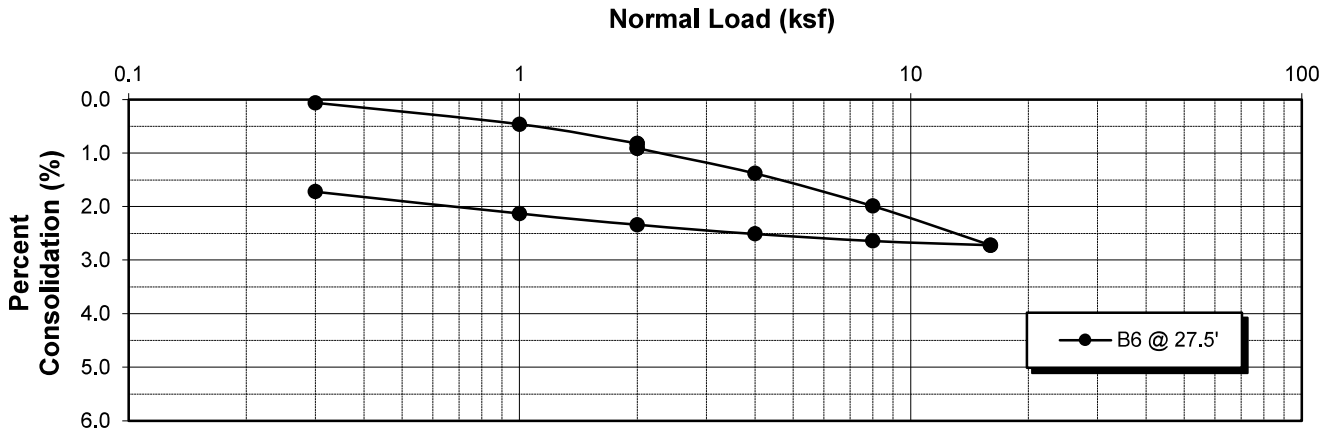
Water added at 2 KSF

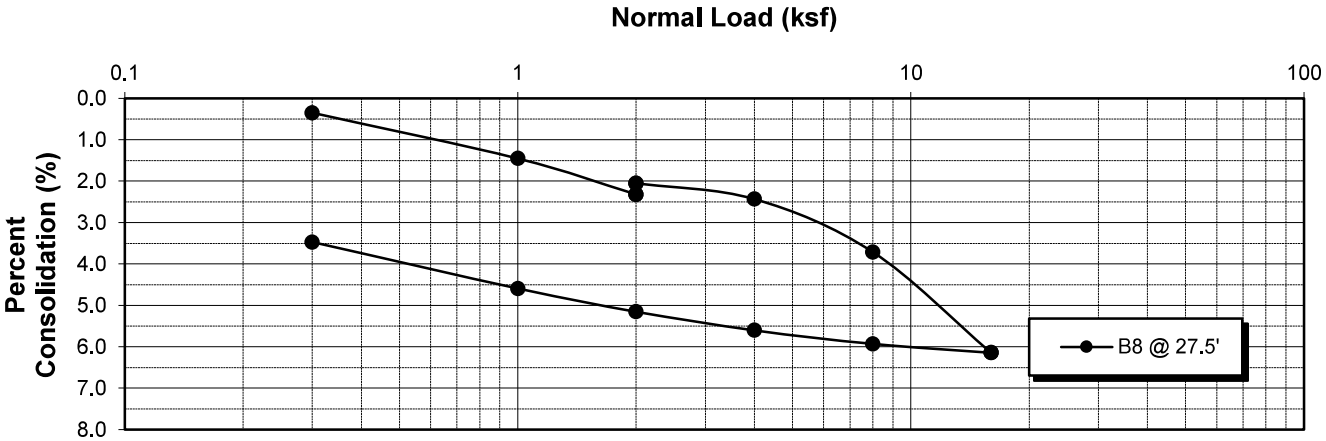
CONSOLIDATION(ASTM D2435)

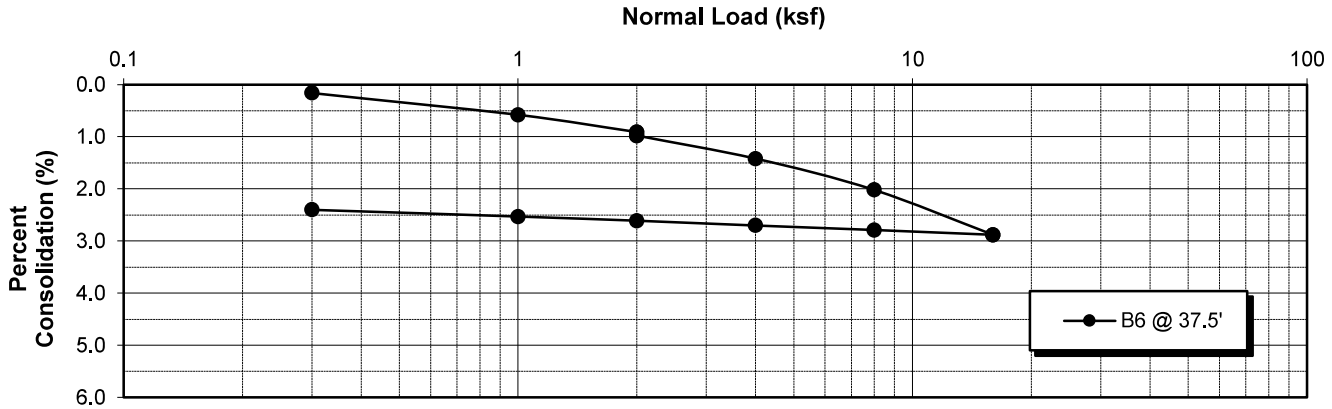
Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: CENTENNIAL GH PARTNERS

FILE NO.: 22662 | PLATE: C-3







Water added at 2 KSF

CONSOLIDATION(ASTM D2435)

Geotechnologies, Inc.
Consulting Geotechnical Engineers

PROJECT: CENTENNIAL GH PARTNERS

FILE NO.: 22662 | PLATE: C-4

LABORATORY COMPACTION CHARACTERISTICS
(ASTM D1557)

SAMPLE	B3 @ 1'-5'	B8 @ 1'-5'	B10 @ 1'-5'	TP2 @ 1'-5'
SOIL TYPE	SC	SC/CL	CL	ML
MAXIMUM DRY DENSITY (PCF)	126.8	122.4	120.5	119.6
OPTIMUM MOISTURE %	10.4	11.6	11.7	12.4

EXPANSION INDEX
(ASTM D4829)

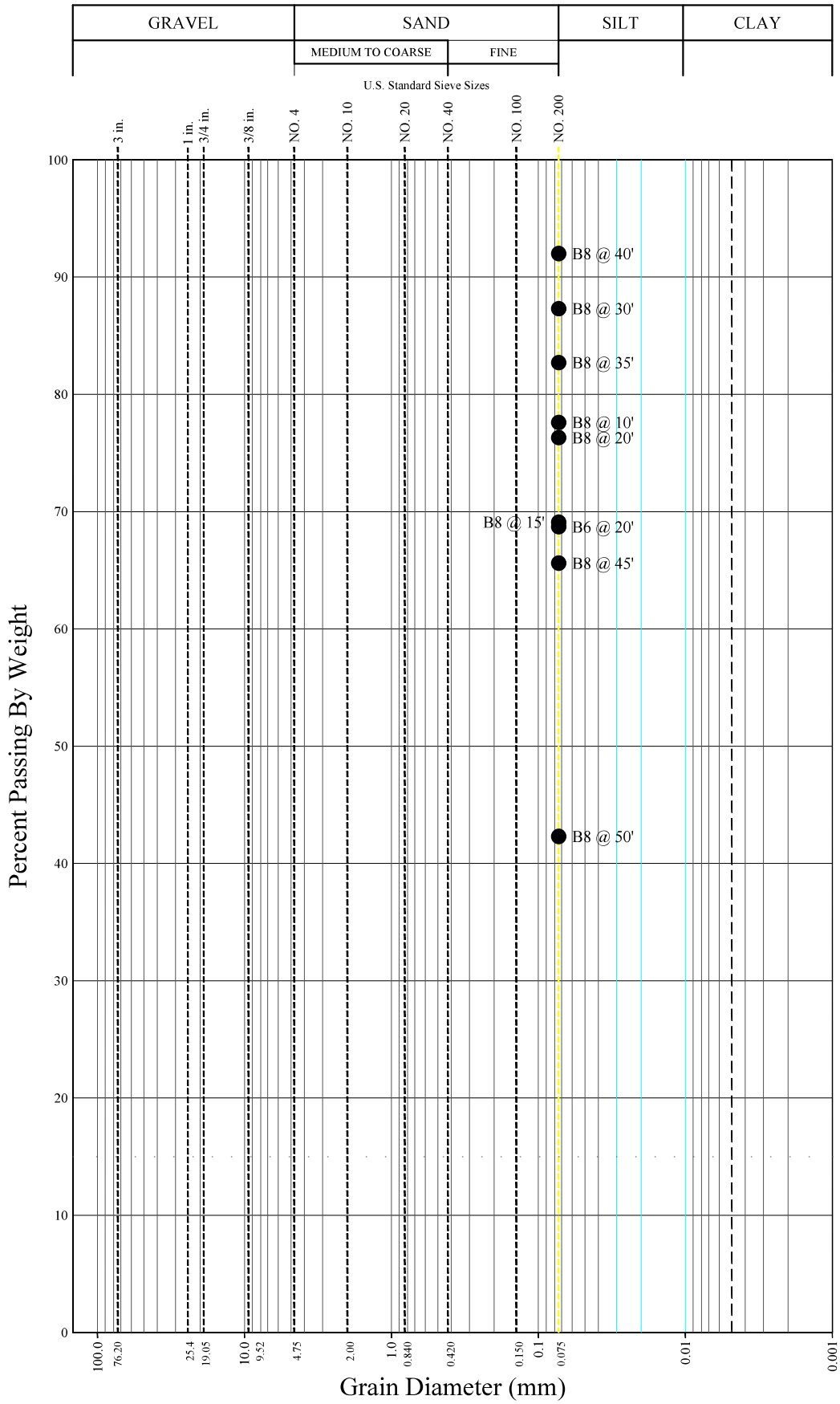
SAMPLE	B3 @ 1'-5'	B8 @ 1'-5'	B10 @ 1'-5'	TP2 @ 1'-5'
SOIL TYPE	SC	SC/CL	CL	ML
EXPANSION INDEX UBC STANDARD 18-2	50	74	99	74
EXPANSION CHARACTER	MODERATE	MODERATE	HIGH	MODERATE

COMPACTION/EXPANSION DATA SHEET

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTENNIAL GH PARTNERS

FILE NO: 22662 | PLATE: D



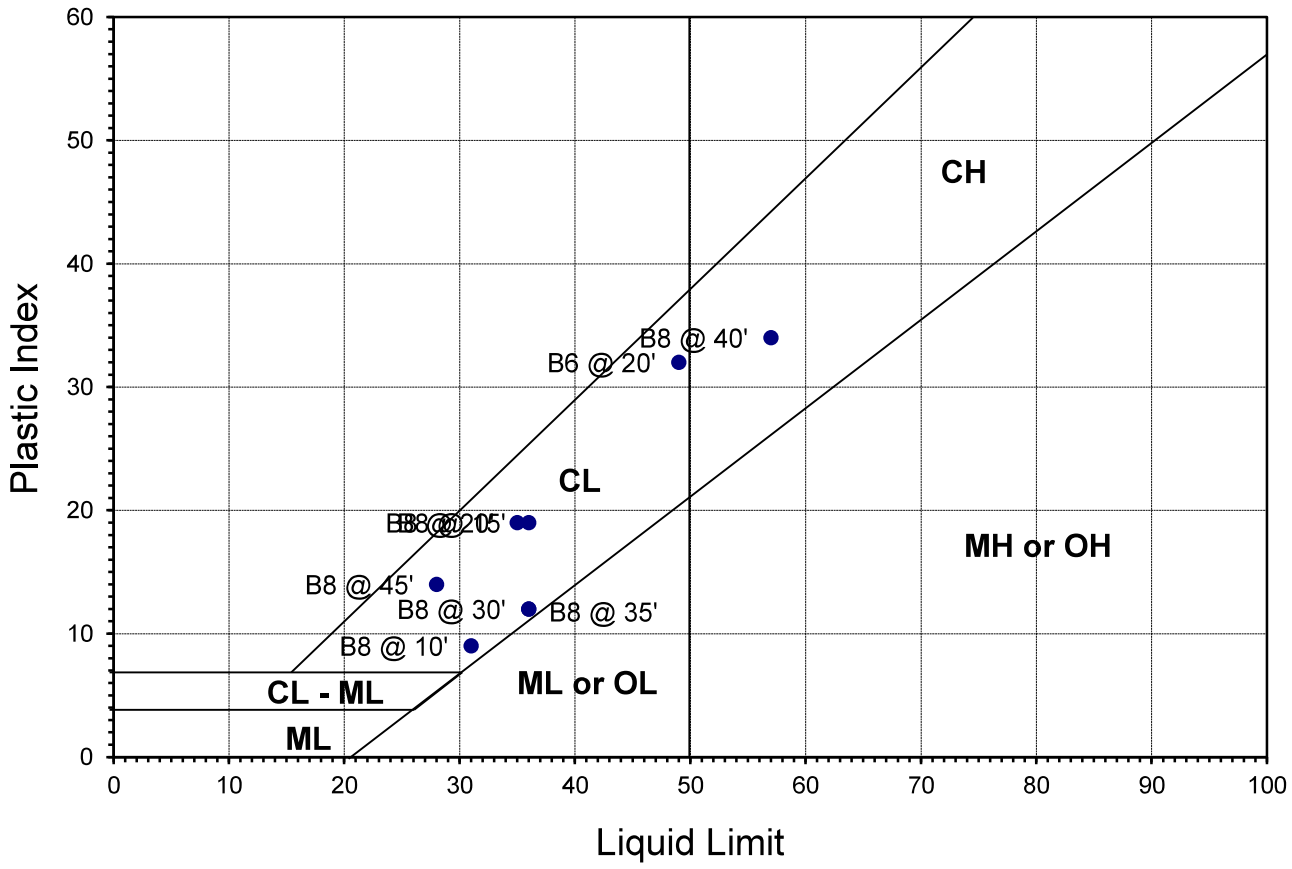
GRAIN SIZE DISTRIBUTION (ASTM D1140)

Geotechnologies, Inc.
Consulting Geotechnical Engineers

CENTENNIAL GH PARTNERS

FILE NO: 22662

PLATE: E



Sample ID	Descriptions	Passing #200	Liquid Limit	Plastic Limit	Plastic Index
B8 @ 10'	CL	77.6	31.0	22.0	9.0
B8 @ 15'	CL	69.0	36.0	17.0	19.0
B6 @ 20'	CH	68.7	49.0	17.0	32.0
B8 @ 20'	CH	76.3	35.0	16.0	19.0
B8 @ 30'	CH	87.3	36.0	24.0	12.0
B8 @ 35'	CH	82.7	36.0	24.0	12.0
B8 @ 40'	CH	92.0	57.0	23.0	34.0
B8 @ 45'	CL	65.6	28.0	14.0	14.0

ATTERBERG LIMITS (ASTM D4318)

Geotechnologies, Inc.

CONSULTING GEOTECHNICAL ENGINEERS

PROJECT: CENTENNIAL GH PARTNERS

FILE NO.: 22662

PLATE: F-1

Geotechnologies, Inc.

Project: Centennial GH Partners
File No.: 22662
Description: Liquefaction Analysis
Boring No: B6

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.9
Peak Ground Horizontal Acceleration, PGA (g):	0.94
Calculated Mag.Wtg.Factor:	1.171

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	26.5
Historically Highest Groundwater Level* (ft):	20.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):	4.3
SPT Sampler with room for Liner (Y/N):	Y

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1.3

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc} (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60\text{-}cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Mag. Scaling Factor (Sand) MSF	Overburden Corr. Factor K_σ	Cyclic Resist. Ratio $CRR_{M7.5,\sigma'vc=1}$	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Lateral Disp. Index ΔLDI_i (feet)	Vertical Strain ε_v	Liquefaction Settlment ΔS_i (inches)
1	128.1	Unsaturated	Unsaturated	14	5	0.0	0	128.1	128.1	27.3	1.00	0.614	1.17	1.10	0.357	0.459	Non-Liq.	0.0	0.00	0.00
2	128.1	Unsaturated	Unsaturated	14	5	0.0	0	256.2	256.2	27.3	1.00	0.612	1.17	1.10	0.357	0.459	Non-Liq.	0.0	0.00	0.00
3	128.1	Unsaturated	Unsaturated	14	5	0.0	0	384.3	384.3	27.3	1.00	0.610	1.17	1.10	0.357	0.459	Non-Liq.	0.0	0.00	0.00
4	128.1	Unsaturated	Unsaturated	14	5	0.0	0	512.4	512.4	27.3	0.99	0.608	1.17	1.10	0.357	0.459	Non-Liq.	0.0	0.00	0.00
5	128.1	Unsaturated	Unsaturated	14	5	0.0	0	640.5	640.5	27.3	0.99	0.606	1.17	1.10	0.358	0.462	Non-Liq.	0.0	0.00	0.00
6	128.1	Unsaturated	Unsaturated	14	5	0.0	0	768.6	768.6	25.7	0.99	0.603	1.17	1.10	0.307	0.396	Non-Liq.	0.0	0.00	0.00
7	128.1	Unsaturated	Unsaturated	14	5	0.0	0	896.7	896.7	24.1	0.98	0.601	1.17	1.10	0.270	0.348	Non-Liq.	0.0	0.00	0.00
8	120.3	Unsaturated	Unsaturated	15	10	0.0	0	1017.0	1017.0	24.5	0.98	0.599	1.17	1.10	0.278	0.358	Non-Liq.	0.0	0.00	0.00
9	120.3	Unsaturated	Unsaturated	15	10	0.0	0	1137.3	1137.3	24.8	0.98	0.596	1.17	1.10	0.286	0.368	Non-Liq.	0.0	0.00	0.00
10	120.3	Unsaturated	Unsaturated	15	10	0.0	0	1257.6	1257.6	23.7	0.97	0.594	1.17	1.08	0.263	0.333	Non-Liq.	0.0	0.00	0.00
11	120.3	Unsaturated	Unsaturated	15	10	0.0	0	1377.9	1377.9	22.8	0.97	0.591	1.17	1.06	0.245	0.306	Non-Liq.	0.0	0.00	0.00
12	120.3	Unsaturated	Unsaturated	15	10	0.0	0	1498.2	1498.2	21.9	0.96	0.589	1.17	1.05	0.232	0.285	Non-Liq.	0.0	0.00	0.00
13	126.9	Unsaturated	Unsaturated	15	10	0.0	0	1625.1	1625.1	21.1	0.96	0.586	1.17	1.04	0.220	0.266	Non-Liq.	0.0	0.00	0.00
14	126.9	Unsaturated	Unsaturated	15	10	0.0	0	1752.0	1752.0	20.3	0.95	0.583	1.17	1.03	0.210	0.252	Non-Liq.	0.0	0.00	0.00
15	126.9	Unsaturated	Unsaturated	15	10	0.0	0	1878.9	1878.9	22.2	0.95	0.581	1.17	1.02	0.237	0.282	Non-Liq.	0.0	0.00	0.00
16	126.9	Unsaturated	Unsaturated	17	15	0.0	0	2005.8	2005.8	24.9	0.95	0.578	1.17	1.01	0.287	0.339	Non-Liq.	0.0	0.00	0.00
17	126.9	Unsaturated	Unsaturated	17	15	0.0	0	2132.7	2132.7	24.1	0.94	0.575	1.17	1.00	0.271	0.317	Non-Liq.	0.0	0.00	0.00
18	124.3	Unsaturated	Unsaturated	17	15	0.0	0	2257.0	2257.0	23.5	0.94	0.572	1.17	0.99	0.258	0.299	Non-Liq.	0.0	0.00	0.00
19	124.3	Unsaturated	Unsaturated	17	15	0.0	0	2381.3	2381.3	22.9	0.93	0.569	1.17	0.98	0.247	0.284	Non-Liq.	0.0	0.00	0.00
20	124.3	Unsaturated	Unsaturated	17	15	0.0	0	2505.6	2505.6	22.3	0.93	0.566	1.17	0.97	0.237	0.271	Non-Liq.	0.0	0.00	0.00
21	124.3	Unsaturated	Saturated	17	20	68.7	32	2629.9	2567.5	27.3	0.92	0.576	1.17	0.96	0.357	0.402	Non-Liq.	0.0	0.00	0.00
22	124.3	Unsaturated	Saturated	17	20	68.7	32	2754.2	2629.4	26.8	0.92	0.586	1.17	0.95	0.339	0.379	Non-Liq.	0.0	0.00	0.00
23	132.3	Unsaturated	Saturated	100	25	0.0	0	2886.5	2699.3	142.1	0.91	0.595	1.17	0.91	2.000	2.000	3.4	0.0	0.00	0.00
24	132.3	Unsaturated	Saturated	100	25	0.0	0	3018.8	2769.2	140.5	0.90	0.603	1.17	0.89	2.000	2.000	3.3	0.0	0.00	0.00
25	132.3	Unsaturated	Saturated	100	25	0.0	0	3151.1	2839.1	138.9	0.90	0.610	1.17	0.88	2.000	2.000	3.3	0.0	0.00	0.00
26	132.3	Unsaturated	Saturated	100	25	0.0	0	3283.4	2909.0	137.4	0.89	0.617	1.17	0.87	2.000	2.000	3.2	0.0	0.00	0.00
27	132.3	Saturated	Saturated	100	25	0.0	0	3415.7	2978.9	136.3	0.89	0.623	1.17	0.86	2.000	2.000	3.2	0.0	0.00	0.00
28	128.3	Saturated	Saturated	69	30	0.0	0	3544.0	3044.8	98.5	0.88	0.628	1.17	0.85	2.000	2.000	3.2	0.0	0.00	0.00
29	128.3	Saturated	Saturated	69	30	0.0	0	3672.3	3110.7	98.0	0.88	0.633	1.17	0.85	2.000	1.990	3.1	0.0	0.00	0.00
30	128.3	Saturated	Saturated	69	30	0.0	0	3800.6	3176.6	97.5	0.87	0.637	1.17	0.84	2.000	1.977	3.1	0.0	0.00	0.00
31	128.3	Saturated	Saturated	69	30	0.0	0	3928.9	3242.5	97.1	0.87	0.641	1.17	0.84	2.000	1.964	3.1	0.0	0.00	0.00
32	128.3	Saturated	Saturated	69	30	0.0	0	4057.2	3308.4	96.6	0.86	0.645	1.17	0.83	2.000	1.952	3.0	0.0	0.00	0.00
33	119.8	Saturated	Saturated	69	30	0.0	0	4177.0	3365.8	96.2	0.85	0.648	1.17	0.83	2.000	1.941	3.0	0.0	0.00	0.00
34	119.8	Saturated	Saturated	69	30	0.0	0	4296.8	3423.2	95.8	0.85	0.651	1.17	0.82	2.000	1.931	3.0	0.0	0.00	0.00
35	119.8	Saturated	Saturated	69	30	0.0	0	4416.6	3480.6	95.5	0.84	0.654	1.17	0.82	2.000	1.920	2.9	0.0	0.00	0.00
36	119.8	Saturated	Saturated	85	35	0.0	0	4536.4	3538.0	117.2	0.84	0.656	1.17	0.82	2.000	1.910	2.9	0.0	0.00	0.00
37	119.8	Saturated	Saturated	85	35	0.0	0	4656.2	3595.4	116.7	0.83	0.658	1.17	0.81	2.000	1.900	2.9	0.0	0.00	0.00
38	132.3	Saturated	Saturated	85	35	0.0	0	4788.5	3665.3	116.2	0.83	0.659	1.17	0.81	2.000	1.888	2.9	0.0	0.00	0.00
39	132.3	Saturated	Saturated	85	35	0.0	0	4920.8	3735.2	115.7	0.82	0.660	1.17	0.80	2.000	1.877	2.8	0.0	0.00	0.00
40	132.3	Saturated	Saturated	85	35	0.0	0	5053.1	3805.1	115.2	0.81	0.661	1.17	0.80	2.000	1.865	2.8	0.0	0.00	0.00
41	132.3	Saturated	Saturated	100	40	0.0	0	5185.4	3875.0	134.9	0.81	0.661	1.17	0.79	2.000	1.854	2.8	0.0	0.00	0.00
42	132.3	Saturated	Saturated	100	40	0.0	0	5317.7	3944.9	134.3	0.80	0.661	1.17	0.79	2.000	1.842	2.8	0.0	0.00	0.00
43	135.1	Saturated	Saturated	100	40	0.0	0	5452.8	4017.6	133.7	0.80	0.661	1.17	0.78	2.000	1.831	2.8	0.0	0.00	0.00
44	135.1	Saturated	Saturated	100	40	0.0	0	5587.9	4090.3	133.2	0.79	0.661	1.17	0.78	2.000	1.820	2.8	0.0	0.00	0.00
45	135.1	Saturated	Saturated	100	40	0.0	0	5723.0	4163.0	132.6	0.79	0.660	1.17	0.77	2.000	1.809	2.7	0.0	0.00	0.00
46	135.1	Saturated	Saturated	100	45	0.0	0	5858.1	4235.7	132.1	0.78	0.659	1.17	0.77	2.000	1.798	2.7	0.0	0.00	0.00
47	135.1	Saturated	Saturated	100	45	0.0	0	5993.2	4308.4	131.5	0.77	0.658	1.17	0.76	2.000	1.787	2.7	0.0	0.00	0.00
48	134.9	Saturated	Saturated	100	45	0.0	0	6128.1	4380.9	131.0	0.77	0.657	1.17	0.76	2.000	1.776	2.7	0.0	0.00	0.00
49	134.9	Saturated	Saturated	100	45	0.0	0	6263.0	4453.4	130.5	0.76	0.655	1.17	0.75	2.000	1.766	2.7	0.0	0.00	0.00
50	134.9	Saturated	Saturated	100	50	0.0	0	6397.9	4525.9	130.0	0.76	0.654	1.17	0.75	2.000	1.756	2.7	0.0	0.00	0.00
																Total Liquefaction Settlement, S =		**0.00**		**inches**

Geotechnologies, Inc.

Project: Centennial GH Partners
File No.: 22662
Description: Liquefaction Analysis
Boring No: B8

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.9
Peak Ground Horizontal Acceleration, PGA (g):	0.94
Calculated Mag.Wtg.Factor:	1.171

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	43.0
Historically Highest Groundwater Level* (ft):	20.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):	4.3
SPT Sampler with room for Liner (Y/N):	Y

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1.3

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc} (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60\text{-}cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Mag. Scaling Factor (Sand) MSF	Overburden Corr. Factor K_σ	Cyclic Resist. Ratio $CRR_{M7.5,\sigma'vc=1}$	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Lateral Disp. Index ΔLDI_i (feet)	Vertical Strain ε_v	Liquefaction Settlment ΔS_i (inches)
1	128.8	Unsaturated	Unsaturated	21	5	0.0	0	128.8	128.8	43.5	1.00	0.614	1.17	1.10	2.000	2.000	Non-Liq.	0.0	0.00	0.00
2	128.8	Unsaturated	Unsaturated	21	5	0.0	0	257.6	257.6	43.5	1.00	0.612	1.17	1.10	2.000	2.000	Non-Liq.	0.0	0.00	0.00
3	128.8	Unsaturated	Unsaturated	21	5	0.0	0	386.4	386.4	41.7	1.00	0.610	1.17	1.10	2.000	2.000	Non-Liq.	0.0	0.00	0.00
4	128.8	Unsaturated	Unsaturated	21	5	0.0	0	515.2	515.2	39.2	0.99	0.608	1.17	1.10	2.000	2.000	Non-Liq.	0.0	0.00	0.00
5	128.8	Unsaturated	Unsaturated	21	5	0.0	0	644.0	644.0	39.1	0.99	0.606	1.17	1.10	2.000	2.000	Non-Liq.	0.0	0.00	0.00
6	128.8	Unsaturated	Unsaturated	21	5	0.0	0	772.8	772.8	37.4	0.99	0.603	1.17	1.10	1.950	2.000	Non-Liq.	0.0	0.00	0.00
7	128.8	Unsaturated	Unsaturated	21	5	0.0	0	901.6	901.6	35.9	0.98	0.601	1.17	1.10	1.360	1.752	Non-Liq.	0.0	0.00	0.00
8	128.8	Unsaturated	Unsaturated	21	5	0.0	0	1030.4	1030.4	34.3	0.98	0.599	1.17	1.10	0.966	1.244	Non-Liq.	0.0	0.00	0.00
9	128.8	Unsaturated	Unsaturated	21	5	0.0	0	1159.2	1159.2	35.0	0.98	0.596	1.17	1.10	1.116	1.437	Non-Liq.	0.0	0.00	0.00
10	128.8	Unsaturated	Unsaturated	21	5	0.0	0	1288.0	1288.0	33.6	0.97	0.594	1.17	1.10	0.851	1.097	Non-Liq.	0.0	0.00	0.00
11	128.8	Unsaturated	Unsaturated	24	10	0.0	0	1416.8	1416.8	37.6	0.97	0.591	1.17	1.10	2.000	2.000	Non-Liq.	0.0	0.00	0.00
12	128.8	Unsaturated	Unsaturated	24	10	0.0	0	1545.6	1545.6	36.3	0.96	0.589	1.17	1.09	1.489	1.898	Non-Liq.	0.0	0.00	0.00
13	124.3	Unsaturated	Unsaturated	24	10	0.0	0	1669.9	1669.9	35.3	0.96	0.586	1.17	1.06	1.169	1.454	Non-Liq.	0.0	0.00	0.00
14	124.3	Unsaturated	Unsaturated	24	10	0.0	0	1794.2	1794.2	34.3	0.95	0.583	1.17	1.04	0.954	1.163	Non-Liq.	0.0	0.00	0.00
15	124.3	Unsaturated	Unsaturated	24	10	0.0	0	1918.5	1918.5	38.0	0.95	0.581	1.17	1.03	2.000	2.000	Non-Liq.	0.0	0.00	0.00
16	124.3	Unsaturated	Unsaturated	20	15	0.0	0	2042.8	2042.8	29.8	0.95	0.578	1.17	1.01	0.474	0.559	Non-Liq.	0.0	0.00	0.00
17	124.3	Unsaturated	Unsaturated	20	15	0.0	0	2167.1	2167.1	29.0	0.94	0.575	1.17	0.99	0.431	0.502	Non-Liq.	0.0	0.00	0.00
18	129.0	Unsaturated	Unsaturated	14	20	76.3	19	2296.1	2296.1	24.1	0.94	0.572	1.17	0.99	0.270	0.312	Non-Liq.	0.0	0.00	0.00
19	129.0	Unsaturated	Unsaturated	14	20	76.3	19	2425.1	2425.1	23.6	0.93	0.569	1.17	0.98	0.260	0.298	Non-Liq.	0.0	0.00	0.00
20	129.0	Unsaturated	Unsaturated	14	20	76.3	19	2554.1	2554.1	23.1	0.93	0.566	1.17	0.97	0.251	0.285	Non-Liq.	0.0	0.00	0.00
21	129.0	Unsaturated	Saturated	14	20	76.3	19	2683.1	2620.7	22.6	0.92	0.576	1.17	0.96	0.243	0.274	Non-Liq.	0.0	0.00	0.00
22	129.0	Unsaturated	Saturated	14	20	76.3	19	2812.1	2687.3	22.2	0.92	0.585	1.17	0.96	0.236	0.265	Non-Liq.	0.0	0.00	0.00
23	135.5	Unsaturated	Saturated	32	25	0.0	0	2947.6	2760.4	45.2	0.91	0.594	1.17	0.90	2.000	2.000	3.4	0.0	0.00	0.00
24	135.5	Unsaturated	Saturated	32	25	0.0	0	3083.1	2833.5	44.6	0.90	0.602	1.17	0.89	2.000	2.000	3.3	0.0	0.00	0.00
25	135.5	Unsaturated	Saturated	32	25	0.0	0	3218.6	2906.6	44.0	0.90	0.609	1.17	0.88	2.000	2.000	3.3	0.0	0.00	0.00
26	135.5	Unsaturated	Saturated	32	25	0.0	0	3354.1	2979.7	43.4	0.89	0.615	1.17	0.86	2.000	2.000	3.3	0.0	0.00	0.00
27	135.5	Unsaturated	Saturated	32	25	0.0	0	3489.6	3052.8	42.9	0.89	0.621	1.17	0.85	2.000	1.995	3.2	0.0	0.00	0.00
28	122.9	Unsaturated	Saturated	25	30	87.3	12	3612.5	3113.3	38.0	0.88	0.626	1.17	0.84	2.000	1.971	3.1	0.0	0.00	0.00
29	122.9	Unsaturated	Saturated	25	30	87.3	12	3735.4	3173.8	37.5	0.88	0.631	1.17	0.83	1.973	1.922	3.0	0.0	0.00	0.00
30	122.9	Unsaturated	Saturated	25	30	87.3	12	3858.3	3234.3	37.0	0.87	0.636	1.17	0.82	1.734	1.670	2.6	0.0	0.00	0.00
31	122.9	Unsaturated	Saturated	25	30	87.3	12	3981.2	3294.8	36.5	0.87	0.640	1.17	0.82	1.539	1.476	2.3	0.0	0.00	0.00
32	122.9	Unsaturated	Saturated	25	30	87.3	12	4104.1	3355.3	36.0	0.86	0.643	1.17	0.82	1.379	1.316	2.0	0.0	0.00	0.00
33	126.5	Unsaturated	Saturated	25	30	87.3	12	4230.6	3419.4	35.5	0.85	0.646	1.17	0.81	1.241	1.181	1.8	0.0	0.00	0.00
34	126.5	Unsaturated	Saturated	25	30	87.3	12	4357.1	3483.5	35.1	0.85	0.649	1.17	0.81	1.126	1.067	1.6	0.0	0.00	0.00
35	126.5	Unsaturated	Saturated	25	30	87.3	12	4483.6	3547.6	34.6	0.84	0.651	1.17	0.81	1.029	0.972	1.5	0.0	0.00	0.00
36	126.5	Unsaturated	Saturated	28	35	82.7	12	4610.1	3611.7	39.4	0.84	0.653	1.17	0.77	2.000	1.802	2.8	0.0	0.00	0.00
37	126.5	Unsaturated	Saturated	28	35	82.7	12	4736.6	3675.8	39.0	0.83	0.655	1.17	0.76	2.000	1.784	2.7	0.0	0.00	0.00
38	127.9	Unsaturated	Saturated	28	35	82.7	12	4864.5	3741.3	38.5	0.83	0.656	1.17	0.75	2.000	1.765	2.7	0.0	0.00	0.00
39	127.9	Unsaturated	Saturated	28	35	82.7	12	4992.4	3806.8	38.1	0.82	0.657	1.17	0.75	2.000	1.747	2.7	0.0	0.00	0.00
40	127.9	Unsaturated	Saturated	28	35	82.7	12	5120.3	3872.3	37.7	0.81	0.658	1.17	0.74	2.000	1.730	2.6	0.0	0.00	0.00
41	127.9	Unsaturated	Saturated	19	40	92.0	34	5248.2	3937.8	23.6	0.81	0.659	1.17	0.86	0.261	0.263	Non-Liq.	0.0	0.00	0.00
42	127.9	Unsaturated	Saturated	19	40	92.0	34	5376.1	4003.3	23.4	0.80	0.659	1.17	0.86	0.256	0.257	Non-Liq.	0.0	0.00	0.00
43	130.0	Unsaturated	Saturated	19	40	92.0	34	5506.1	4070.9	23.1	0.80	0.659	1.17	0.86	0.251	0.252	Non-Liq.	0.0	0.00	0.00
44	130.0	Saturated	Saturated	19	40	92.0	34	5636.1	4138.5	23.0	0.79	0.658	1.17	0.85	0.249	0.249	Non-Liq.	0.0	0.00	0.00
45	130.0	Saturated	Saturated	19	40	92.0	34	5766.1	4206.1	22.8	0.79	0.658	1.17	0.85	0.247	0.247	Non-Liq.	0.0	0.00	0.00
46	130.0	Saturated	Saturated	17	45	65.6	34	5896.1	4273.7	20.4	0.78	0.657	1.17	0.87	0.211	0.214	Non-Liq.	0.0	0.00	0.00
47	130.0	Saturated	Saturated	17	45	65.6	34	6026.1	4341.3	20.3	0.77	0.657	1.17	0.86	0.209	0.212	Non-Liq.	0.0	0.00	0.00
48	137.0	Saturated	Saturated	34	48	65.6	14	6163.1	4415.9	46.6	0.77	0.655	1.17	0.70	2.000	1.638	2.5	0.0	0.00	0.00
49	137.0	Saturated	Saturated	34	48	65.6	14	6300.1	4490.5	46.4	0.76	0.654	1.17	0.70	2.000	1.629	2.5	0.0	0.00	0.00
50	137.0	Saturated	Saturated	34	48	42.3	14	6437.1	4565.1	46.2	0.76	0.652	1.17	0.69	2.000	1.620	2.5	0.0	0.00	0.00
												Total Liquefaction Settlement, S =						**0.00**		**inches**

Geotechnologies, Inc.

Project: Centennial GH Partners
File No.: 22662
Description: Liquefaction Analysis
Boring No: B1 (from File No. 22579)

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.9
Peak Ground Horizontal Acceleration, PGA (g):	0.94
Calculated Mag.Wtg.Factor:	1.171

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	29.5
Historically Highest Groundwater Level* (ft):	20.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):	4.3
SPT Sampler with room for Liner (Y/N):	Y

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1.3

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc}, (psf)	Effective Vert. Stress σ_{vc}', (psf)	Fines Corrected $(N_1)_{60\text{-}cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Mag. Scaling Factor (Sand) MSF	Overburden Corr. Factor K_σ	Cyclic Resist. Ratio $CRR_{M7.5,\sigma'v/C=1}$	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Lateral Disp. Index ΔLDI_i (feet)	Vertical Strain ε_v	Liquefaction Settlment ΔS_i (inches)
1	131.6	Unsaturated	Unsaturated	12	5	0.0	0	131.6	131.6	22.8	1.00	0.614	1.17	1.10	0.246	0.317	Non-Liq.	0.0	0.00	0.00
2	131.6	Unsaturated	Unsaturated	12	5	0.0	0	263.2	263.2	22.8	1.00	0.612	1.17	1.10	0.246	0.317	Non-Liq.	0.0	0.00	0.00
3	131.6	Unsaturated	Unsaturated	12	5	0.0	0	394.8	394.8	22.8	1.00	0.610	1.17	1.10	0.246	0.317	Non-Liq.	0.0	0.00	0.00
4	131.6	Unsaturated	Unsaturated	12	5	0.0	0	526.4	526.4	22.8	0.99	0.608	1.17	1.10	0.246	0.317	Non-Liq.	0.0	0.00	0.00
5	131.6	Unsaturated	Unsaturated	12	5	0.0	0	658.0	658.0	23.4	0.99	0.606	1.17	1.10	0.256	0.330	Non-Liq.	0.0	0.00	0.00
6	131.6	Unsaturated	Unsaturated	12	5	0.0	0	789.6	789.6	21.9	0.99	0.603	1.17	1.10	0.231	0.297	Non-Liq.	0.0	0.00	0.00
7	131.6	Unsaturated	Unsaturated	12	5	0.0	0	921.2	921.2	20.4	0.98	0.601	1.17	1.10	0.211	0.272	Non-Liq.	0.0	0.00	0.00
8	124.0	Unsaturated	Unsaturated	12	5	0.0	0	1045.2	1045.2	19.3	0.98	0.599	1.17	1.09	0.197	0.252	Non-Liq.	0.0	0.00	0.00
9	124.0	Unsaturated	Unsaturated	12	5	0.0	0	1169.2	1169.2	19.5	0.98	0.596	1.17	1.08	0.199	0.252	Non-Liq.	0.0	0.00	0.00
10	124.0	Unsaturated	Unsaturated	12	5	0.0	0	1293.2	1293.2	18.5	0.97	0.594	1.17	1.06	0.189	0.235	Non-Liq.	0.0	0.00	0.00
11	124.0	Unsaturated	Unsaturated	9	10	76.1	29	1417.2	1417.2	18.6	0.97	0.591	1.17	1.05	0.190	0.234	Non-Liq.	0.0	0.00	0.00
12	124.0	Unsaturated	Unsaturated	9	10	76.1	29	1541.2	1541.2	18.1	0.96	0.589	1.17	1.04	0.184	0.224	Non-Liq.	0.0	0.00	0.00
13	118.7	Unsaturated	Unsaturated	9	10	76.1	29	1659.9	1659.9	17.6	0.96	0.586	1.17	1.03	0.179	0.216	Non-Liq.	0.0	0.00	0.00
14	118.7	Unsaturated	Unsaturated	9	10	76.1	29	1778.6	1778.6	17.1	0.95	0.583	1.17	1.02	0.175	0.209	Non-Liq.	0.0	0.00	0.00
15	118.7	Unsaturated	Unsaturated	9	10	76.1	29	1897.3	1897.3	18.1	0.95	0.581	1.17	1.01	0.185	0.219	Non-Liq.	0.0	0.00	0.00
16	118.7	Unsaturated	Unsaturated	10	15	72.3	25	2016.0	2016.0	19.2	0.95	0.578	1.17	1.01	0.196	0.231	Non-Liq.	0.0	0.00	0.00
17	118.7	Unsaturated	Unsaturated	10	15	72.3	25	2134.7	2134.7	18.8	0.94	0.575	1.17	1.00	0.192	0.224	Non-Liq.	0.0	0.00	0.00
18	122.9	Unsaturated	Unsaturated	10	15	72.3	25	2257.6	2257.6	18.4	0.94	0.572	1.17	0.99	0.187	0.218	Non-Liq.	0.0	0.00	0.00
19	122.9	Unsaturated	Unsaturated	10	15	72.3	25	2380.5	2380.5	18.0	0.93	0.569	1.17	0.99	0.183	0.212	Non-Liq.	0.0	0.00	0.00
20	122.9	Unsaturated	Unsaturated	10	15	72.3	25	2503.4	2503.4	17.6	0.93	0.566	1.17	0.98	0.180	0.206	Non-Liq.	0.0	0.00	0.00
21	122.9	Unsaturated	Saturated	11	20	64.7	44	2626.3	2563.9	18.6	0.92	0.576	1.17	0.97	0.190	0.217	Non-Liq.	0.0	0.00	0.00
22	122.9	Unsaturated	Saturated	11	20	64.7	44	2749.2	2624.4	18.3	0.92	0.586	1.17	0.97	0.187	0.212	Non-Liq.	0.0	0.00	0.00
23	126.1	Unsaturated	Saturated	11	20	64.7	44	2875.3	2688.1	18.0	0.91	0.595	1.17	0.96	0.183	0.207	Non-Liq.	0.0	0.00	0.00
24	126.1	Unsaturated	Saturated	11	20	64.7	44	3001.4	2751.8	17.7	0.90	0.603	1.17	0.96	0.180	0.202	Non-Liq.	0.0	0.00	0.00
25	126.1	Unsaturated	Saturated	11	20	64.7	44	3127.5	2815.5	17.4	0.90	0.611	1.17	0.95	0.177	0.198	Non-Liq.	0.0	0.00	0.00
26	126.1	Unsaturated	Saturated	14	25	61.3	39	3253.6	2879.2	20.9	0.89	0.617	1.17	0.94	0.217	0.239	Non-Liq.	0.0	0.00	0.00
27	126.1	Unsaturated	Saturated	14	25	61.3	39	3379.7	2942.9	20.6	0.89	0.624	1.17	0.94	0.213	0.233	Non-Liq.	0.0	0.00	0.00
28	133.8	Unsaturated	Saturated	14	25	61.3	39	3513.5	3014.3	21.1	0.88	0.629	1.17	0.93	0.221	0.240	Non-Liq.	0.0	0.00	0.00
29	133.8	Unsaturated	Saturated	14	25	61.3	39	3647.3	3085.7	20.8	0.88	0.634	1.17	0.92	0.216	0.234	Non-Liq.	0.0	0.00	0.00
30	133.8	Saturated	Saturated	14	25	61.3	39	3781.1	3157.1	20.6	0.87	0.638	1.17	0.92	0.213	0.230	Non-Liq.	0.0	0.00	0.00
31	133.8	Saturated	Saturated	72	30	0.0	0	3914.9	3228.5	100.1	0.87	0.642	1.17	0.82	2.000	1.932	3.0	0.0	0.00	0.00
32	133.8	Saturated	Saturated	72	30	0.0	0	4048.7	3299.9	99.6	0.86	0.645	1.17	0.82	2.000	1.919	3.0	0.0	0.00	0.00
33	126.7	Saturated	Saturated	72	30	0.0	0	4175.4	3364.2	99.2	0.85	0.648	1.17	0.81	2.000	1.908	2.9	0.0	0.00	0.00
34	126.7	Saturated	Saturated	72	30	0.0	0	4302.1	3428.5	98.7	0.85	0.651	1.17	0.81	2.000	1.897	2.9	0.0	0.00	0.00
35	126.7	Saturated	Saturated	72	30	0.0	0	4428.8	3492.8	98.3	0.84	0.653	1.17	0.80	2.000	1.886	2.9	0.0	0.00	0.00
36	126.7	Saturated	Saturated	77	35	0.0	0	4555.5	3557.1	104.7	0.84	0.655	1.17	0.80	2.000	1.875	2.9	0.0	0.00	0.00
37	126.7	Saturated	Saturated	77	35	0.0	0	4682.2	3621.4	104.3	0.83	0.657	1.17	0.80	2.000	1.864	2.8	0.0	0.00	0.00
38	125.4	Saturated	Saturated	77	35	0.0	0	4807.6	3684.4	103.9	0.83	0.659	1.17	0.79	2.000	1.854	2.8	0.0	0.00	0.00
39	125.4	Saturated	Saturated	77	35	0.0	0	4933.0	3747.4	103.5	0.82	0.660	1.17	0.79	2.000	1.844	2.8	0.0	0.00	0.00
40	125.4	Saturated	Saturated	77	35	0.0	0	5058.4	3810.4	103.1	0.81	0.661	1.17	0.78	2.000	1.834	2.8	0.0	0.00	0.00
41	125.4	Saturated	Saturated	90	40	0.0	0	5183.8	3873.4	120.1	0.81	0.661	1.17	0.78	2.000	1.824	2.8	0.0	0.00	0.00
42	125.4	Saturated	Saturated	90	40	0.0	0	5309.2	3936.4	119.6	0.80	0.662	1.17	0.77	2.000	1.815	2.7	0.0	0.00	0.00
43	139.3	Saturated	Saturated	90	40	0.0	0	5448.5	4013.3	119.1	0.80	0.661	1.17	0.77	2.000	1.803	2.7	0.0	0.00	0.00
44	139.3	Saturated	Saturated	90	40	0.0	0	5587.8	4090.2	118.6	0.79	0.661	1.17	0.76	2.000	1.791	2.7	0.0	0.00	0.00
45	139.3	Saturated	Saturated	90	40	0.0	0	5727.1	4167.1	118.1	0.79	0.660	1.17	0.76	2.000	1.780	2.7	0.0	0.00	0.00
46	139.3	Saturated	Saturated	88	45	0.0	0	5866.4	4244.0	115.0	0.78	0.659	1.17	0.76	2.000	1.769	2.7	0.0	0.00	0.00
47	139.3	Saturated	Saturated	88	45	0.0	0	6005.7	4320.9	114.5	0.77	0.657	1.17	0.75	2.000	1.758	2.7	0.0	0.00	0.00
48	137.2	Saturated	Saturated	88	45	0.0	0	6142.9	4395.7	114.0	0.77	0.656	1.17	0.75	2.000	1.748	2.7	0.0	0.00	0.00
49	137.2	Saturated	Saturated	88	45	0.0	0	6280.1	4470.5	113.6	0.76	0.655	1.17	0.74	2.000	1.737	2.7	0.0	0.00	0.00
50	137.2	Saturated	Saturated	88	45	0.0	0	6417.3	4545.3	113.1	0.76	0.653	1.17	0.74	2.000	1.727	2.6	0.0	0.00	0.00
51	137.2	Saturated	Saturated	89	50	0.0	0	6554.5	4620.1	114.0	0.75	0.651	1.17	0.73	2.000	1.717	2.6	0.0	0.00	0.00
52	137.2	Saturated	Saturated	89	50	0.0	0	6691.7	4694.9	113.6	0.75	0.650	1.17	0.73	2.000	1.708	2.6	0.0	0.00	0.00
53	121.7	Saturated	Saturated	89	50	0.0	0	6813.4	4754.2	113.2	0.74	0.648	1.17	0.73	2.000	1.700	2.6	0.0	0.00	0.00
54	121.7	Saturated	Saturated	89	50	0.0	0	6935.1	4813.5	112.9	0.73	0.647	1.17	0.72	2.000	1.692	2.6	0.0	0.00	0.00
55	121.7	Saturated	Saturated	89	50	0.0	0	7056.8	4872.8	112.6	0.73	0.646	1.17	0.72	2.000	1.685	2.6	0.0	0.00	0.00
56	121.7	Saturated	Saturated	100	55	0.0	0	7178.5	4932.1	126.1	0.72	0.644	1.17	0.72	2.000	1.677	2.6	0.0	0.00	0.00
57	121.7	Saturated	Saturated	100	55	0.0	0	7300.2	4991.4	125.8	0.72	0.642	1.17	0.71	2.000	1.670	2.6	0.0	0.00	0.00
58	142.2	Saturated	Saturated	100	55	0.0	0	7442.4	5071.2	125.3	0.71	0.640	1.17	0.71	2.000	1.660	2.6	0.0	0.00	0.00
59	142.2	Saturated	Saturated	100	55	0.0	0	7584.6	5151.0	124.8	0.71	0.637	1.17	0.70	2.000	1.650	2.6	0.0	0.00	0.00
60	142.2	Saturated	Saturated	100	55	0.0	0	7726.8	5230.8	124.4	0.70	0.635	1.17	0.70	2.000	1.641	2.6	0.0	0.00	0.00
												Total Liquefaction Settlement, S =						**0.00**		**inches**

Geotechnologies, Inc.

Project: Centennial GH Partners
File No.: 22662
Description: Liquefaction Analysis
Boring No: B2 (from File No. 22579)

LIQUEFACTION EVALUATION (Idriss & Boulanger, EERI NO 12)

EARTHQUAKE INFORMATION:

Earthquake Magnitude (M):	6.9
Peak Ground Horizontal Acceleration, PGA (g):	0.94
Calculated Mag.Wtg.Factor:	1.171

GROUNDWATER INFORMATION:

Current Groundwater Level (ft):	29.5
Historically Highest Groundwater Level* (ft):	20.0
Unit Weight of Water (pcf):	62.4

* Based on California Geological Survey Seismic Hazard Evaluation Report

BOREHOLE AND SAMPLER INFORMATION:

Borehole Diameter (inches):	4.3
SPT Sampler with room for Liner (Y/N):	Y

LIQUEFACTION BOUNDARY:

Plastic Index Cut Off (PI):	18
Minimum Liquefaction FS:	1.3

Depth to Base Layer (feet)	Total Unit Weight (pcf)	Current Water Level (feet)	Historical Water Level (feet)	Field SPT Blowcount N	Depth of SPT Blowcount (feet)	Fines Content #200 Sieve (%)	Plastic Index (PI)	Vetical Stress σ_{vc} (psf)	Effective Vert. Stress σ_{vc}' (psf)	Fines Corrected $(N_1)_{60\text{-}cs}$	Stress Reduction Coeff, r_d	Cyclic Shear Ratio CSR	Mag. Scaling Factor (Sand) MSF	Overburden Corr. Factor K_σ	Cyclic Resist. Ratio $CRR_{M7.5,\sigma'v/C=1}$	Cyclic Resistance Ratio (CRR)	Factor of Safety CRR/CSR (F.S.)	Lateral Disp. Index ΔLDI_i (feet)	Vertical Strain ε_v	Liquefaction Settlment ΔS_i (inches)
1	122.9	Unsaturated	Unsaturated	5	5	0.0	0	122.9	122.9	8.8	1.00	0.614	1.17	1.10	0.110	0.141	Non-Liq.	0.0	0.00	0.00
2	122.9	Unsaturated	Unsaturated	5	5	0.0	0	245.8	245.8	8.8	1.00	0.612	1.17	1.10	0.110	0.141	Non-Liq.	0.0	0.00	0.00
3	122.9	Unsaturated	Unsaturated	5	5	0.0	0	368.7	368.7	8.8	1.00	0.610	1.17	1.10	0.110	0.141	Non-Liq.	0.0	0.00	0.00
4	122.9	Unsaturated	Unsaturated	5	5	0.0	0	491.6	491.6	8.8	0.99	0.608	1.17	1.10	0.110	0.141	Non-Liq.	0.0	0.00	0.00
5	122.9	Unsaturated	Unsaturated	5	5	0.0	0	614.5	614.5	9.4	0.99	0.606	1.17	1.10	0.114	0.146	Non-Liq.	0.0	0.00	0.00
6	122.9	Unsaturated	Unsaturated	5	5	44.6	0	737.4	737.4	15.0	0.99	0.603	1.17	1.10	0.156	0.201	Non-Liq.	0.0	0.00	0.00
7	122.9	Unsaturated	Unsaturated	5	5	44.6	0	860.3	860.3	14.6	0.98	0.601	1.17	1.10	0.153	0.197	Non-Liq.	0.0	0.00	0.00
8	124.1	Unsaturated	Unsaturated	5	5	44.6	0	984.4	984.4	14.0	0.98	0.599	1.17	1.08	0.148	0.188	Non-Liq.	0.0	0.00	0.00
9	124.1	Unsaturated	Unsaturated	5	5	44.6	0	1108.5	1108.5	14.0	0.98	0.596	1.17	1.07	0.148	0.185	Non-Liq.	0.0	0.00	0.00
10	124.1	Unsaturated	Unsaturated	5	5	44.6	0	1232.6	1232.6	13.5	0.97	0.594	1.17	1.06	0.144	0.178	Non-Liq.	0.0	0.00	0.00
11	124.1	Unsaturated	Unsaturated	12	10	64.8	16	1356.7	1356.7	23.7	0.97	0.591	1.17	1.07	0.262	0.328	Non-Liq.	0.0	0.00	0.00
12	124.1	Unsaturated	Unsaturated	12	10	64.8	16	1480.8	1480.8	22.9	0.96	0.589	1.17	1.05	0.248	0.306	Non-Liq.	0.0	0.00	0.00
13	129.3	Unsaturated	Unsaturated	12	10	64.8	16	1610.1	1610.1	22.2	0.96	0.586	1.17	1.04	0.236	0.287	Non-Liq.	0.0	0.00	0.00
14	129.3	Unsaturated	Unsaturated	12	10	64.8	16	1739.4	1739.4	21.5	0.95	0.583	1.17	1.03	0.226	0.272	Non-Liq.	0.0	0.00	0.00
15	129.3	Unsaturated	Unsaturated	12	10	64.8	16	1868.7	1868.7	23.0	0.95	0.581	1.17	1.02	0.249	0.296	Non-Liq.	0.0	0.00	0.00
16	129.3	Unsaturated	Unsaturated	15	15	57.9	18	1998.0	1998.0	27.2	0.95	0.578	1.17	1.01	0.352	0.417	Non-Liq.	0.0	0.00	0.00
17	129.3	Unsaturated	Unsaturated	15	15	57.9	18	2127.3	2127.3	26.5	0.94	0.575	1.17	1.00	0.331	0.387	Non-Liq.	0.0	0.00	0.00
18	132.2	Unsaturated	Unsaturated	15	15	57.9	18	2259.5	2259.5	25.9	0.94	0.572	1.17	0.99	0.312	0.361	Non-Liq.	0.0	0.00	0.00
19	132.2	Unsaturated	Unsaturated	15	15	57.9	18	2391.7	2391.7	25.3	0.93	0.569	1.17	0.98	0.297	0.340	Non-Liq.	0.0	0.00	0.00
20	132.2	Unsaturated	Unsaturated	15	15	57.9	18	2523.9	2523.9	24.7	0.93	0.566	1.17	0.97	0.284	0.323	Non-Liq.	0.0	0.00	0.00
21	132.2	Unsaturated	Saturated	26	20	43.3	0	2656.1	2593.7	42.4	0.92	0.576	1.17	0.93	2.000	2.000	3.5	0.0	0.00	0.00
22	132.2	Unsaturated	Saturated	26	20	43.3	0	2788.3	2663.5	41.6	0.92	0.586	1.17	0.92	2.000	2.000	3.4	0.0	0.00	0.00
23	130.9	Unsaturated	Saturated	26	20	43.3	0	2919.2	2732.0	40.9	0.91	0.594	1.17	0.90	2.000	2.000	3.4	0.0	0.00	0.00
24	130.9	Unsaturated	Saturated	26	20	43.3	0	3050.1	2800.5	40.2	0.90	0.602	1.17	0.89	2.000	2.000	3.3	0.0	0.00	0.00
25	130.9	Unsaturated	Saturated	26	20	43.3	0	3181.0	2869.0	39.6	0.90	0.609	1.17	0.88	2.000	2.000	3.3	0.0	0.00	0.00
26	130.9	Unsaturated	Saturated	47	25	0.0	0	3311.9	2937.5	64.4	0.89	0.616	1.17	0.87	2.000	2.000	3.2	0.0	0.00	0.00
27	130.9	Unsaturated	Saturated	47	25	0.0	0	3442.8	3006.0	63.8	0.89	0.622	1.17	0.86	2.000	2.000	3.2	0.0	0.00	0.00
28	128.5	Unsaturated	Saturated	47	25	0.0	0	3571.3	3072.1	66.5	0.88	0.627	1.17	0.84	2.000	1.979	3.2	0.0	0.00	0.00
29	128.5	Unsaturated	Saturated	47	25	0.0	0	3699.8	3138.2	65.9	0.88	0.632	1.17	0.83	2.000	1.954	3.1	0.0	0.00	0.00
30	128.5	Saturated	Saturated	47	25	0.0	0	3828.3	3204.3	65.4	0.87	0.637	1.17	0.83	2.000	1.936	3.0	0.0	0.00	0.00
31	128.5	Saturated	Saturated	81	30	0.0	0	3956.8	3270.4	112.3	0.87	0.640	1.17	0.82	2.000	1.924	3.0	0.0	0.00	0.00
32	128.5	Saturated	Saturated	81	30	0.0	0	4085.3	3336.5	111.8	0.86	0.644	1.17	0.82	2.000	1.913	3.0	0.0	0.00	0.00
33	136.1	Saturated	Saturated	81	30	0.0	0	4221.4	3410.2	111.2	0.85	0.647	1.17	0.81	2.000	1.900	2.9	0.0	0.00	0.00
34	136.1	Saturated	Saturated	81	30	0.0	0	4357.5	3483.9	110.7	0.85	0.649	1.17	0.81	2.000	1.887	2.9	0.0	0.00	0.00
35	136.1	Saturated	Saturated	81	30	0.0	0	4493.6	3557.6	110.2	0.84	0.651	1.17	0.80	2.000	1.875	2.9	0.0	0.00	0.00
36	136.1	Saturated	Saturated	100	35	0.0	0	4629.7	3631.3	135.4	0.84	0.653	1.17	0.80	2.000	1.863	2.9	0.0	0.00	0.00
37	136.1	Saturated	Saturated	100	35	0.0	0	4765.8	3705.0	134.8	0.83	0.654	1.17	0.79	2.000	1.851	2.8	0.0	0.00	0.00
38	138.1	Saturated	Saturated	100	35	0.0	0	4903.9	3780.7	134.1	0.83	0.655	1.17	0.78	2.000	1.839	2.8	0.0	0.00	0.00
39	138.1	Saturated	Saturated	100	35	0.0	0	5042.0	3856.4	133.5	0.82	0.655	1.17	0.78	2.000	1.827	2.8	0.0	0.00	0.00
40	138.1	Saturated	Saturated	100	35	0.0	0	5180.1	3932.1	132.9	0.81	0.656	1.17	0.77	2.000	1.815	2.8	0.0	0.00	0.00
41	138.1	Saturated	Saturated	100	40	0.0	0	5318.2	4007.8	132.4	0.81	0.656	1.17	0.77	2.000	1.804	2.8	0.0	0.00	0.00
42	138.1	Saturated	Saturated	100	40	0.0	0	5456.3	4083.5	131.8	0.80	0.655	1.17	0.77	2.000	1.792	2.7	0.0	0.00	0.00
43	139.3	Saturated	Saturated	100	40	0.0	0	5595.6	4160.4	131.2	0.80	0.655	1.17	0.76	2.000	1.781	2.7	0.0	0.00	0.00
44	139.3	Saturated	Saturated	100	40	0.0	0	5734.9	4237.3	130.7	0.79	0.654	1.17	0.76	2.000	1.770	2.7	0.0	0.00	0.00
45	139.3	Saturated	Saturated	100	40	0.0	0	5874.2	4314.2	130.1	0.79	0.654	1.17	0.75	2.000	1.759	2.7	0.0	0.00	0.00
46	139.3	Saturated	Saturated	100	45	0.0	0	6013.5	4391.1	129.6	0.78	0.653	1.17	0.75	2.000	1.748	2.7	0.0	0.00	0.00
47	139.3	Saturated	Saturated	100	45	0.0	0	6152.8	4468.0	129.1	0.77	0.651	1.17	0.74	2.000	1.738	2.7	0.0	0.00	0.00
48	140.2	Saturated	Saturated	100	45	0.0	0	6293.0	4545.8	128.6	0.77	0.650	1.17	0.74	2.000	1.727	2.7	0.0	0.00	0.00
49	140.2	Saturated	Saturated	100	45	0.0	0	6433.2	4623.6	128.1	0.76	0.648	1.17	0.73	2.000	1.717	2.6	0.0	0.00	0.00
50	140.2	Saturated	Saturated	100	45	0.0	0	6573.4	4701.4	127.6	0.76	0.647	1.17	0.73	2.000	1.707	2.6	0.0	0.00	0.00
51	140.2	Saturated	Saturated	100	50	0.0	0	6713.6	4779.2	127.1	0.75	0.645	1.17	0.72	2.000	1.697	2.6	0.0	0.00	0.00
52	140.2	Saturated	Saturated	100	50	0.0	0	6853.8	4857.0	126.6	0.75	0.643	1.17	0.72	2.000	1.687	2.6	0.0	0.00	0.00
53	138.8	Saturated	Saturated	100	50	0.0	0	6992.6	4933.4	126.1	0.74	0.641	1.17	0.72	2.000	1.677	2.6	0.0	0.00	0.00
54	138.8	Saturated	Saturated	100	50	0.0	0	7131.4	5009.8	125.7	0.73	0.639	1.17	0.71	2.000	1.668	2.6	0.0	0.00	0.00
55	138.8	Saturated	Saturated	100	50	0.0	0	7270.2	5086.2	125.2	0.73	0.637	1.17	0.71	2.000	1.658	2.6	0.0	0.00	0.00
56	138.8	Saturated	Saturated	100	55	0.0	0	7409.0	5162.6	124.8	0.72	0.635	1.17	0.70	2.000	1.649	2.6	0.0	0.00	0.00
57	138.8	Saturated	Saturated	100	55	0.0	0	7547.8	5239.0	124.4	0.72	0.633	1.17	0.70	2.000	1.640	2.6	0.0	0.00	0.00
58	138.6	Saturated	Saturated	100	55	0.0	0	7686.4	5315.2	123.9	0.71	0.630	1.17	0.70	2.000	1.631	2.6	0.0	0.00	0.00
59	138.6	Saturated	Saturated	100	55	0.0	0	7825.0	5391.4	123.5	0.71	0.628	1.17	0.69	2.000	1.622	2.6	0.0	0.00	0.00
60	138.6	Saturated	Saturated	100	55	0.0	0	7963.6	5467.6	123.1	0.70	0.626	1.17	0.69	2.000	1.613	2.6	0.0	0.00	0.00
																Total Liquefaction Settlement, S =		**0.00**		**inches**

Geotechnologies, Inc.

Project: Centennial GH Partners
File No.: 22662
Description: Slot Cut

Slot Cut Calculation

Input:

Height of Slots	(H)	6.0	feet
Unit Weight of Soils	(γ)	120.0	pcf
Friction Angle of Soils	(ϕ)	16.0	degrees
Cohesion of Soils	(c)	365.0	psf
Factor of Safety	(FS)	1.25	

Factor of Safety = Resistance Force/Driving Force

Coefficient of Lateral Earth Pressure At-Rest	K_o	0.5	

Surcharge Pressure:

Line Load	(q_L)	500.0	plf
Distance Away from Edge of Excavation	(X)	0.0	feet

Design Equations

$b = H/(\tan \alpha)$

$A = 0.5*H*b$

$W = 0.5*H*b*\gamma$ *(per lineal foot of slot width)*

$F_1 = d*W*(\sin \alpha)*(\cos \alpha)$

$F_2 = d*L$

$R_1 = d*[W*(\cos^2 \alpha)*(\tan \phi)+(c*b)]$

$R_2 = 2*\Delta F$

$\Delta F = A*[1/3*\gamma*H*K_o*(\tan \phi)+c]$

FS = Resistance Force/Driving Force

$FS = (R_1+R_2)/(F_1+F_2)$

Failure Angle (α) degrees	Base Width of Failure Wedge (b) feet	Area of Failure Wedge (A) feet2	Weight of Failure Wedge (W) lbs/lineal foot	**Driving Force** Wedge + Surcharge per lineal foot of Slot Wdith	**Resisting Force** Failure Wedge per lineal foot of Slot Width	**Resisting Force** Side Resistance Force (ΔF) lbs	Allowable Width of Slots* (d) feet
60	3.5	10	1247.1	756.5	1389.6	4150.8	8.0
61	3.3	10	1197.3	719.7	1328.3	3985.1	8.0
62	3.2	10	1148.5	683.3	1268.6	3822.7	8.0
63	3.1	9	1100.6	647.4	1210.5	3663.2	8.0
64	2.9	9	1053.5	612.1	1153.7	3506.5	8.0
65	2.8	8	1007.2	577.3	1098.4	3352.5	8.0
66	2.7	8	961.7	543.1	1044.4	3200.9	8.0
67	2.5	8	916.9	509.6	991.6	3051.7	8.0
68	2.4	7	872.7	476.8	940.1	2904.7	8.0
69	2.3	7	829.1	444.7	889.6	2759.7	8.0
70	2.2	7	786.2	413.4	840.2	2616.7	8.0
71	2.1	6	743.7	382.9	791.9	2475.5	8.0
72	1.9	6	701.8	353.2	744.5	2336.0	8.0
73	1.8	6	660.4	324.4	698.0	2198.0	8.0
74	1.7	5	619.4	296.6	652.4	2061.5	8.0
75	1.6	5	578.8	269.7	607.5	1926.4	8.0
76	1.5	4	538.5	243.8	563.5	1792.5	8.0
77	1.4	4	498.7	218.9	520.1	1659.8	8.0
78	1.3	4	459.1	195.1	477.4	1528.1	8.0
79	1.2	3	419.9	172.3	435.3	1397.5	8.0
80	1.1	3	380.9	150.6	393.8	1267.7	8.0
81	1.0	3	342.1	130.1	352.8	1138.7	8.0
82	0.8	3	303.6	110.7	312.2	1010.4	8.0
83	0.7	2	265.2	92.6	272.2	882.7	8.0
84	0.6	2	227.0	75.6	232.5	755.6	8.0
85	0.5	2	189.0	59.8	193.1	629.0	8.0

* Width of Slots to achieve a minimum o f 1.5 Factor of Saferty, with a Maximum Allowable Slot Width of 8-feet.

Critical Slot Width with Factor of Safety equal or exceeding 1.5:

d_{allow} 8.0 feet

The proposed excavation may be made using the A-B-C Slot-Cutting Method with a Maximum Allowable Slot Width of 8 Feet, and up to 6 Feet in Height, with a Factor of Safety Equal or Exceeding 1.25.

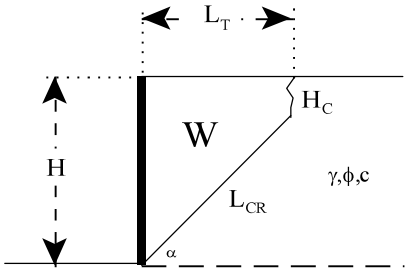
Geotechnologies, Inc.

Project: Centennial GH Partners
File No.: 22662
Description: Temporary Shoring (Daylighted Bedrock)

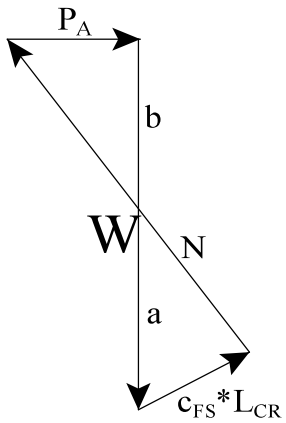
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	10.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	14.0	degrees
Cohesion of Retained Soils	(c)	220.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	11.3	degrees
	(c_{FS})	176.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	3.9	50	6058.8	9.5	3404.5	2654.2	1454.3
41	3.8	49	5882.2	9.4	3266.7	2615.5	1493.0
42	3.8	48	5707.1	9.3	3136.3	2570.8	1527.6
43	3.7	46	5533.9	9.2	3013.0	2520.9	1558.2
44	3.7	45	5363.0	9.1	2896.3	2466.6	1584.8
45	3.7	43	5194.4	9.0	2785.9	2408.5	1607.5
46	3.6	42	5028.4	8.8	2681.3	2347.1	1626.4
47	3.6	41	4865.0	8.7	2582.1	2282.9	1641.6
48	3.6	39	4704.1	8.6	2488.0	2216.2	1653.1
49	3.6	38	4545.9	8.5	2398.6	2147.4	1660.9
50	3.6	37	4390.3	8.4	2313.5	2076.8	1665.0
51	3.6	35	4237.2	8.3	2232.5	2004.7	1665.5
52	3.6	34	4086.5	8.1	2155.1	1931.4	1662.4
53	3.6	33	3938.2	8.0	2081.2	1856.9	1655.6
54	3.6	32	3792.1	7.9	2010.5	1781.6	1645.2
55	3.6	30	3648.2	7.8	1942.5	1705.6	1631.1
56	3.7	29	3506.3	7.7	1877.2	1629.0	1613.2
57	3.7	28	3366.3	7.5	1814.2	1552.0	1591.5
58	3.7	27	3228.1	7.4	1753.3	1474.7	1566.0
59	3.8	26	3091.5	7.3	1694.3	1397.2	1536.6
60	3.8	25	2956.5	7.1	1636.9	1319.6	1503.1
61	3.9	24	2822.9	7.0	1580.8	1242.1	1465.6
62	4.0	22	2690.5	6.8	1525.9	1164.6	1423.9
63	4.0	21	2559.2	6.7	1471.8	1087.4	1377.9
64	4.1	20	2428.8	6.5	1418.2	1010.5	1327.5
65	4.2	19	2299.2	6.4	1365.0	934.1	1272.6



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 1665.5 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 33.3 pcf

Design Shoring for an Equivalent Fluid Pressure: 35 pcf

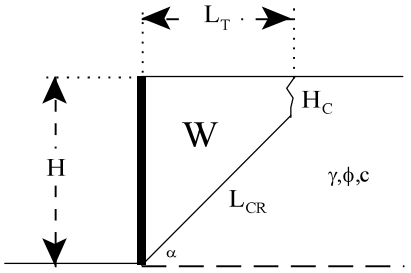
Geotechnologies, Inc.

Project: Centennial GH Partners
File No.: 22662
Description: Temporary Shoring (Daylighted Bedrock)

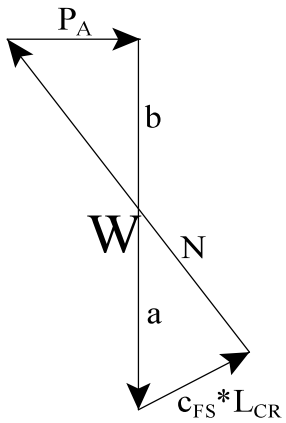
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	15.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	14.0	degrees
Cohesion of Retained Soils	(c)	220.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	11.3	degrees
	(c_{FS})	176.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	3.9	125	14996.9	17.3	6198.6	8798.4	4820.9
41	3.8	121	14510.0	17.0	5920.1	8589.9	4903.5
42	3.8	117	14036.7	16.8	5661.1	8375.6	4977.0
43	3.7	113	13576.7	16.5	5419.8	8156.9	5041.7
44	3.7	109	13129.4	16.3	5194.7	7934.8	5097.9
45	3.7	106	12694.4	16.0	4984.4	7710.0	5145.8
46	3.6	102	12271.1	15.8	4787.6	7483.4	5185.6
47	3.6	99	11858.8	15.6	4603.3	7255.6	5217.4
48	3.6	95	11457.2	15.3	4430.2	7026.9	5241.5
49	3.6	92	11065.6	15.1	4267.6	6798.0	5257.8
50	3.6	89	10683.6	14.9	4114.5	6569.0	5266.5
51	3.6	86	10310.6	14.7	3970.2	6340.4	5267.6
52	3.6	83	9946.2	14.5	3833.9	6112.2	5261.0
53	3.6	80	9589.8	14.3	3705.0	5884.8	5246.8
54	3.6	77	9241.2	14.1	3582.8	5658.3	5225.0
55	3.6	74	8899.7	13.9	3466.9	5432.8	5195.3
56	3.7	71	8565.1	13.7	3356.6	5208.5	5157.7
57	3.7	69	8236.8	13.5	3251.5	4985.3	5112.1
58	3.7	66	7914.6	13.3	3151.2	4763.4	5058.3
59	3.8	63	7598.0	13.1	3055.1	4542.9	4996.0
60	3.8	61	7286.6	12.9	2962.9	4323.7	4925.0
61	3.9	58	6980.2	12.7	2874.2	4106.0	4845.0
62	4.0	56	6678.3	12.5	2788.6	3889.7	4755.7
63	4.0	53	6380.6	12.3	2705.6	3675.0	4656.6
64	4.1	51	6086.8	12.1	2625.0	3461.8	4547.5
65	4.2	48	5796.5	11.9	2546.3	3250.2	4427.8



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 5267.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 46.8 pcf

Design Shoring for an Equivalent Fluid Pressure: 47 pcf

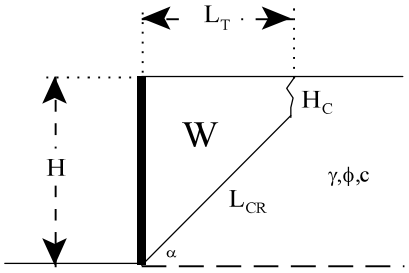
Geotechnologies, Inc.

Project: Centennial GH Partners
File No.: 22662
Description: Temporary Shoring (Daylighted Bedrock)

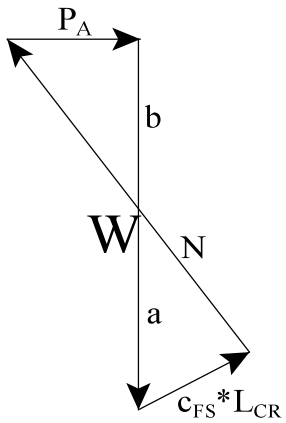
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	20.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	14.0	degrees
Cohesion of Retained Soils	(c)	220.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	11.3	degrees
	(c_{FS})	176.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	3.9	229	27510.4	25.0	8992.6	18517.8	10146.5
41	3.8	222	26588.8	24.6	8573.5	18015.4	10284.0
42	3.8	214	25698.1	24.2	8185.8	17512.3	10406.2
43	3.7	207	24836.6	23.8	7826.6	17010.0	10513.7
44	3.7	200	24002.5	23.5	7493.1	16509.5	10606.9
45	3.7	193	23194.4	23.1	7182.9	16011.5	10686.3
46	3.6	187	22410.8	22.7	6894.0	15516.8	10752.2
47	3.6	180	21650.2	22.4	6624.4	15025.8	10805.0
48	3.6	174	20911.4	22.1	6372.5	14538.9	10844.7
49	3.6	168	20193.1	21.8	6136.7	14056.4	10871.8
50	3.6	162	19494.1	21.4	5915.6	13578.5	10886.1
51	3.6	157	18813.3	21.1	5707.9	13105.4	10887.9
52	3.6	151	18149.6	20.8	5512.7	12637.0	10877.1
53	3.6	146	17502.1	20.5	5328.8	12173.4	10853.6
54	3.6	141	16869.9	20.3	5155.2	11714.6	10817.4
55	3.6	135	16251.9	20.0	4991.2	11260.6	10768.3
56	3.7	130	15647.4	19.7	4836.0	10811.4	10706.1
57	3.7	125	15055.6	19.4	4688.8	10366.8	10630.5
58	3.7	121	14475.7	19.2	4549.0	9926.7	10541.2
59	3.8	116	13907.0	18.9	4415.9	9491.1	10437.8
60	3.8	111	13348.8	18.7	4289.0	9059.8	10319.8
61	3.9	107	12800.4	18.4	4167.6	8632.8	10186.6
62	4.0	102	12261.2	18.2	4051.3	8210.0	10037.7
63	4.0	98	11730.6	17.9	3939.5	7791.2	9872.3
64	4.1	93	11208.0	17.7	3831.7	7376.3	9689.6
65	4.2	89	10692.7	17.4	3727.5	6965.2	9488.8



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 10887.9 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 54.4 pcf

Design Shoring for an Equivalent Fluid Pressure: **55 pcf**

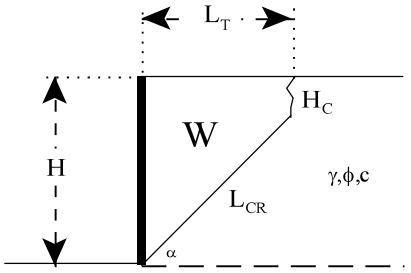
Geotechnologies, Inc.

Project: Centennial GH Partners
File No.: 22662
Description: Temporary Shoring (Fill)

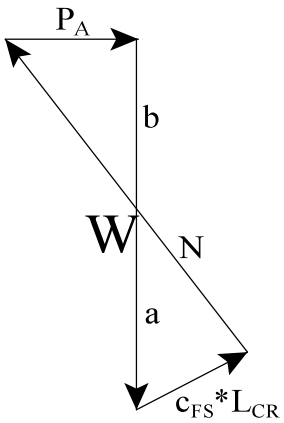
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	10.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	15.0	degrees
Cohesion of Retained Soils	(c)	356.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	12.1	degrees
	(c_{FS})	284.8	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	6.5	35	4153.8	5.5	3264.7	889.2	470.8
41	6.4	34	4108.4	5.5	3195.0	913.4	504.3
42	6.3	34	4048.9	5.6	3118.9	930.0	534.8
43	6.2	33	3978.0	5.6	3038.4	939.6	562.4
44	6.1	32	3897.7	5.6	2955.0	942.8	586.9
45	6.0	32	3809.8	5.6	2869.8	940.1	608.2
46	6.0	31	3715.6	5.6	2783.6	932.0	626.3
47	5.9	30	3616.3	5.5	2697.2	919.1	641.2
48	5.9	29	3512.7	5.5	2610.9	901.8	652.8
49	5.9	28	3405.6	5.4	2525.0	880.6	661.2
50	5.9	27	3295.7	5.4	2439.9	855.8	666.3
51	5.9	27	3183.4	5.3	2355.5	827.9	668.1
52	5.9	26	3069.1	5.2	2271.9	797.2	666.6
53	5.9	25	2953.2	5.1	2189.2	764.0	661.8
54	5.9	24	2835.8	5.1	2107.2	728.6	653.8
55	5.9	23	2717.2	5.0	2025.8	691.4	642.5
56	6.0	22	2597.5	4.8	1945.0	652.5	627.9
57	6.0	21	2476.8	4.7	1864.6	612.2	610.1
58	6.1	20	2355.1	4.6	1784.2	570.9	589.1
59	6.2	19	2232.4	4.5	1703.8	528.6	564.9
60	6.3	18	2108.7	4.3	1622.9	485.8	537.7
61	6.4	17	1984.0	4.2	1541.4	442.6	507.4
62	6.5	15	1858.1	4.0	1458.7	399.3	474.2
63	6.6	14	1730.9	3.8	1374.7	356.2	438.3
64	6.7	13	1602.2	3.6	1288.7	313.5	399.9
65	6.9	12	1471.8	3.4	1200.2	271.6	359.1



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 668.1 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 13.4 pcf

Design Shoring for an Equivalent Fluid Pressure: **28 pcf**

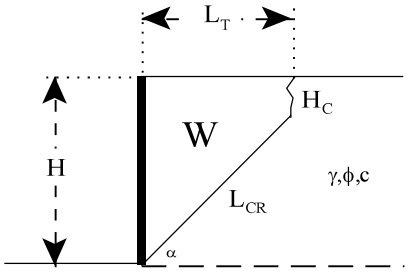
Geotechnologies, Inc.

Project: Centennial GH Partners
File No.: 22662
Description: Temporary Shoring (Fill)

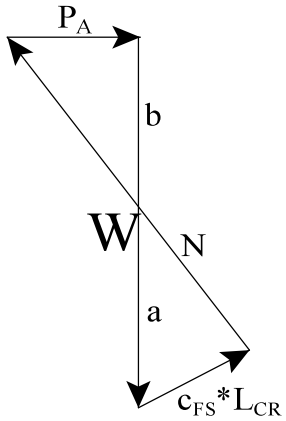
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	15.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	15.0	degrees
Cohesion of Retained Soils	(c)	356.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	12.1	degrees
	(c_{FS})	284.8	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	6.5	109	13092.0	13.3	7893.7	5198.3	2752.5
41	6.4	106	12736.1	13.2	7586.3	5149.9	2843.0
42	6.3	103	12378.5	13.1	7293.1	5085.4	2924.4
43	6.2	100	12020.8	12.9	7013.8	5007.0	2996.8
44	6.1	97	11664.2	12.8	6747.9	4916.3	3060.3
45	6.0	94	11309.8	12.7	6494.8	4815.0	3115.1
46	6.0	91	10958.3	12.5	6254.0	4704.3	3161.3
47	5.9	88	10610.1	12.4	6024.6	4585.5	3199.1
48	5.9	86	10265.7	12.2	5806.1	4459.7	3228.4
49	5.9	83	9925.3	12.1	5597.6	4327.6	3249.4
50	5.9	80	9589.0	11.9	5398.7	4190.3	3262.2
51	5.9	77	9256.8	11.7	5208.5	4048.3	3266.7
52	5.9	74	8928.8	11.6	5026.4	3902.3	3263.0
53	5.9	72	8604.8	11.4	4851.9	3753.0	3251.1
54	5.9	69	8284.9	11.2	4684.2	3600.7	3230.8
55	5.9	66	7968.8	11.1	4522.8	3446.0	3202.3
56	6.0	64	7656.3	10.9	4367.1	3289.2	3165.4
57	6.0	61	7347.3	10.7	4216.5	3130.8	3120.0
58	6.1	59	7041.6	10.5	4070.5	2971.1	3066.1
59	6.2	56	6738.9	10.3	3928.4	2810.4	3003.4
60	6.3	54	6438.8	10.1	3789.7	2649.1	2931.9
61	6.4	51	6141.3	9.9	3653.9	2487.4	2851.5
62	6.5	49	5845.9	9.7	3520.3	2325.6	2761.9
63	6.6	46	5552.3	9.4	3388.3	2164.0	2662.9
64	6.7	44	5260.2	9.2	3257.2	2002.9	2554.6
65	6.9	41	4969.1	9.0	3126.4	1842.7	2436.6



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 3266.7 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 29.0 pcf

Design Shoring for an Equivalent Fluid Pressure: 30 pcf

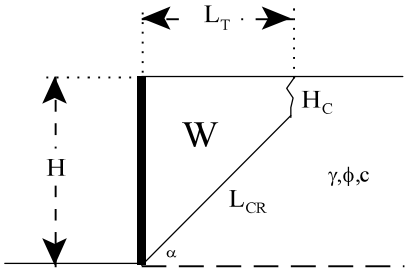
Geotechnologies, Inc.

Project: Centennial GH Partners
File No.: 22662
Description: Temporary Shoring (Fill)

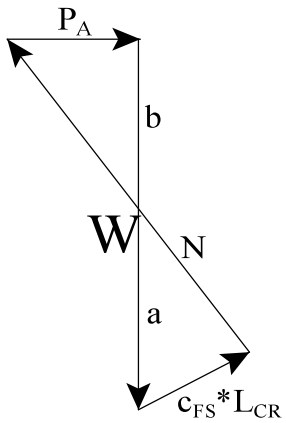
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	20.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	15.0	degrees
Cohesion of Retained Soils	(c)	356.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	12.1	degrees
	(c_{FS})	284.8	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	6.5	213	25605.4	21.0	12522.7	13082.7	6927.3
41	6.4	207	24815.0	20.8	11977.6	12837.4	7087.0
42	6.3	200	24039.9	20.5	11467.3	12572.7	7230.0
43	6.2	194	23280.6	20.3	10989.2	12291.5	7356.6
44	6.1	188	22537.3	20.0	10540.8	11996.5	7467.5
45	6.0	182	21809.8	19.7	10119.9	11689.9	7562.9
46	6.0	176	21098.0	19.5	9724.3	11373.7	7643.2
47	5.9	170	20401.5	19.2	9352.0	11049.5	7708.6
48	5.9	164	19720.0	19.0	9001.2	10718.7	7759.4
49	5.9	159	19052.8	18.7	8670.3	10382.5	7795.8
50	5.9	153	18399.5	18.4	8357.5	10042.0	7817.8
51	5.9	148	17759.5	18.2	8061.5	9698.0	7825.6
52	5.9	143	17132.3	17.9	7781.0	9351.3	7819.2
53	5.9	138	16517.2	17.7	7514.6	9002.6	7798.6
54	5.9	133	15913.6	17.4	7261.2	8652.4	7763.6
55	5.9	128	15320.9	17.2	7019.8	8301.2	7714.3
56	6.0	123	14738.7	16.9	6789.2	7949.5	7650.3
57	6.0	118	14166.1	16.6	6568.5	7597.7	7571.5
58	6.1	113	13602.7	16.4	6356.7	7246.0	7477.6
59	6.2	109	13047.9	16.1	6153.1	6894.8	7368.3
60	6.3	104	12501.0	15.9	5956.6	6544.4	7243.2
61	6.4	100	11961.5	15.6	5766.5	6195.1	7101.9
62	6.5	95	11428.8	15.3	5581.9	5847.0	6943.8
63	6.6	91	10902.3	15.1	5401.9	5500.4	6768.5
64	6.7	87	10381.4	14.8	5225.8	5155.6	6575.4
65	6.9	82	9865.3	14.5	5052.6	4812.8	6363.9



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 7825.6 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 39.1 pcf

Design Shoring for an Equivalent Fluid Pressure: **40 pcf**

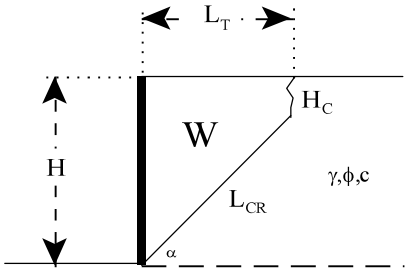
Geotechnologies, Inc.

Project: Centennial GH Partners
File No.: 22662
Description: Temporary Shoring (Alluvium)

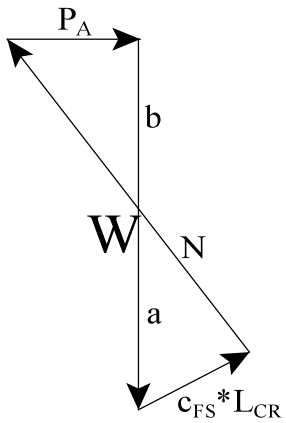
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	15.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	23.0	degrees
Cohesion of Retained Soils	(c)	280.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	18.8	degrees
	(c_{FS})	224.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	6.4	110	13189.0	13.4	7861.1	5327.9	2071.2
41	6.2	107	12888.0	13.4	7527.0	5361.0	2192.5
42	6.0	105	12572.8	13.4	7207.4	5365.4	2304.4
43	5.9	102	12248.0	13.4	6903.2	5344.8	2406.9
44	5.8	99	11917.1	13.3	6614.3	5302.7	2500.2
45	5.7	97	11582.6	13.2	6340.6	5242.0	2584.3
46	5.6	94	11246.7	13.1	6081.5	5165.2	2659.5
47	5.5	91	10910.7	13.0	5836.4	5074.3	2725.8
48	5.4	88	10575.9	12.9	5604.5	4971.4	2783.4
49	5.3	85	10243.1	12.8	5385.0	4858.0	2832.4
50	5.3	83	9912.8	12.7	5177.3	4735.5	2872.9
51	5.3	80	9585.6	12.5	4980.4	4605.2	2904.9
52	5.2	77	9261.7	12.4	4793.6	4468.1	2928.7
53	5.2	75	8941.3	12.2	4616.2	4325.1	2944.2
54	5.2	72	8624.5	12.1	4447.4	4177.1	2951.4
55	5.2	69	8311.4	11.9	4286.6	4024.8	2950.4
56	5.2	67	8001.9	11.8	4133.2	3868.7	2941.2
57	5.2	64	7696.0	11.6	3986.5	3709.5	2923.7
58	5.3	62	7393.5	11.5	3845.8	3547.7	2897.9
59	5.3	59	7094.3	11.3	3710.7	3383.6	2863.8
60	5.4	57	6798.2	11.1	3580.5	3217.7	2821.2
61	5.4	54	6505.1	10.9	3454.7	3050.4	2770.2
62	5.5	52	6214.6	10.8	3332.7	2882.0	2710.5
63	5.6	49	5926.7	10.6	3213.9	2712.7	2642.0
64	5.7	47	5640.9	10.4	3097.9	2543.0	2564.7
65	5.8	45	5357.1	10.2	2984.0	2373.0	2478.3



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 2951.4 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 26.2 pcf

Design Shoring for an Equivalent Fluid Pressure: 28 pcf

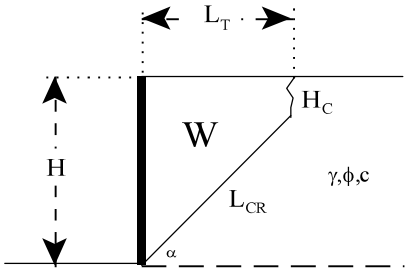
Geotechnologies, Inc.

Project: Centennial GH Partners
File No.: 22662
Description: Temporary Shoring (Alluvium)

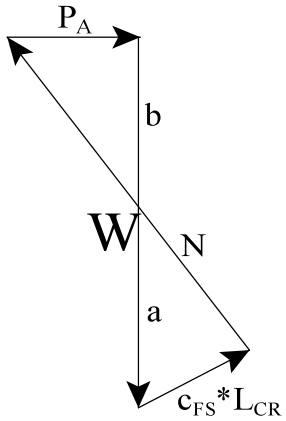
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	20.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	23.0	degrees
Cohesion of Retained Soils	(c)	280.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	18.8	degrees
	(c_{FS})	224.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	6.4	214	25702.4	21.2	12414.6	13287.8	5165.6
41	6.2	208	24966.9	21.1	11797.3	13169.6	5386.1
42	6.0	202	24234.2	20.9	11223.5	13010.7	5588.1
43	5.9	196	23507.9	20.7	10690.2	12817.7	5772.2
44	5.8	190	22790.1	20.5	10194.2	12596.0	5938.9
45	5.7	184	22082.6	20.3	9732.4	12350.2	6088.7
46	5.6	178	21386.4	20.1	9302.1	12084.3	6222.1
47	5.5	173	20702.1	19.9	8900.7	11801.5	6339.4
48	5.4	167	20030.1	19.6	8525.7	11504.5	6441.1
49	5.3	161	19370.6	19.4	8174.9	11195.6	6527.4
50	5.3	156	18723.3	19.2	7846.4	10877.0	6598.6
51	5.3	151	18088.3	19.0	7538.2	10550.1	6655.0
52	5.2	146	17465.2	18.7	7248.6	10216.6	6696.7
53	5.2	140	16853.6	18.5	6976.0	9877.6	6723.8
54	5.2	135	16253.2	18.3	6719.1	9534.1	6736.4
55	5.2	131	15663.6	18.1	6476.5	9187.1	6734.7
56	5.2	126	15084.3	17.8	6246.9	8837.4	6718.5
57	5.2	121	14514.8	17.6	6029.3	8485.5	6687.8
58	5.3	116	13954.6	17.4	5822.6	8132.0	6642.6
59	5.3	112	13403.3	17.1	5625.8	7777.5	6582.7
60	5.4	107	12860.4	16.9	5438.0	7422.4	6507.8
61	5.4	103	12325.3	16.7	5258.3	7067.0	6417.8
62	5.5	98	11797.6	16.4	5085.9	6711.7	6312.3
63	5.6	94	11276.7	16.2	4919.9	6356.8	6191.1
64	5.7	90	10762.1	15.9	4759.6	6002.5	6053.8
65	5.8	85	10253.3	15.7	4604.1	5649.2	5899.9



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 6736.4 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 33.7 pcf

Design Shoring for an Equivalent Fluid Pressure: 34 pcf

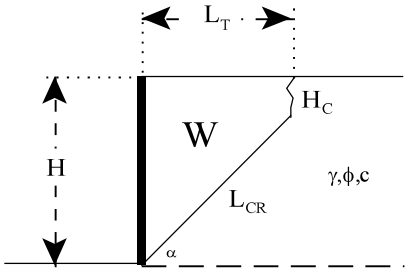
Geotechnologies, Inc.

Project: Centennial GH Partners
File No.: 22662
Description: Temporary Shoring (Bedrock)

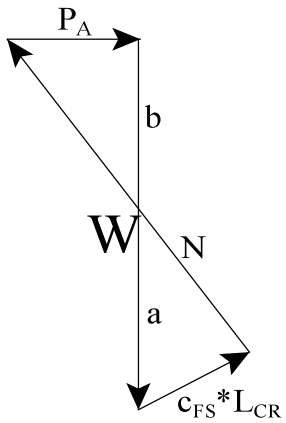
Shoring Design with Level Backfill
(Vector Analysis)

Input:

Shoring Height	(H)	20.00	feet
Unit Weight of Retained Soils	(γ)	120.0	pcf
Friction Angle of Retained Soils	(ϕ)	31.0	degrees
Cohesion of Retained Soils	(c)	480.0	psf
Factor of Safety	(FS)	1.25	
Factored Parameters:	(ϕ_{FS})	25.7	degrees
	(c_{FS})	384.0	psf



Failure Angle (α) degrees	Height of Tension Crack (H_C) feet	Area of Wedge (A) $feet^2$	Weight of Wedge (W) lbs/lineal foot	Length of Failure Plane (L_{CR}) feet	a lbs/lineal foot	b lbs/lineal foot	Active Pressure (P_A) lbs/lineal foot
40	15.2	100	12049.8	7.4	10412.3	1637.5	418.2
41	14.5	110	13182.1	8.4	11061.7	2120.4	581.1
42	13.8	116	13954.5	9.3	11397.4	2557.1	749.0
43	13.2	120	14456.0	9.9	11516.7	2939.3	917.0
44	12.8	123	14751.0	10.4	11486.0	3265.0	1081.5
45	12.3	124	14887.1	10.9	11351.9	3535.2	1239.9
46	12.0	124	14899.7	11.2	11146.7	3752.9	1390.3
47	11.6	123	14815.5	11.4	10893.7	3921.8	1531.2
48	11.3	122	14655.2	11.6	10609.1	4046.0	1661.6
49	11.1	120	14434.4	11.8	10304.8	4129.6	1780.8
50	10.9	118	14165.6	11.9	9989.1	4176.5	1888.1
51	10.7	115	13858.5	12.0	9668.0	4190.5	1983.2
52	10.6	113	13520.6	12.0	9345.7	4174.9	2065.8
53	10.4	110	13158.0	12.0	9025.2	4132.8	2135.6
54	10.3	106	12775.6	11.9	8708.4	4067.2	2192.4
55	10.3	103	12377.1	11.9	8396.5	3980.6	2236.3
56	10.2	100	11965.7	11.8	8090.4	3875.3	2267.0
57	10.2	96	11543.8	11.7	7790.2	3753.6	2284.6
58	10.2	93	11113.2	11.6	7496.1	3617.1	2289.0
59	10.2	89	10675.5	11.4	7207.7	3467.9	2280.3
60	10.2	85	10231.9	11.3	6924.6	3307.4	2258.4
61	10.3	82	9783.3	11.1	6646.2	3137.1	2223.4
62	10.4	78	9330.1	10.9	6371.8	2958.4	2175.3
63	10.5	74	8872.9	10.7	6100.4	2772.5	2114.2
64	10.6	70	8411.9	10.4	5831.1	2580.8	2040.2
65	10.8	66	7947.1	10.2	5562.7	2384.4	1953.5



Design Equations (Vector Analysis):

$a = c_{FS}*L_{CR}*\sin(90+\phi_{FS})/\sin(\alpha-\phi_{FS})$

$b = W-a$

$P_A = b*\tan(\alpha-\phi_{FS})$

$EFP = 2*P_A/H^2$

Maximum Active Pressure Resultant

$P_{A,\,max}$ 2289.0 lbs/lineal foot

Equivalent Fluid Pressure (per lineal foot of shoring)

$EFP = 2*P_A/H^2$

EFP 11.4 pcf

Design Shoring for an Equivalent Fluid Pressure: **38 pcf**

August 15, 2025 via email: stang@geoteq.com

Geotechnologies, Inc.
439 Western Avenue
Glendale, CA, 91201

Attention: Stanley Tang

Re: Soil Corrosivity Study
Centennial - LACGH
Los Angeles, CA
HDR #25-0336SCS, Geotechnologies #22662

Introduction

Laboratory tests have been completed on four soil samples provided to HDR for the Centennial - LACGH project. The purpose of these tests was to determine whether the soils are likely to have deleterious effects on underground utility piping, hydraulic elevator cylinders, post tensioning systems, concrete piles, and concrete structures. HDR assumes that the samples provided are representative of the most corrosive soils at the site.

The proposed structure will have multiple stories and up to two subterranean levels. The site is located at 1200 North State Street in Los Angeles, California, and the water table is reportedly 26 to 43 feet deep.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. HDR's recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, HDR will be happy to work with them as a separate phase of this project.

Soil Corrosivity Testing

Laboratory Testing

The electrical resistivity of each sample was measured in a soil box per ASTM International (ASTM) G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured per ASTM G51. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327, ASTM D6919, and American Water Works Association (AWWA) Standard Method 2320-B.

The laboratory analyses were performed under HDR laboratory number 25-0336SCS. The full set of test results are shown in the attached Table A1.

Discussion

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil. A correlation between electrical resistivity and corrosivity toward ferrous metals is shown in Table 1.[1]

Table 1: Soil Corrosivity Categories

Soil Resistivity (ohm-cm)	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,001 to 10,000	Moderately Corrosive
1,001 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

Electrical resistivities were in the mildly corrosive to corrosive categories with as-received moisture. When saturated, the resistivities were in the corrosive to severely corrosive categories.

Soil pH values varied from 7.1 to 7.4. This range is neutral to mildly alkaline.[2] These values do not particularly increase soil corrosivity.

The soluble salt content of the samples ranged from low to moderate.

Per American Concrete Institute (ACI)-318, the soil is classified as negligible (S0) with respect to sulfate concentration.[3]

Nitrate was detected in low concentrations. Ammonium was not detected.

Tests were not made for sulfide and oxidation-reduction (redox) potential because these samples did not exhibit characteristics typically associated with anaerobic conditions.

In conclusion, this soil is classified as severely corrosive to ferrous metals and negligible (S0) for sulfate attack on concrete.

Corrosion Control Recommendations

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more

[1] Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.
[2] Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.
[3] American Concrete Institute (ACI) 318-19 Table 19.3.1.1.

practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion. The following recommendations are based on the evaluation of soil corrosivity described above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.

2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

3. To prevent differential aeration corrosion cells, provide at least 2 inches of pipe bedding or backfill material all around metallic piping, including the bottom. Do not lay pipe directly on undisturbed soil.

Steel Pipe

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:

    a. At each end of the pipeline.

    b. At each end of all casings.

    c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

3. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically isolate each buried steel pipeline per The Association for Materials Performance and Protection (AMPP) NACE SP0286 from:

    a. Dissimilar metals.

    b. Dissimilarly coated piping (cement-mortar vs. dielectric).

    c. Above ground steel pipe.

    d. All existing piping.

4. Choose one of the following corrosion control options:

   OPTION 1

   a. Apply a suitable dielectric coating intended for underground use such as:

      i. Polyurethane per AWWA C222 *or*

      ii. Extruded polyethylene per AWWA C215 *or*

      iii. A tape coating system per AWWA C214 *or*

      iv. Hot applied coal tar enamel per AWWA C203 *or*

      v. Fusion bonded epoxy per AWWA C213.

   b. Apply cathodic protection to steel piping as per NACE SP0169.

   OPTION 2

   a. As an alternative to the coating systems described in Option 1 and cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement. Install joint bonds, test stations, and insulated joints to provide for corrosion monitoring and/or the future application of cathodic protection if needed.

NOTE: Some steel piping systems, such as oil, gas, insulated, or high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Ductile Iron Pipe

1. To prevent dissimilar metal corrosion cells and to facilitate the application of cathodic protection, electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE SP0286.

2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and cathodic protection.

3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the application of cathodic protection:

   a. At each end of the pipeline.

   b. At each end of any casings.

   c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

4. Choose one of the following corrosion control options:

   OPTION 1

   a. Apply a suitable coating intended for underground use such as:

      i. Polyethylene encasement per AWWA C105; *or*

      ii. Epoxy coating; *or*

      iii. Polyurethane; *or*

      iv. Wax tape.

      NOTE: The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.

   b. Apply cathodic protection to ductile iron piping as per NACE SP0169.

   OPTION 2

   a. As an alternative to the coating systems described in Option 1 and cathodic protection, encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement. Install joint bonds, test stations, and insulated joints to provide for corrosion monitoring and/or the future application of cathodic protection if needed.

NOTE: Some iron piping systems, such as for fire water piping, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Cast Iron Soil Pipe

1. Protect cast iron soil pipe with either a double wrap 4-mil or single wrap 8-mil polyethylene encasement per AWWA C105.

2. It is not necessary to bond the pipe joints or apply cathodic protection.

3. Provide 6 inches of clean sand backfill all around the pipe. Use the following parameters for clean sand backfill:

   a. Minimum saturated resistivity of no less than 3,000 ohm-cm; *and*

   b. pH between 6.0 and 8.0.

   c. All backfill testing should be performed by a corrosion engineering laboratory.

Copper Tubing

1. Use Type K or Type L copper tubing as required by the applicable local plumbing code. Type M tubing should not be used for buried applications.[4]

2. Electrically insulate underground copper pipe from dissimilar metals and from above ground copper pipe with insulating devices per NACE SP0286. Sleeve copper pipe through footings and foundations to prevent pH concentration cells and prevent leaks caused by settlement.

3. Electrically insulate cold water piping from hot water piping systems.

4. Protect cold water pipe using all of the following measures:

    a. Place cold water copper tubing in an 8-mil polyethylene sleeve or encase in double 4-mil thick polyethylene sleeves. Ensure that sleeves are intact and free of cuts, tears, punctures, or other damage.

    b. Remove any construction debris, rocks, wood, or organic matter from the trench prior to backfill.

    c. Bed and backfill with at least 2 inches of clean sand all around the tubing, including the bedding. Use the following parameters for clean sand backfill:

        i. Minimum saturated resistivity of no less than 3,000 ohm-cm; and

        ii. pH between 6.0 and 8.0.

        iii. All backfill testing should be performed by a corrosion engineering laboratory.

    d. Copper tubing for cold water can also be treated the same as for hot water.

5. Hot water tubing may be subject to a higher corrosion rate. Protect hot copper tubing using one of the following measures:

    a. Prevent soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing with PVC pipe with solvent-welded joints. Either seal the PVC pipe at both ends using ammonia- and methanol-free caulk, or terminate both ends above-grade in a manner that doesn't allow water to infiltrate; *or*

    b. Applying cathodic protection per NACE SP0169. The amount of cathodic protection current needed can be minimized by coating the tubing with a suitable dielectric coating that is compatible with cathodic protection, such as Polyken 930.

---

[4] *2022 California Plumbing Code (CPC) Section 604.3.*

Plastic and Vitrified Clay Pipe

1. No special corrosion control measures are required for plastic and vitrified clay piping placed underground.

2. Protect all metallic fittings and valves with wax tape per AWWA C217, or with epoxy and appropriately designed cathodic protection system per NACE SP0169.

Concrete Structures and Pipe

1. From a corrosion standpoint, any type of ASTM C150 cement may be used for concrete structures and pipe because the sulfate concentration is negligible (S0), from 0 to 0.10 percent. Use a minimum strength of 2,500 psi per applicable codes.[5,6,7]

2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentrations found on site.[8] Limit the water-soluble chloride ion content in the concrete mix design to less than 0.06 percent by weight of cement.[9]

3. Due to the high ground water table encountered at this site, cyclical or continual wetting may be an issue. Any contact between concrete structures and ground water should be prevented as follows:

    a. For structures that extend below the water table, contact can be prevented with an impermeable waterproofing system. Options include a membrane such as Grace PrePrufe® products, a liquid applied barrier coating, or a waterproofing admixture such as Xypex® Admix. Visqueen, similar rolled barriers, or bentonite-based membranes are not viable waterproofing systems for corrosion protection.

    b. For structures above the water table, contact can be prevented with a gravel capillary break under the concrete and a vapor retarding membrane. Note that per ASTM E1643, “vapor retarders are not intended to provide a waterproofing function.”[10] Alternatively, an impermeable waterproofing system may be used.

Post-Tensioned Slabs: Unbonded Single-Stranded Tendons and Anchors

Although chloride levels were relatively low, soil is considered an aggressive environment for post-tensioning strands and anchors. Protect post-tensioning strands and anchors against

---

[5] 2021 International Building Code (IBC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1
[6] 2021 International Residential Code (IRC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1
[7] 2022 California Building Code (CBC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1
[8] Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65
[9] American Concrete Institute (ACI) 318-19 Table 19.3.2.1
[10] ASTM E1643-18a: Standard Practice for Selection, Design, Installation, and Inspection of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs. ASTM International, 2018.

corrosion by implementing all the following measures:[11,12,13]

1. Limit the water-soluble chloride ion content in the concrete mix design to less than 0.06 percent by weight of cement.
2. Design all tendons to prevent ingress of moisture. A corrosion-inhibiting coating should be incorporated into the tendon sheaths.
3. Use non-shrink grout mixes for all post-tensioning pockets.
4. Prior to grouting the pocket, apply a protective grease cap filled with corrosion protection material that provides a watertight seal for the strand end and wedge cavity. Ensure the cap fully seats against the face of the standard anchor at the live end.
5. Protect all components from moisture prior to installation and within one working day after installation.
6. Ensure the minimum concrete cover over the tendon tail is 1 inch, or greater if required by the applicable building code.
7. Install caps within one working day after the cutting of the tendon tails and acceptance of the elongation records by the engineer.
8. Install pre-cast concrete plug over the grease cap to ensure the live end is sealed from further moisture intrusion.
9. Limit the access of direct runoff onto the anchorage area by designing proper drainage. Do not allow water to pond against anchors.
10. Provide at least 2 inches of space between finish grade and the anchorage area, or more if required by applicable building codes.
11. Protect post-tensioned slabs from groundwater in accordance with the recommendations for concrete structures and pipe in this report.

Concrete Piles

Precast Concrete Piles

1. It is assumed that precast concrete piles will contain a minimum of 8 sacks of ASTM C150 Type V cement per cubic yard of concrete, a water/cement ratio not exceeding 0.45, and 2 inches of concrete cover. No further corrosion control measures are required for such piles.

---

[11] Post-Tensioning Manual, sixth edition. Post-Tensioning Institute (PTI), Phoenix, AZ, 2006.
[12] PTI M10.2-17: Specification for Unbonded Single Strand Tendons. Post-Tensioning Institute (PTI), Phoenix, AZ, 2017.
[13] ACI 423.6-01: Specification for Unbonded Single Strand Tendons. American Concrete Institute (ACI), 2001

2. If groundwater is present, solid steel lifting lugs are recommended to prevent groundwater from wicking into the pile interior. If wire rope lifting lugs are used, they should be carefully drilled out 1.5 inches deep and the hole filled with epoxy.

3. HDR understands that there may be no practical way to waterproof precast concrete piles. The concrete mix design for the piles should include supplementary cementitious admixtures to reduce permeability.

Steel Reinforced Cast in Place Concrete Piles

1. Protect steel reinforced cast-in-place and cast-in-drilled-hole concrete piles in accordance with the recommendations of the concrete structures section in this report.

2. If the piles are uncased or there is uncertainty about the shaft profile, consider whether additional concrete cover will be needed to ensure the minimum is achieved.

3. HDR understands that there may be no practical way to waterproof cast in place concrete piles. The concrete mix design for the piles should include supplementary cementitious admixtures to reduce permeability.

Hydraulic Elevators

1. Choose one of the following corrosion control options for the hydraulic steel cylinders.

   OPTION 1

   a. Coat hydraulic elevator cylinders with a suitable dielectric coating intended for underground use such as:

      i. Polyurethane per AWWA C222 or

      ii. Extruded polyethylene per AWWA C215 or

      iii. A tape coating system per AWWA C214 or

      iv. Hot applied coal tar enamel per AWWA C203 or

      v. Fusion bonded epoxy per AWWA C213.

   b. Electrically insulate each cylinder from building metals by installing dielectric material between the piston platen and car, insulating the bolts, and installing an insulated joint in the oil line; and

   c. Apply cathodic protection to hydraulic cylinders as per NACE SP0169.

   OPTION 2

   a. As an alternative to electrical insulation and cathodic protection, place each cylinder in a plastic casing with a plastic watertight seal at the bottom.

2. The elevator oil line should be placed above ground if possible but, if underground, should be protected by one of the following corrosion control options:

   OPTION 1

   a. Provide a bonded dielectric coating,

   b. Electrically isolate the pipeline, and

   c. Apply cathodic protection to steel piping as per NACE SP0169.

   OPTION 2

   a. Place the oil line in a PVC casing pipe with solvent-welded joints and sealed at both ends to prevent contact with soil and moisture.

Closure

The analysis and recommendations presented in this report are based upon data obtained from the laboratory samples. This report does not reflect variations that may occur across the site or due to the modifying effects of construction. If variations appear, HDR should be notified immediately so that further evaluation and supplemental recommendations can be provided.

HDR's services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
HDR Engineering, Inc.



Steven Pierce, PE
Materials Engineer

Bradley M. Stuart, PE
Corrosion Deputy Project Manager

Enc: Table A1 – Laboratory Test on Soil Samples

25-0336SCS SCS TA_BS SP.docx

Table A1 - Laboratory Tests on Soil Samples

Geotechnologies, Inc.
Centennial - LACGH
Your #22662, HDR Lab #25-0336SCS
31-Jul-25

Sample ID			B2 @ 7.5'	B5 @ 15'	B7 @ 5'	B10 @ 2.5'
Resistivity		**Units**				
as-received		ohm-cm	11,200	2,040	10,000	1,080
saturated		ohm-cm	1,240	1,040	1,880	680
pH			7.4	7.1	7.3	7.1
Electrical Conductivity		mS/cm	0.27	0.23	0.14	0.25
Chemical Analyses						
Cations						
calcium	Ca^{2+}	mg/kg	84	39	26	46
magnesium	Mg^{2+}	mg/kg	22	19	11	15
sodium	Na^{1+}	mg/kg	142	208	97	196
potassium	K^{1+}	mg/kg	8.0	1.0	1.0	8.0
ammonium	NH_4^{1+}	mg/kg	ND	ND	ND	ND
Anions						
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	268	540	250	174
fluoride	F^{1-}	mg/kg	4.0	28	14	9.0
chloride	Cl^{1-}	mg/kg	26	15	7.0	49
sulfate	SO_4^{2-}	mg/kg	292	90	35	301
nitrate	NO_3^{1-}	mg/kg	1.0	1.0	2.0	7.0
phosphate	PO_4^{3-}	mg/kg	ND	ND	4.0	4.0
Other Tests						
% moisture	H_2O	%	na	na	na	na
total acidity	H^{1+}	mmol/kg	na	na	na	na
sulfide	S^{2-}	qual	na	na	na	na
Redox		mV	na	na	na	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

LOGS BY GEOTECHNOLOGIES, INC., NOVEMBER 5, 2024, FILE NO. 22579 (4 BORINGS)

(10 PAGES)

BORING LOG NUMBER 1

Centennial GH Partners, LLC **Date: 10/09/24** **Elevation: 312.8'**

File No. 22579 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		9-inch Asphalt, No Base
				1 --		FILL: Sandy Silt, yellow to dark brown, moist, stiff
2.5	47	3.9	126.6	2 --		Silty Sand to Sand, dark brown, slightly moist, medium dense, fine grained, occasional cobbles, brick and asphalt fragments
				3 --		
				4 --		
5	12	22.9	SPT	5 --		Silty Sand to Sandy Clay, dark brown, moist, medium dense to medium firm, fine grained, rock and brick fragments
				6 --		
7.5	20	23.5	100.4	7 --		
				8 --		
				9 --		
10	9	28.1	SPT	10 --		
				11 --	CL	ALLUVIUM: Sandy Clay, dark brown to dark gray, moist, medium firm to stiff
12.5	23	28.8	92.2	12 --		
				13 --		Sandy Clay, dark brown to yellowish brown, moist, firm to stiff
				14 --		
15	10	28.7	SPT	15 --		
				16 --		
17.5	27	28.4	95.7	17 --		
				18 --	CH	Silty Clay, dark brown to gray, moist, firm to stiff
				19 --		
20	11	26.0	SPT	20 --		
				21 --		
22.5	33	23.2	102.3	22 --		
				23 --		Silty Clay, dark brown to gray, moist, stiff
				24 --		
25	14	22.6	SPT	25 --		

BORING LOG NUMBER 1

Centennial GH Partners, LLC

File No. 22579

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26		
				27		
27.5	60	16.2	115.1	28	SM/SP	Silty Sand to Sand, gray to dark gray, moist, dense, fine grained
				29		
30	72	16.8	SPT	30	SP	Sand, gray to dark gray, wet, dense
				31		
				32		
32.5	88	20.9	104.8	33		Sand, dark gray, wet, very dense, fine to medium grained
				34		
35	77	19.1	SPT	35		Sand, dark gray, wet, dense, fine grained
				36		
				37		
37.5	38 50/5"	28.3	97.7	38	SM/SP	Silty Sand to Sand with pebbles, gray to dark gray, wet, very dense, fine to medium grained, with gravel
				39		
40	90	13.5	SPT	40	SM/SW	Silty Sand to Gravelly Sand, dark gray, wet, very dense, fine to coarse grained
				41		
				42		
42.5	100/8"	12.5	123.9	43	SW	Gravelly Sand, dark gray, wet, very dense, fine to coarse grained
				44		
45	88	11.3	SPT	45	SP	Sand, gray to dark gray, moist to wet, very dense, fine to coarse grained
				46		
				47		
47.5	100/8"	16.5	117.8	48		Sand, dark gray, wet, very dense, fine to medium grained
				49		
50	89	16.2	SPT	50		Sand, dark gray, wet, very dense, fine to medium grained

BORING LOG NUMBER 1

Centennial GH Partners, LLC

File No. 22579

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51 --		
				52 --		
52.5	100/8"	14.5	106.3	53 --		Sand, dark gray, wet, very dense, fine to medium grained
				54 --		
55	42 50/5"	13.2	SPT	55 --		Sand, gray to dark gray, wet, very dense, fine to coarse grained
				56 --		
				57 --		
57.5	100/9"	10.2	129.0	58 --		Sand, gray to dark gray, wet, very dense, fine to coarse grained
				59 --		
60	86	18.2	SPT	60 --		Total Depth 60 feet Water at 29½ feet Fill to 10 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted SPT=Standard Penetration Test
				61 --		
				62 --		
				63 --		
				64 --		
				65 --		
				66 --		
				67 --		
				68 --		
				69 --		
				70 --		
				71 --		
				72 --		
				73 --		
				74 --		
				75 --		

BORING LOG NUMBER 2

Centennial GH Partners, LLC **Date: 10/10/24** **Elevation: 314.3'**

File No. 22579 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Driveway
				0 --		4-inch Asphalt over 4-inch Base
				-		
				1 --		FILL: Sandy Clay with rock fragments, yellowish brown, moist, medium firm
				-		
				2 --		
2.5	19	19.5	102.8	-		
				3 --		
				-		
				4 --		
				-		
5	5	15.6	SPT	5 --		Clayey Sand, dark to yellowish brown, moist, medium dense to medium firm, fine grained, minor asphalt and brick fragments
				-		
				6 --		
				-		
				7 --		
7.5	17	20.4	103.1	-		Silty Clay, dark brown, moist, firm
				8 --		
				-		
				9 --		
				-		
10	12	19.3	SPT	10 --	CL	ALLUVIUM: Sandy Clay, dark brown, moist, medium firm to stiff
				-		
				11 --		
				-		
				12 --		
12.5	24	21.8	106.1	-		
				13 --		
				-		
				14 --		
				-		
15	15	21.2	SPT	15 --		Sandy Clay, dark brown, moist, medium firm to stiff
				-		
				16 --		
				-		
				17 --		
17.5	29	22.3	108.1	-		
				18 --		
				-		
				19 --		
				-		
20	26	19.4	SPT	20 --	SC	Clayey Sand, gray to yellowish brown, moist, medium dense, fine grained
				-		
				21 --		
				-		
				22 --		
22.5	28 50/4"	14.2	114.6	-		
				23 --		
				-		
				24 --		
				-		
25	47	5.3	SPT	25 --		
				-	SP	Sand, dark to yellowish brown, slightly moist, dense, fine grained

BORING LOG NUMBER 2

Centennial GH Partners, LLC

File No. 22579

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				26 --		
				27 --		
27.5	39 50/5"	18.5	108.7	28 --	SM/SP	Silty Sand to Sand, yellow to grayish brown, very moist, very dense, fine grained
				29 --		
30	81	20.6	SPT	30 --	SP	Sand, dark to yellowish brown, gray, wet, very dense, fine to medium grained
				31 --		
				32 --		
32.5	40 50/5"	19.1	114.3	33 --		
				34 --		
35	45 50/5"	14.1	SPT	35 --	SM/SP	Silty Sand to Sand, wet, very dense, fine to coarse grained, occasional gravel
				36 --		
				37 --		
37.5	39 50/5"	10.8	124.6	38 --		
				39 --		
40	40 50/5"	15.7	SPT	40 --	SW	Gravelly Sand, dark gray, wet, very dense, fine to coarse grained, with gravel
				41 --		
				42 --		
42.5	48 50/2"	13.6	122.6	43 --	SP	Sand, dark gray, wet, very dense, fine to medium grained, minor with occasional gravel
				44 --		
45	40 50/3"	14.0	SPT	45 --		
				46 --		
				47 --		
47.5	100/8"	11.1	126.2	48 --		
				49 --		
50	50/6"	14.2	SPT	50 --		

BORING LOG NUMBER 2

Centennial GH Partners, LLC

File No. 22579

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				51 --		
				52 --		
52.5	100/9"	10.8	125.2	53 --		Sand, dark gray, wet, very dense, fine to coarse grained, with occasional gravel
				54 --		
55	50/6"	12.6	SPT	55 --		
				56 --		
				57 --		
57.5	100/9"	8.5	127.8	58 --		
				59 --		
60	45 50/5"	13.3	SPT	60 --		Total Depth 60 feet Water at 29½ feet Fill to 10 feet
				61 --		
				62 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				63 --		
				64 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop
				65 --		Modified California Sampler used unless otherwise noted
				66 --		SPT=Standard Penetration Test
				67 --		
				68 --		
				69 --		
				70 --		
				71 --		
				72 --		
				73 --		
				74 --		
				75 --		

BORING LOG NUMBER 3

Centennial GH Partners, LLC **Date: 10/10/24** **Elevation: 318.3'**

File No. 22579 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		3½-inch Asphalt over 8-inch Base
				-		
				1 --		
				-		FILL: Silty Sand to Sandy Silt, dark brown, moist, medium dense, fine grained, stiff
				2 --		
2.5	40	14.9	116.7	-		
				3 --		
				-	CL	ALLUVIUM: Silty Sand to Sandy Clay, dark brown, moist, dense to stiff, fine grained
				4 --		
				-		
5	30	11.5	SPT	5 --		
				-	SM/SP	Silty Sand to Sand, dark to yellowish brown, moist, dense, fine grained
				6 --		
				-		
				7 --		
7.5	69	29.2	94.6	-		
				8 --		
				-	RX	BEDROCK: Siltstone, yellow to grayish brown, moist, moderately hard, weathered
				9 --		
				-		
10	29	22.0	SPT	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	44	26.3	98.9	-		
				13 --		Siltstone to Claystone, dark brown, moist, moderately hard
				-		
				14 --		
				-		
15	32	26.4	SPT	15 --		
				-		
				16 --		
				-		
				17 --		
17.5	38	10.8	105.1	-		
	50/4"			18 --		Sandstone, yellowish brown to grayish brown, moist, moderately hard to hard
				-		
				19 --		
				-		
20	38	12.3	SPT	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	77	25.3	99.6	25 --		
				-		Siltstone, grayish brown, moist, moderately hard

BORING LOG NUMBER 3

Centennial GH Partners, LLC

File No. 22579

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 -- - 28 -- - 29 -- -		
30	77	25.3	102.1	30 -- - 31 -- - 32 -- - 33 -- - 34 -- -		Siltstone, grayish brown, moist, moderately hard
35	85	21.3	106.1	35 -- - 36 -- - 37 -- - 38 -- - 39 -- -		Siltstone, dark grayish brown, moist, moderately hard
40	88	25.8	99.7	40 -- - 41 -- - 42 -- - 43 -- - 44 -- - 45 -- - 46 -- - 47 -- - 48 -- - 49 -- - 50 -- -		Total Depth 40 feet No Water Fill to 3 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted SPT=Standard Penetration Test

BORING LOG NUMBER 4

Centennial GH Partners, LLC **Date: 10/09/24** **Elevation: 322.9'**

File No. 22579 **Method: 8-inch diameter Hollow Stem Auger**

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt for Parking
				0 --		2-inch Asphalt over 7-inch Base
				-		
				1 --		
				-		
				2 --		FILL: Silty Sand, yellowish brown, moist, medium dense to
2.5	50	22.2	104.9	-		dense, fine grained
				3 --		
				-	ML	ALLUVIUM: Sandy Silt, yellowish brown, moist, stiff
				4 --		
				-		
5	20	25.2	SPT	5 --		
				-	RX	BEDROCK: Siltstone, yellowish brown, moist, moderately
				6 --		hard, weathered
				-		
				7 --		
7.5	82	20.2	98.4	-		
				8 --		
				-		
				9 --		
				-		
10	89	22.1	SPT	10 --		
				-		Siltstone, yellowish brown, moist, moderately hard
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	89	26.5	99.2	15 --		
				-		Siltstone, yellowish brown, moist, moderately hard
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	65 50/5"	25.0	98.3	20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
25	45 50/5"	23.9	102.8	25 --		
				-		

BORING LOG NUMBER 4

Centennial GH Partners, LLC

File No. 22579

km

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 -- - 28 -- - 29 -- -		
30	40 50/5"	26.7	93.1	30 -- - 31 -- - 32 -- - 33 -- - 34 -- -		Siltstone, yellowish brown, moist, moderately hard
35	45 50/5"	28.0	96.8	35 -- - 36 -- - 37 -- - 38 -- - 39 -- -		Siltstone to Sandstone, yellowish brown to grayish brown, moist, moderately hard to hard
40	46 50/4"	26.9	96.8	40 -- - 41 -- - 42 -- - 43 -- - 44 -- - 45 -- - 46 -- - 47 -- - 48 -- - 49 -- - 50 -- -		Total Depth 40 feet No Water Fill to 3 feet NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted SPT=Standard Penetration Test

LOGS BY GEOTECHNOLOGIES, INC., FEBRUARY 7, 2024, FILE NO. 22453 (4 BORINGS)

(7 PAGES)

BORING LOG NUMBER 1

Pankow **Date: 01/11/24** **Elevation: 320.5'***

File No. 22473 **Method: 8 inch diameter Hollow Stem Auger**

kk *REFERENCE: Topographic Survey by D.W. & Associates, dated 12/11/23

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Barren Ground
				0 --		UNCERTIFIED FILL: Silty Sand to Clayey Sand, dark brown, moist, medium dense, fine grained
				1 --		
				2 --		
				3 --		
				4 --		
5	11	13.8	SPT	5 --		
				6 --		
				7 --		
7.5	72	15.4	113.2			
				8 --		Sandy Silt to Sandy Clay, dark brown, moist, very stiff, minor asphalt fragments
				9 --		
10	15	13.6	SPT	10 --		
				11 --		stiff
				12 --		
12.5	48	14.5	112.5			
				13 --		Clayey Sand, dark brown, moist, dense, fine grained
				14 --		
15	33	19.4	SPT	15 --		
				16 --	CL/CH	NATIVE SOILS: Silty Clay, dark brown, moist, stiff
				17 --		
17.5	87	24.4	102.2	18 --		
				19 --		
20	21	27.9	SPT	20 --		
				21 --		
				22 --		
22.5	72	25.1	100.8	23 --		
				24 --		
25	16	25.4	SPT	25 --		

BORING LOG NUMBER 1

Pankow

File No. 22473

kk

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 --		
27.5	37	25.5	99.6	- 28 -- - 29 -- -		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual. Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop
30	12	26.6	SPT	30 -- - 31 -- - 32 -- -		Modified California Sampler used unless otherwise noted SPT=Standard Penetration Test
32.5	83	21.5	105.6	33 -- - 34 -- -	ML	Sandy to Clayey Silt, dark gray to gray, moist, very stiff
35	28	23.7	SPT	35 -- - 36 -- - 37 -- -		
37.5	82	17.8	107.6	38 -- - 39 -- -	SP	Sand, gray to dark gray, wet, very dense, fine grained
40	67	20.7	SPT	40 -- - 41 -- - 42 -- -	SM/SP	Silty Sand to Sand, gray to dark gray, very moist, dense, fine grained
42.5	88	13.0	118.8	43 -- - 44 -- -	SP	Sand, dark gray, wet, very dense, fine grained
45	66	23.4	SPT	45 -- - 46 -- - 47 -- -	SM/ML	Silty Sand to Sandy Silt, gray to dark gray, moist to wet, medium dense, very stiff, fine grained
47.5	40 50/5"	25.2	98.2	48 -- - 49 -- -	SM/SP	Silty Sand to Sand, dark gray to gray, wet very dense, fine grained
50	72	25.2	SPT	50 -- -		Total Depth 50 feet Water at 32.5 feet Fill to 15 feet

BORING LOG NUMBER 2

Pankow **Date: 01/11/24** **Elevation: 321.5'***

File No. 22473 **Method: 8 inch diameter Hollow Stem Auger**

kk *REFERENCE: Topographic Survey by D.W. & Associates, dated 12/11/23

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Barren Ground
				0 --		UNCERTIFIED FILL: Silty Sand to Clayey Sand, dark brown, moist, medium dense, fine grained
				1 --		
				2 --		
				3 --		
				4 --		
5	28	13.7	115.1	5 --		Silty Sand, dark brown, moist, medium dense, fine grained, few gravel
				6 --		
				7 --		
				8 --		
				9 --		
10	16	25.2	95.0	10 --		Sandy Silt to Silty Clay, dark brown, moist, stiff
				11 --		
12.5	22	24.4	97.1	12 --		Sand and Clayey Silt, dark and gray, dark brown, moist, stiff, medium dense, fine grained
				13 --		
				14 --		
15	29	23.8	100.7	15 --	CL	NATIVE SOILS: Sandy Silt to Silty Clay, dark brown, moist, stiff
				16 --		
				17 --		
				18 --		
				19 --		
20	47	22.6	99.9	20 --		dark and yellowish brown, moist, stiff
				21 --		
				22 --		
				23 --		
				24 --		
25	62	24.4	102.2	25 --		Silty Clay, dark gray, moist, very stiff, minor rock fragments

BORING LOG NUMBER 2

Pankow

File No. 22473

kk

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				- 26 -- - 27 -- - 28 -- - 29 -- -		
30	62	24.9	97.6	30 -- - 31 -- - 32 -- - 33 -- - 34 -- - 35 -- - 36 -- - 37 -- - 38 -- - 39 -- - 40 -- - 41 -- - 42 -- - 43 -- - 44 -- - 45 -- - 46 -- - 47 -- - 48 -- - 49 -- - 50 -- -		**Total Depth 30 feet** **No Water** **Fill to 15 feet** **NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.** **Used 8-inch diameter Hollow-Stem Auger** **140-lb. Automatic Hammer, 30-inch drop** **Modified California Sampler used unless otherwise noted**

BORING LOG NUMBER 3

Pankow **Date: 01/11/24** **Elevation: 322'***

File No. 22473 **Method: 8 inch diameter Hollow Stem Auger**

kk *REFERENCE: Topographic Survey by D.W. & Associates, dated 12/11/23

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Barren Ground
				0 --		CERTIFIED FILL: Silty Sand, dark brown, moist, dense, fine grained, few gravel
				1 --		
				2 --		
2.5	38 50/5"	12.1	111.7	3 --		very dense, few gravel
				4 --		
5	76	19.9	104.0	5 --		
				6 --		Sandy to Clayey Silt, dark and yellowish brown, moist, very stiff
				7 --		
7.5	44	21.7	105.7	8 --		Silty Clay, dark and yellowish brown, moist, very stiff
				9 --		
10	63	16.4	106.2	10 --		Silty to Sandy Clay, dark brown and gray, moist, very stiff
				11 --		
				12 --		
12.5	44	19.6	101.4	13 --	CL	Sandy to Silty Clay, dark brown and dark gray, moist, stiff
				14 --		
15	35	21.9	100.1	15 --		Silty Clay, dark brown
				16 --		
				17 --		
				18 --		
				19 --		brown, few rock fragments
20	38	21.0	99.6	20 --		Total Depth 20 feet No Water Fill to 13 feet
				21 --		
				22 --		
				23 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				24 --		
				25 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted

BORING LOG NUMBER 4

Pankow **Date: 01/11/24** **Elevation: 323'***

File No. 22473 **Method: 8 inch diameter Hollow Stem Auger**

kk *REFERENCE: Topographic Survey by D.W. & Associates, dated 12/11/23

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Barren Ground
				0 --		CERTIFIED FILL: Silty Sand, dark brown, moist, dense, fine grained, minor rock fragments
				-		
				1 --		
				-		
				2 --		
				-		
3	100/7"	6.5	106.8	3 --		
				-		few gravel, very dense
				4 --		
				-		
5	68	9.4	127.2	5 --		
				-		Clayey Sand, dark brown, moist, very dense, fine grained, few gravel
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		Clayey Sand to Sandy Clay, dark brown, moist, very dense, very stiff, fine grained
10	85	12.2	115.2	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	40	13.4	117.5	-		
	50/5"			13 --		Silty Sand, dark brown, moist, very dense, fine grained, minor rock fragments
				-		
				14 --		
				-		
15	45	14.3	118.1	15 --		
	50/4"			-		Clayey Sand to Sandy Clay, brown to yellowish brown, moist, very dense, very stiff, fine grained, few gravel
				16 --		
				-		
				17 --		
17.5	44	15.2	111.1	-		
	50/5"			18 --		
				-		
				19 --		
				-		
20	42	15.2	109.0	20 --		
	50/5"			-		
				21 --		
				-		
				22 --		
22.5	100/11"	13.3	Disturbed	-		
				23 --		minor concrete fragments
				-		
				24 --		
				-		
				25 --		
				-		

BORING LOG NUMBER 4

Pankow

File No. 22473

kk

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description
				-		
26	75	18.8	111.4	26 --		
				-	CL	NATIVE SOILS: Sandy Clay, dark and yellowish brown, moist, very stiff
				27 --		
27.5	45 50/5"	17.2	114.6	-		
				28 --		
				-		
				29 --		
				-		
30	45 50/4"	17.3	118.3	30 --		
				-		Total Depth 30 feet
				31 --		Water at 24 feet
				-		Fill to 26 feet
				32 --		
				-		
				33 --		
				-		
				34 --		
				-		NOTE: The stratification lines represent the approximate
				35 --		boundary between earth types; the transition may be gradual.
				-		
				36 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				37 --		Modified California Sampler used unless otherwise noted
				-		
				38 --		
				-		
				39 --		
				-		
				40 --		
				-		
				41 --		
				-		
				42 --		
				-		
				43 --		
				-		
				44 --		
				-		
				45 --		
				-		
				46 --		
				-		
				47 --		
				-		
				48 --		
				-		
				49 --		
				-		
				50 --		
				-		

LOGS BY GEOTECHNICAL PROFESSIONALS, INC., NOVEMBER 7, 2022, PROJECT NO. 3080.221

(3 BORINGS)

(7 PAGES)

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
			B	0	4-Inch AC over 3-Inch BASE Fill: **SILTY SAND (SM)** brown, moist, loose, with gravel (2-inch diameter)	
18.4	99	13	D	5	**SILTY CLAY (CL)** dark brown, moist to very moist, firm to stiff, trace porosity	320 315
18.1	104	19	D	10	Natural: **CLAYEY SILT (ML)** dark brown, moist, stiff	
11.3		11	S		**SILTY SAND (SM)** dark grey, moist, loose, fine-grained, trace clay	
4.9					**SAND (SP)** light grey, slightly moist, fine-grained	310
2.4	113	62	D	15	@ 15 feet, dense	
1.7	110	75	D		@ 17.5 feet, yellowish brown, dry to slightly moist, dense to very dense, fine to medium-grained	305
2.3	113	50/6"	D	20	@ 20 feet, slightly moist, very dense, trace gravel	
2.4	112					
5.5	110	50/5"	D		@ 22.5 feet, slightly moist, with gravel (3-inch diameter)	300
24.8		70	D	25	**CLAYEY SILT (ML)** dark brown, wet, hard	295
16.3		27	S	30	@ 30 feet, very moist, very stiff	290
16.1	112	85/11"	D	35	Bedrock: Puente Formation- **SILTSTONE** grey, very moist, hard, slight cementation, interbedded with gray, fine grained sandstone	
					SANDSTONE grey, moist, dense, fine-grained	285

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 9-22-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 54

GPI

PROJECT NO.: 3080.22I

LAC USC OLD PHARMACY

LOG OF BORING NO. B-1

FIGURE A-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
9.6		37	S	40	**SANDSTONE** grey, moist, dense, fine-grained	
7.3						280
8.2	104	50/6"	D	45	@ 45 feet, very dense	
					@ 47.5 feet, Interbedded **SANDSTONE AND CLAYSTONE** grey, wet, dense and hard	275
24.4		45	S	50		
						270
10.3	126	50/5"	D	55	@ 55 feet, moist, very dense	
					CLAYSTONE dark grey, slightly moist, hard, with thin layers of sandstone, (wet)	265
22.5		50/5"	S	60	@ 60 feet, very moist	
						260
				65		
					SANDSTONE grey, very moist, very dense, medium to coarse-grained	255
14.2		50/6"	S	70	@ 70 feet, with cementation	
						250
				75		
						245

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 9-22-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 54

GPI

PROJECT NO.: 3080.22I

LAC USC OLD PHARMACY

LOG OF BORING NO. B-1

FIGURE A-1

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	13.0		50/6"	S	80	**SANDSTONE** grey, moist to very moist, very dense, medium to coarse-grained	
						Total Depth 81.5 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 9-22-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 54

GPI

PROJECT NO.: 3080.22I

LAC USC OLD PHARMACY

LOG OF BORING NO. B-1

FIGURE A-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
			B	0	5-Inch AC	305
					Fill: **SILTY CLAY (CL)** dark brown, very moist, firm to stiff, with gravel (2-inch diameter), with brick and concrete fragments	
					SANDY SILT (ML) olive brown, wet, stiff	
21.8	102	17	D	5		300
					SILTY CLAY (CL) very dark brown, very moist, stiff, trace gravel, with glass fragments	
24.6	97	20	D	10		295
23.0	98	21	D			
23.5	98	10	D	15	Natural: **SANDY CLAY (CL)** dark brown, very moist, firm, with caliche	290
27.6		9	S		@ 17.5 feet, wet, stiff	
29.0	92	17	D	20		285
21.3		7	S		@ 22.5 feet, very moist, firm	
13.6	114	82	D	25	Bedrock: Puente Formation- **SANDSTONE** grey brown, very moist, very dense	280
13.2		65	S	30		275
20.9	106	50/6"	D	35	@ 35 feet, grey, wet, with gravel (<1-inch diameter)	270

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 9-23-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 26

GPI

PROJECT NO.: 3080.22I

LAC USC OLD PHARMACY

LOG OF BORING NO. B-2

FIGURE A-2

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
12.2		50/5"	S	40	**CONGLOMERATIC SANDSTONE** grey, moist, very dense, with gravel (2-inch diameter)	265
12.6	131	50/6"	D	45	@ 45 feet, grey, moist, very dense, with claystone interbeds	260
24.2 15.2		50/6"	S	50	@ 50 feet, very moist to wet	255
21.7	101	95/9"	D	55	@ 55 feet, wet	250
8.5		50/6"	S	60	@ 60 feet, moist, steep bedding, sandstone/pebble conglomerate contact	245
					Total Depth 61.5 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 9-23-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 26

GPI

PROJECT NO.: 3080.22I

LAC USC OLD PHARMACY

LOG OF BORING NO. B-2

FIGURE A-2

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
			B	0	6-Inch AC over 4-Inch BASE	
					Fill: **SILTY SAND (SM)** brown, moist, loose to medium dense, with gravel	305
8.3	114	30	D	5	**CLAYEY SAND (SC)** dark brown, moist, medium dense, trace gravel	300
					@ 8 feet, with wood fragments	
13.2	117	43	D	10	@ 10 feet, very moist, with concrete, mortar, and brick fragments	295
19.6		8	S		**SANDY CLAY (CL)** dark brown, very moist, firm to stiff @ 12 feet, with glass fragments	
21.0	98	28	D	15	@ 15 feet, very stiff @ 16 feet, with brick fragments	290
13.1	108	49	D			
27.7		25	S	20	Bedrock: Puente Formation- **SILTSTONE** dark grey, wet, hard, hydrocarbon odor, with interbedded sandstone and claystone	285
7.3	118	48	D		**SANDSTONE** olive brown, slightly moist to moist, dense	
20.4	105				@ 23 feet, wet	
20.1		27	S	25	@ 25 feet, medium dense	280
6.5	115	89	D	30	@ 30 feet, grey, slightly moist to moist, very dense, coarse-grained	275
24.3	98				@ 31 feet, wet	
21.8		51	S	35	@ 35 feet, dense to very dense	270

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 9-23-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 35

GPI

PROJECT NO.: 3080.22I

LAC USC OLD PHARMACY

LOG OF BORING NO. B-3

FIGURE A-3

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
19.0	105	92/7"	D	40	**SANDSTONE** grey, wet, very dense	265
12.1		50/5"	S	45	@ 45 feet, moist	260
15.1	116	50/3"	S	50	@ 50 feet, very moist	255
12.0		50/6"	S	55	**CONGLOMERATIC SANDSTONE** grey, moist, very dense	250
18.4	113	50/5"	D	60	@ 60 feet, wet	245
					Total Depth 61 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 9-23-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 35

GPI

PROJECT NO.: 3080.22I

LAC USC OLD PHARMACY

LOG OF BORING NO. B-3

FIGURE A-3

LOGS BY GEOTECHNOLOGIES, INC., SEPTEMBER 1, 2021, FILE NO. 22146 (8 BORINGS)

(8 PAGES)

BORING LOG NUMBER 1

Onyx Architects **Date: 07/28/21**

File No. 22146 **Method: 8-Inch Diameter Hollow Stem Auger**

ln

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt For Parking
				0 --		1.5 Inch Asphalt, No Base
				-		FILL: Sandy to Clayey Silt, dark and yellowish brown, moist,
				1 --		stiff
				-		
				2 --		
				-		
3	32	24.8	102.0	3 --		
				-		Sandy to Clayey Silt, dark and yellowish brown, moist, stiff,
				4 --		minor metal and concrete fragments
				-		
5	69	22.7	92.1	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	21	24.2	95.9	-		
				8 --		
				-		
				9 --		
				-		
10	26	29.7	90.0	10 --		
				-		Clayey Silt to Silty Clay, dark and yellowish brown, moist,
				11 --		stiff
				-		
				12 --		
12.5	28	31.5	89.1	-		
				13 --		
				-		
				14 --		
				-		
15	47	31.8	89.3	15 --		
				-		Sandy Silt to Silty Clay, gray to dark gray, moist, stiff
				16 --		
				-		
				17 --		
17.5	31	25.0	95.9	-		
				18 --		Silty Clay, dark gray, moist, stiff
				-		
				19 --		
				-		
20	44	28.7	95.7	20 --		
				-	CL	Silty Clay, dark brown, moist, stiff, minor caliche
				21 --		
				-		
				22 --		
22.5	25	21.3	106.8	-		
				23 --		Silty Clay, dark brown, moist, stiff, minor pebble
				-		
				24 --		Silty Sand, dark brown, very moist, medium dense to dense,
				-	SM	fine grained
25	57	22.4	101.3	25 --		Total Depth: 25 Feet
				-		Water at 25 Feet
						Fill To 20 Feet

BORING LOG NUMBER 2

Onyx Architects **Date: 07/28/21**

File No. 22146 **Method: 8-Inch Diameter Hollow Stem Auger**

ln

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt For Parking
				0 --		2.5 Inch Asphalt, 5.5 Inch Base
				-		FILL: Sandy Silt, dark and yellowish brown, moist, stiff
				1 --		
				-		
				2 --		
2.5	28	23.6	100.4	-		
				3 --		Sandy Silt, dark and yellowish brown, moist, stiff
				-		
				4 --		
				-		
5	31	21.2	94.7	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	26	23.1	98.1	-		
				8 --		
				-	CL	Silty Clay, Dark brown, moist, stiff
				9 --		
				-		
10	20	23.5	100.4	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	26	24.5	101.4	-		
				13 --		
				-		
				14 --		
				-		
15	45	22.8	105.3	15 --		
				-	ML/CL	Clayey Silt to Silty Clay, dark brown, moist, stiff
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	69	23.7	102.8	20 --		
				-		Total Depth: 20 Feet
				21 --		No Water
				-		Fill To 8 Feet
				22 --		
				-		NOTE: The stratification lines represent the approximate
				23 --		boundary between earth types; the transition may be gradual.
				-		
				24 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				25 --		Modified California Sampler used unless otherwise noted
				-		

BORING LOG NUMBER 3

Onyx Architects **Date: 07/28/21**

File No. 22146 **Method: 8-Inch Diameter Hollow Stem Auger**

ln

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt For Parking
				0 --		3.5 Inch Asphalt, 2.5 Inch Base
				-		FILL: Sandy Silt, dark and yellow brown, moist, stiff
				1 --		
				-		
				2 --		
2.5	29	25.7	94.8	-		
				3 --		Sandy Silt, dark and yellowish brown, moist, stiff
				-		
				4 --		
				-		
5	54	25.7	93.9	5 --		
				-		Sandy Silt to Silty Clay, dark and yellowish brown, moist, stiff
				6 --		
				-		
				7 --		
7.5	23	26.4	90.9	-		
				8 --		Sandy Silt, dark and yellowish brown, moist, stiff
				-		
				9 --		
				-		
10	45	19.8	98.8	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	27	23.1	100.7	-		
				13 --		Sandy Silt to Silty Clay, yellowish brown to dark gray, moist, stiff
				-		
				14 --		
				-		
15	27	26.2	95.0	15 --		
				-		Silty Clay to Clayey Silt, dark gray, moist, stiff
				16 --		
				-		
				17 --		
17.5	27	23.7	101.4	-		
				18 --		
				-		
				19 --		
				-		
20	37	28.6	95.3	20 --		
				-	CL	Silty Clay , dark brown, moist, stiff
				21 --		
				-		
				22 --		
22.5	33	23.4	101.7	-		
				23 --		
				-		
				24 --		
				-		
25	41	22.8	104.0	25 --		Total Depth: 25 Feet Water at 21 Feet Fill to 20 Feet
				-		

BORING LOG NUMBER 4

Onyx Architects **Date: 07/28/21**

File No. 22146 **Method: 8-Inch Diameter Hollow Stem Auger**

ln

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Bare Ground
				0 --		Fill: Sandy Silt, dark brown, moist, stiff
				-		
				1 --		
				-		
				2 --		
				-		
				3 --		
				-		
				4 --		
				-		
5	44	18.0	112.7	5 --		
				-		
				6 --		
				-	ML	Sandy to Clayey Silt, dark and yellowish brown, moist, stiff
				7 --		
7.5	83	13.1	124.3	-		
				8 --	SM	Silty Sand, dark and grayish brown, moist, dense, fine grained
				-		
				9 --		
				-		
10	84	25.4	102.4	10 --		
				-	ML	Sandy Silt, dark and grrayish brown, moist, stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	83	22.9	106.6	15 --		
				-		Sandy to Clayey Silt, dark brown, moist, stiff, some caliche
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	72	20.5	101.6	20 --		
				-		Total Depth: 20 Feet No Water Fill To 6 Feet
				21 --		
				-		
				22 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				-		
				23 --		
				-		
				24 --		Used 8-inch diameter Hollow-Stem Auger 140-lb. Automatic Hammer, 30-inch drop Modified California Sampler used unless otherwise noted
				-		
				25 --		
				-		

BORING LOG NUMBER 5

Onyx Architects **Date: 07/29/21**

File No. 22146 **Method: 8-Inch Diameter Hollow Stem Auger**

ln

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt For Parking
				0 --		9.5 Inch Asphalt, 2 Inch Base
				-		FILL: Sandy Silt with Rock Fragments, dark and yellowish
				1 --		brown, moist, stiff
				-		
				2 --		
				-		
3	27	20.3	89.5	3 --		
				-		
				4 --		
				-		
5	16	26.2	83.1	5 --		
				-		
				6 --		
				-		
				7 --		
7.5	23	19.6	94.4	-		
				8 --		
				-		
				9 --		
				-		
10	18	26.4	81.4	10 --		
				-		
				11 --		
				-		
				12 --		
12.5	20	22.8	95.7	-		
	50/3"			13 --		Total Depth: 12.5 Feet
				-		End By Refusal Due To Obstruction
				14 --		No Water
				-		Fill To 12.5 Feet
				15 --		
				-		NOTE: The stratification lines represent the approximate
				16 --		boundary between earth types; the transition may be gradual.
				-		
				17 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				18 --		Modified California Sampler used unless otherwise noted
				-		
				19 --		
				-		
				20 --		
				-		
				21 --		
				-		
				22 --		
				-		
				23 --		
				-		
				24 --		
				-		
				25 --		
				-		

BORING LOG NUMBER 6

Onyx Architects **Date: 07/29/21**

File No. 22146 **Method: 8-Inch Diameter Hollow Stem Auger**

ln

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt For Parking
				0 --		7.5 Inch Asphalt, No Base
				-		FILL: Silty Sand to Sany Silt, dark brown, moist, medium
				1 --		dense, fine grained, stiff
				-		
				2 --		
2.5	50	17.0	115.0	-		
				3 --		
				-	SM	Silty Sand, dark and yellowish brown, moist, very dense
				4 --		
				-		
5	42	8.4	112.9	5 --		
	50/4"			-	SM/SP	Silty Sand to Sand, yellowish and grayish brown, moist, very
				6 --		dense, fine grained
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	88	21.9	104.0	10 --		
				-	ML	Sandy to Clayey Silt, dark and grayish brown, moist, stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	45	13.0	113.2	15 --		
	50/3"			-	SM	Silty Sand, dark and yellowish brown, moist, very dense
				16 --		fine grained
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		
				-		
20	40	28.2	95.5	20 --		
	50/5"			-		Total Depth: 20 Feet
				21 --		No Water
				-		Fill To 3 Feet
				22 --		
				-		NOTE: The stratification lines represent the approximate
				23 --		boundary between earth types; the transition may be gradual.
				-		
				24 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				25 --		Modified California Sampler used unless otherwise noted
				-		

BORING LOG NUMBER 7

Onyx Architects **Date: 07/29/21**

File No. 22146 **Method: 8-Inch Diameter Hollow Stem Auger**

ln

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt For Parking
				0 --		9 Inch Asphalt, 8 Inch Base
				-		Fill: Sandy Silt, dark and yellowish brown, moist, stiff
				1 --		
				-		
				2 --		
2.5	53	19.0	110.0	-		
				3 --		Silty Sand to Sandy Silt, dark and yellowish brown, moist, medium dense, fine grained, stiff
				-		
				4 --		
				-		
5	77	19.7	108.3	5 --		
				-	SM/ML	Silty Sand to Sandy Silt, dark and grayish brown, moist, dense, fine grained, stiff
				6 --		
				-		
				7 --		
7.5	46	17.8	109.9	-		
	50/4"			8 --	SM	Silty Sand, dark and grayish brown, moist, medium dense, fine grained
				-		
				9 --		
				-		
10	90	22.3	103.4	10 --		
				-	ML	Sandy Silt, dark and grayish brown, moist, very stiff
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	42	13.8	118.2	15 --		
	50/4"			-	SM	Silty Sand, dark brown, moist, very dense, fine grained
				16 --		
				-		
				17 --		
				-		
				18 --		
				-		
				19 --		Sand, yellowish and grayish brown, moist, very dense, fine
				-	SP	grained
20	46	10.8	110.3	20 --		
	50/3"			-		Total Depth: 20 Feet No Water, Fill to 5 Feet
				21 --		
				-		
				22 --		NOTE: The stratification lines represent the approximate boundary between earth types; the transition may be gradual.
				-		
				23 --		
				-		Used 8-inch diameter Hollow-Stem Auger
				24 --		140-lb. Automatic Hammer, 30-inch drop
				-		Modified California Sampler used unless otherwise noted
				25 --		
				-		

BORING LOG NUMBER 8

Onyx Architects **Date: 07/29/21**

File No. 22146 **Method: 8-Inch Diameter Hollow Stem Auger**

ln

Sample Depth ft.	Blows per ft.	Moisture content %	Dry Density p.c.f.	Depth in feet	USCS Class.	Description Surface Conditions: Asphalt For Parking
				0 --		8 Inch Asphalt, No Base
				-		FILL: Sandy Silt, dark and yellowish brown, moist, stiff,
				1 --		minor rock fragments
				-		
				2 --		
				-		
				3 --		
				-		
				4 --		
				-		
5	20	22.5	93.8	5 --		
				-		
				6 --		
				-		
				7 --		
				-		
				8 --		
				-		
				9 --		
				-		
10	24	27.2	83.2	10 --		
				-		
				11 --		
				-		
				12 --		
				-		
				13 --		
				-		
				14 --		
				-		
15	32	20.8	101.5	15 --		
				-	ML	Sandy Silt, dark brown, moist, stiff
				16 --		
				-		
				17 --		
17.5	34	23.3	103.9	-		
				18 --		
				-		
				19 --		
				-	ML/CL	Clayey Silt to Silty Clay, dark brown, moist, stiff
20	56	18.7	110.7	20 --		
				-		Total Depth: 20 Feet
				21 --		No Water
				-		Fill To 15 Feet
				22 --		
				-		NOTE: The stratification lines represent the approximate
				23 --		boundary between earth types; the transition may be gradual.
				-		
				24 --		Used 8-inch diameter Hollow-Stem Auger
				-		140-lb. Automatic Hammer, 30-inch drop
				25 --		Modified California Sampler used unless otherwise noted
				-		

LOGS BY AECOM, OCTOBER 21, 2019, JOB NO. 60595699

(10 BORINGS)

(15 PAGES)

Date(s) Drilled	1/24/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Not Encountered		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,703.3 East 1,845,147.4

Boring B-19-01

Sheet 1 of 1

Job Number	60595699
Total Depth Drilled (ft)	21.5
Approximate Ground Surface Elevation (ft)	331.0 MSL

Elevation (ft)	Depth (ft)	Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	0							2.5-inch thick ASPHALT CONCRETE							
330			B-1				CL	4-inch thick BASE **FILL:** **Lean CLAY with SAND** dark yellowish brown			73				
	5		S-1 S-2	16	5 7 9			hard, dark brown, moist, with fine to coarse GRAVEL	14						
							SM	**ALLUVIUM:** **Silty SAND** medium dense, dark yellowish brown, moist, fine grained							
							SC	**Clayey SAND** dense, olive brown, moist, fine grained							
	10		S-3	13	3 5 8				16	112					DSCD
320							ML	**SILT** stiff, olive brown, moist, little CLAY and fine SAND							
	15		S-4	4	1 2 2										
	20		S-5	10	3 4 6				21	103					
310								Bottom of borehole at 21.5 feet Borehole backfilled with cement bentonite grout and soil cuttings with concrete cap to exisiting grade. Groundwater was not encountered.							
	25														
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	1/23/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Groundwater encountered at 32 feet below ground surface		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,695.3 East 1,845,069.5

Boring B-19-02

Sheet 1 of 2

Job Number	60595699
Total Depth Drilled (ft)	61.5
Approximate Ground Surface Elevation (ft)	330.5 MSL

Elevation (ft)	Depth (ft)	Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
330	0							1-inch thick ASPHALT CONCRETE							
			B-1				CL	7.5-inch thick BASE **FILL:** **Lean CLAY with SAND** dark brown with yellowish brown mottling, moist, fine grained			77				
	5		S-1	36	9 15 21			stiff	15	117					
							SM	**Silty SAND** dense, dark yellowish brown, moist, fine grained							
320	10		S-2	22	8 11 11		ML	**SILT with Sand** stiff, light olive brown, moist, fine grained							
	15		S-3	56	14 23 33			some CLAY	23	103					
							CL	**ALLUVIUM:** **Lean CLAY** hard, olive brown, moist, some very fine grained SAND			95				
310	20		S-4 S-5	22	7 9 13										
							CH	**Fat CLAY** hard, olive brown, moist, little fine SAND, high plasticity							
	25		S-6	42	10 18 24				28	97		61	41		
							SC	**Clayey SAND** medium dense, olive brown, moist, fine grained							
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

LAC+USC Medical Center

FOR: Los Angeles County, Department of Public Works

Boring B-19-02

Sheet 2 of 2

Elevation (ft)	Depth (ft)	Sample Type	Sample Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	30	S	S-7	18	4 7 11		SC	**Clayey SAND***(continued)* medium dense, olive brown, moist, fine grained							
-300							SM	**Silty SAND** dense, olive brown, moist, fine grained							
	35	S	S-8	39	13 14 25		CL	**Lean CLAY with SAND** olive brown, moist, fine grained	22	106	87				
	40	S	S-9	31	7 13 18			light olive brown, medium to fine grained							
-290							SM	**Silty SAND** dense, light olive brown, wet, fine to medium grained, some coarse, trace fines							
	45	S	S-10	42	10 18 24				19	113					
							CL	**Sandy Lean CLAY** hard, olive, moist, little coarse GRAVEL							
-280	50	S	S-11	18	4 7 11										
	55	S	S-12	39	13 14 25			some fine SAND							
	60	S	S-13	31	7 13 18			trace fine SAND	23						
-270								Bottom of borehole at 61.5 feet Borehole backfilled with cement bentonite grout to 5 feet above GW, and soil cuttings with concrete cap to existing grade							
	65														

LOG OF BORING

LAC+USC WC Hospital Demolition

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	1/23/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Groundwater encountered at 38 feet below ground surface		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,762.6 East 1,844,944.2

Boring B-19-03

Sheet 1 of 2

Job Number	60595699
Total Depth Drilled (ft)	41.5
Approximate Ground Surface Elevation (ft)	322.5 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	0							3-inch thick ASPHALT CONCRETE							
								9.5-inch thick BASE							
			B-1				CL	**FILL:** **Lean CLAY with SAND** stiff, dark yellowish brown, moist			74				
320															
			S-1	21	5 8 13			becomes hard							
	5														
							SC	**Clayey SAND** dense, dark yellowish brown, moist							
	10		S-2	100+	14 40 50/5"				10	128					
310							SM	**Silty SAND** dense, light olive brown, moist, fine grained, trace fines							
	15		S-3	28	9 12 16			some CLAY							
	20		S-4 S-5	41	20 17 24				16	118	22				
300							CL	**Lean CLAY** hard, olive brown, moist							
							SC	**ALLUVIUM:** **Clayey SAND** dense, light olive brown, moist, fine grained							
	25		S-6 S-7	23	5 9 14				16						
							CL	**Lean CLAY** olive brown, moist							
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

LAC+USC Medical Center

FOR: Los Angeles County, Department of Public Works

Boring B-19-03

Sheet 2 of 2

Elevation (ft)	Depth (ft)	SAMPLES: Type	SAMPLES: Number	SAMPLES: Blows per foot	SAMPLES: Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	30	S	S-8	38	12 16 22		CL	**Lean CLAY with SAND** *(continued)* olive brown, moist	19	111	77	47	33		
290															
	35	S	S-9 S-10	21	6 9 12		SC	**Clayey SAND** hard, dark yellowish brown, moist, fine grained							
	40	S	S-11	43	14 18 25		CL	**Lean CLAY** hard, olive brown, moist	26	100					
280								Bottom of borehole at 41.5 feet Borehole backfilled with cement bentonite grout to 5 feet above GW, and soil cuttings with concrete cap to existing grade							
	45														
	50														
270															
	55														
	60														
260															
	65														

LOG OF BORING

LAC+USC WC Hospital Demolition

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	1/23/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Not Encountered		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,906.2 East 1,844,843.1

Boring B-19-04

Sheet 1 of 1

Job Number	60595699
Total Depth Drilled (ft)	21.5
Approximate Ground Surface Elevation (ft)	318.5 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	0							5-inch thick ASPHALT CONCRETE							CORR (0-5')
			B-1				SC	5.5-inch thick BASE			78				
								FILL: **Lean CLAY with SAND** dark yellowish brown, moist, fine grained							
	5		S-1	32	9 14 18			stiff	13	118					El(101)
310							CL	**ALLUVIUM:** **Lean CLAY** hard, very dark brown, moist, little GRAVEL and trace fine SAND							
	10		S-2 S-3	12	3 5 7										
	15		S-4	22	6 10 12			stiff, light olive brown, moist, little fine SAND and GRAVEL	19	111					
300															
	20		S-5	13	4 6 7			light olive brown with yellowish brown mottling							
								Bottom of borehole at 21.5 feet Borehole backfilled with cement bentonite grout and soil cuttings with concrete cap to exisiting grade. Groundwater was not encountered.							
	25														
290															
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	2/7/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Groundwater encountered at 33 feet below ground surface		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,809.7 East 1,844,729.0

Boring B-19-05

Sheet 1 of 3

Job Number	60595699
Total Depth Drilled (ft)	81.0
Approximate Ground Surface Elevation (ft)	320.0 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
320	0							8-inch thick ASPHALT CONCRETE 7-inch thick BASE							
							SC	**FILL:** **Clayey SAND** dark brown, moist, fine grained							
	5	S-1		14	3 6 8				23	106					
310	10	S-2		3	1 1 2		CL	**ALLUVIUM:** **Lean CLAY with SAND** stiff, olive brown, moist, fine grained	24						
	15	S-3		23	6 11 12				26	100	84	44	31		CON
300	20	S-4		9	3 4 5										SE
	25	S-5		25	6 11 14			becomes very dark grey (5Y, 3/1)	27	97	89				DSCD
290	30						SC	**Clayey SAND** con't							

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	SAMPLES Blows per foot	SAMPLES Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
290	30		S-6	37	14 18 19		SC	**Clayey SAND***(continued)* dark greenish gray, moist, fine grained, little clay							
							SM	**Silty SAND** olive gray, moist, fined grained, trace silt							
	35		S-7	64	17 25 39				23						
							SC	**Clayey SAND** dense, olive gray, moist, fine grained							
280	40		S-8	42	9 20 22										
	45			100+	50/6"		SM	**Silty SAND** dense, olive grey, wet							
270	50		S-9	45	21 22 23				20						
							SC	**Clayey SAND** medium dense, olive gray, moist, fine grained, little gravel							
							CL	**Silty CLAY with SAND** hard, dark grey, moist							
	55		S-11	77	16 35 42				22	103	89	75	57		CON
260	60		S-12	63	18 30 33										
							SC	**Clayey SAND** dense, dark olive brown, moist, fine grained							
	65		S-13	100+	27 50/5"				23	104					

LOG OF BORING

LAC+USC WC Hospital Demolition

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

LAC+USC Medical Center

FOR: Los Angeles County, Department of Public Works

Boring B-19-05

Sheet 3 of 3

Elevation (ft)	Depth (ft)	SAMPLES Type Number	SAMPLES Blows per foot	SAMPLES Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
						SC	**Clayey SAND***(continued)* dense, dark olive brown, moist, fine grained							
250	70	S-14	100+	25 53/8"		ML	**BEDROCK (PUENTE FORMATION):** **Sandy SILT** hard, dark greyish brown, moist, fine gained							
						MLG	**Gravely SILT** hard, dark grayish brown, moist, fine grained, trace of GRAVEL							
	75	S-15	100+	50/25"		ML	**Sandy SILT** hard, dark greyish brown, moist, fine gained	44						
240	80	S-16	100+	30 54/6"										
							Bottom of borehole at 81.0 feet Borehole backfilled with cement bentonite grout to 5 feet above groundwater, and soil cuttings with concrete cap to existing grade							
	85													
230	90													
	95													
220	100													
	105													

LOG OF BORING

LAC+USC WC Hospital Demolition

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	1/23/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Not Encountered		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,532.5 East 1,844,832.2

Boring B-19-06

Sheet 1 of 1

Job Number	60595699
Total Depth Drilled (ft)	20.9
Approximate Ground Surface Elevation (ft)	332.5 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	0		B-1				CL	**FILL:** **Lean CLAY with SAND** very dark brown, moist, little fine SAND and GRAVEL	17						
330															
	5		S-1 S-2	19	6 8 11		CLS	**Sandy Lean CLAY** hard, yellowish brown, dry, with fine SAND			83				
							ML	**SILT** hard, light yellowish brown, moist, with little CLAY							
320	10		S-3	100+	30 50/4"				13	111					
	15		S-4	30	15 13 17		SC	**Clayey SAND** dense, dark yellowish brown, moist, fine grained							
	20		S-5	100+	28 50/5"		CL	**Lean CLAY** dark yellowish brown, moist	10	123					DSCD
310								Bottom of borehole at 21.5 feet Borehole backfilled with cement bentonite grout and soil cuttings with concrete cap to exisiting grade. Groundwater was not encountered.							
	25														
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	1/23/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Not Encountered		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,457.3 East 1,844,944.1

Boring B-19-07

Sheet 1 of 2

Job Number	60595699
Total Depth Drilled (ft)	41.5
Approximate Ground Surface Elevation (ft)	333.0 MSL

Elevation (ft)	Depth (ft)	Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	0							4-inch thick ASPHALT CONCRETE							
								5-inch thick BASE							
330		B	1				CL	**FILL:** **Lean CLAY with SAND** very dark brown, moist, little fine SAND			73	36	22		
	5			46	11 20 26			hard	16	117					
		S	1				SC	**Clayey SAND** dense, dark yellowish brown, moist							
							SM	**Silty SAND** dense, dark yellow, moist							
	10	S	2	23	9 10 13										
		S	3				ML	**SILT** hard, light olive brown, moist, few fine grained SAND							
320															
	15	S	4	100+	23 50/5"				13	115	89				
							SC	**Clayey SAND** dense, dark yellowish brown, moist, fine grained, fines decrease with depth							
	20	S	5	19	7 9 10										
310							SM	**Silty SAND** dense, light olive brown, moist, little SILT							
	25	S	6	100+	50/5"				8	116					
							ML	**ALLUVIUM:** **SILT** olive, moist, little fine SAND							
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

LAC+USC Medical Center

FOR: Los Angeles County, Department of Public Works

Boring B-19-07

Sheet 2 of 2

Elevation (ft)	Depth (ft)	Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	30		S-7	18	5 7 11		ML	**SILT** *(continued)* very stiff, olive, moist, little fine SAND							
300							CL	**Lean CLAY** hard, olive brown, moist, some SILT and fine SAND							
	35		S-8	50	12 20 30				23	102					DSCD
	40		S-9	19	5 8 11			dark yellowish brown							
290								Bottom of borehole at 41.5 feet Borehole backfilled with cement bentonite grout and soil cuttings with concrete cap to exisiting grade. Groundwater was not encountered.							
	45														
	50														
280															
	55														
	60														
270															
	65														

LOG OF BORING

LAC+USC WC Hospital Demolition

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	1/23/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Not Encountered		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,591.1 East 1,845,058.5

Boring B-19-08

Sheet 1 of 1

Job Number	60595699
Total Depth Drilled (ft)	21.5
Approximate Ground Surface Elevation (ft)	332.5 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	SAMPLES Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	0							4-inch thick ASPHALT CONCRETE							
330			B-1				CL	5-inch thick BASE **FILL:** **Lean CLAY with SAND** dark brown, moist, fine grained SAND							
	5		S-1	29	6 12 17			hard, dark yellowish brown	15		76				
	10		S-2 S-3	68	18 28 40				14	117					El(111)
320							ML	**SILT** hard, light yellowish brown, moist, with some CLAY							
	15		S-4	35	11 16 19										
							CL	**Sandy Lean CLAY** hard, dark brown, moist, some fine grained							
	20		S-5	51	12 21 30				20	108					
310								Bottom of borehole at 21.5 feet Borehole backfilled with cement bentonite grout and soil cuttings with concrete cap to exisiting grade. Groundwater was not encountered.							
	25														
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	2/6/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Not Encountered		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,524.5 East 1,845,058.6

Boring B-19-09

Sheet 1 of 1

Job Number	60595699
Total Depth Drilled (ft)	11.5
Approximate Ground Surface Elevation (ft)	333.0 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	0							3-inch thick ASPHALT CONCRETE							
			B-1				SC	6-inch thick BASE	15						R-VALUE
330								**FILL:** **Clayey SAND** dense, dark yellowish brown, moist, fine grained							
	5		S-1	39	10 16 23										
							ML	**SILT** hard, pale olive, moist, fine grained, trace SAND							
	10		S-2	18	8 9 9										CORR
320								Bottom of borehole at 11.5 feet Borehole backfilled with filter sand and soil cuttings with concrete cap to exisiting grade. Groundwater was not encountered.							
	15														
	20														
310															
	25														
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

Date(s) Drilled	2/6/19	Logged By	S. Denise
Drilling Method	Hollow Stem Auger	Drill Bit Size	8-inch O.D.
Drill Rig Type	LAR CME 75	Hammer Data	140 lbs, 30-inch drop
Sampling Method(s)	Bulk, SPT, Modified California		
Approximate Groundwater Depth and Date Measured	Not Encountered		
Coordinate System	California State Plane Coord. Zone 5 NAD83 (ft)	Location (ft and/or °)	North 6,497,681.0 East 1,844,753.3

Boring B-19-10

Sheet 1 of 1

Job Number	60595699
Total Depth Drilled (ft)	17.5
Approximate Ground Surface Elevation (ft)	333.0 MSL

Elevation (ft)	Depth (ft)	SAMPLES Type	Number	Blows per foot	Blows per 6 inches	Graphic Log	USCS	MATERIAL DESCRIPTION	Moisture Content (%)	Dry Density (pcf)	% Passing No. 200 Sieve	Liquid Limit (%)	Plasticity Index (%)	PID Reading (ppm)	OTHER TESTS AND REMARKS
	0		B-1				SM	**FILL:** **Silty SAND** dense, olive brown, moist, fine grained, little CLAY							
330															
	5		S-1	19	5 8 11										
							SC	**Clayey SAND** dense, olive brown, moist, fine grained							
	10		S-2	11	4 5 6										
320															
	15		S-3	16	4 10 6										
								Bottom of borehole at 17.5 feet Borehole backfilled with filter sand and soil cuttings with concrete cap to exisiting grade. Groundwater was not encountered.							
	20														
310															
	25														
	30														

This log is part of the report prepared by URS for this project and should be read together with the report. This summary applies only at the location of the exploration and at the time of drilling or excavation. Subsurface conditions may differ at other locations and may change at this location with time. Data presented are a simplification of actual conditions encountered.

LOG OF BORING

LAC+USC WC Hospital Demolition
LAC+USC Medical Center
FOR: Los Angeles County, Department of Public Works

AECOM

Report: DMG4; Project File: C:\USERS\WEICHENG.JIANG\DESKTOP\LAC+USC\LACDPW_PMD_LAC_USC_WD HOSPITAL DEMO.GPJ; Data Template:URS HSA LOGS 2010.GDT Printed: 10/22/19

LOGS BY GEOCON WEST, INC., MAY 20, 2019, PROJECT NO. A8559-06-74 (10 BORINGS)

(10 PAGES)

PROJECT NO. A8559-06-74

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 1** ELEV. (MSL.) **328** DATE COMPLETED **4/22/19** EQUIPMENT **HAND AUGER** BY: **RP** MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silt with Clay, soft to firm, slightly moist, dark brown with yellowish brown mottles, fine-grained, plastic, some sandstone/siltstone fragments, some rootlets.			
2	B1@2.5'				- yellowish brown with olive brown mottles		92.3	23.8
4	B1@5'				- increase in bedrock fragments, trace quartz fragments		101.7	19.1
6								
8	B1@7.5'				- trace rootlets, slight oxidation staining, trace carbon deposits		96.0	23.4
10	B1@10'						94.5	27.7
					- refusal on rock Total depth of boring: 11 feet All fill. No groundwater encountered. Backfilled with soil cuttings and tamped.			

A8559-06-74 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 2** ELEV. (MSL.) **323** DATE COMPLETED **4/22/19** EQUIPMENT **HAND AUGER** BY: **RP** MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 4" BASE: 12"**			
					ARTIFICIAL FILL			
2					Silt with Clay, firm, slightly moist, yellowish brown with brown mottles, some bedrock fragments, plastic, trace rootlets.			
	B2@3'						102.5	23.1
4					- yellowish brown with olive brown mottles, trace diatomaceous content, some oxidation.			
6	B2@6'						96.1	26.8
8					- soft, brown with yellowish brown mottles, increase in moisture - refusal on hard surface			
					Total depth of boring: 9 feet All fill. No groundwater encountered. Backfilled with soil cuttings and tamped. Surface patched with asphalt.			

A8559-06-74 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 3** ELEV. (MSL.) **323** DATE COMPLETED **4/22/19** EQUIPMENT **HAND AUGER** BY: **RP**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**AC: 4" BASE: 12"** **ARTIFICIAL FILL** Silt with Clay, firm, slightly moist, yellowish brown with brown mottles, some bedrock fragments, plastic, trace rootlets.			
2								
4					- yellowish brown with olive brown mottles, trace diatomaceous content, some oxidation.			
6								
8					- soft, moist, brown with yellowish brown mottles - refusal on hard surface Total depth of boring: 8.7 feet All fill. No groundwater encountered. Backfilled with soil cuttings and tamped. Surface patched with asphalt.			

A8559-06-74 BORING LOGS.GPJ

Figure A3,
Log of Boring 3, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

BORING 4

ELEV. (MSL.) **323** DATE COMPLETED **4/23/19**

EQUIPMENT **HAND AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**AC: 4" BASE: 12"**			
					ARTIFICIAL FILL			
2					Silt with Clay, slightly moist, yellowish brown with brown mottles, some bedrock fragments, plastic, trace rootlets.			
4					- yellowish brown with olive brown mottles, trace diatomaceous content			
6								
8								
10			▼		- soft, brown			
					PUENTE FORMATION			
12	B4@11.5'				Sandy Siltstone, brown with olive brown mottles, fine-grained, poorly bedded, massive, hard, moderately weathered.		---	---
	B4@12.5'				- no recovery		94.4	29.0
	B4@13'						---	---
					Total depth of boring; 13.5 feet			
					Fill to 10 feet.			
					Perched groundwater encountered at 10 feet.			
					Backfilled with soil cuttings and tamped.			
					Surface patched with asphalt.			

A8559-06-74 BORING LOGS.GPJ

Figure A4,
Log of Boring 4, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 5** ELEV. (MSL.) **323** DATE COMPLETED **4/23/19** EQUIPMENT **HAND AUGER** BY: **RP**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**AC: 4" BASE: 12"** **ARTIFICIAL FILL** Silt with Clay, slightly moist, yellowish brown with brown mottles, some bedrock fragments, plastic, trace rootlets.			
2								
4					- yellowish brown with olive brown mottles, trace diatomaceous content			
6								
8								
10					- soft, brown **PUENTE FORMATION** Sandy Siltstone, brown with olive brown mottles, fine-grained, poorly bedded, massive, hard, moderately weathered.			
12								
					Total depth of boring; 13 feet Fill to 10 feet. Perched groundwater encountered at 10 feet. Backfilled with soil cuttings and tamped. Surface patched with asphalt.			

A8559-06-74 BORING LOGS.GPJ

Figure A5,
Log of Boring 5, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 6** ELEV. (MSL.) **324** DATE COMPLETED **4/23/19** EQUIPMENT **HAND AUGER** BY: **RP**	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**GRADED SLOPE** **ARTIFICIAL FILL** Silt with Clay, slightly moist, yellowish brown with brown mottles, some bedrock fragments, plastic, trace rootlets.			
2								
4					- yellowish brown with olive brown mottles, trace diatomaceous content, some oxidation			
6								
8								
10					- refusal on hard surface Total depth of boring: 10 feet All fill. No groundwater encountered. Backfilled with soil cuttings and tamped.			

Figure A6,
Log of Boring 6, Page 1 of 1

A8559-06-74 BORING LOGS.GPJ

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

BORING 7

ELEV. (MSL.) **335** DATE COMPLETED **4/23/19**

EQUIPMENT **HAND AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ARTIFICIAL FILL** Silt with Clay, firm, slightly moist, yellowish brown with olive brown mottles, some bedrock fragments.			
2	B7@2.5'						90.2	27.2
4								
6	B7@5'				- brown with dark brown mottles, slight increase in moisture, plastic		104.5	17.6
8	B7@7.5'				- brown to yellowish brown, increase in silt some rootlets		101.1	19.5
10	B7@10'				**COLLUVIUM** Silt with Clay, firm, slightly moist, brown with olive brown mottles, some carbon deposits.		106.4	21.7
12	B7@12'				- grayish brown, decrease in silt, plastic, increase in moisture - reddish brown with olive brown mottles, increased carbon deposits, some fine-grained sand		108.4	19.3
14				CL-ML				
16			▼					
18	B7@17.5'				**PUENTE FORMATION** Sandy Siltstone, yellowish brown with olive brown mottles, fine-grained, poorly bedded, massive, hard, highly weathered.		109.3	18.5

Total depth of boring: 18 feet
Fill to 10 feet.
Perched groundwater enountered at 16 feet.
Backfilled with soil cuttings and tamped.

A8559-06-74 BORING LOGS.GPJ

Figure A7,
Log of Boring 7, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

BORING 8

ELEV. (MSL.) **323** DATE COMPLETED **4/30/19**

EQUIPMENT **BUCKET AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK @0-5'				**AC: 3.5" BASE: 12"** **ARTIFICIAL FILL** Silt with Clay, firm, slightly moist, yellowish brown with olive brown mottles, fine-grained, some bedrock fragments, plastic.			
2								
4	B8@3'				- dark brown	1	97.2	26.0
6	B8@6'				**COLLUVIUM** Silt with Clay, firm to hard, slightly moist, reddish brown, some caliche.	5	110.6	18.8
8					- abundant diatomaceous content			
10	B8@9'			CL-ML	- hard	10	110.0	17.4
12	B8@12'				- brown to reddish brown	6	110.7	19.7
14					Silty Sand, medium dense, slightly moist, olive brown, fine-grained, some clay, some oxidation staining.			
16	B8@15'			SM		7	112.4	19.5
18			▼		- moist, groundwater encountered			
20	B8@20'				**PUENTE FORMATION** Sandy Siltstone, brown with olive brown mottles, poorly bedded, massive, medium dense, moderately weathered to highly weathered.	6	100.7	24.5
22								
24								
	B8@25'					13	107.9	19.0
					Total depth of boring: 25.5 feet Fill to 6 feet. Groundwater encountered at 18 feet. Backfilled with soil cuttings and tamped. Surface patched with asphalt. *Kelley Bar Weights: 0-24' 2,400 lbs 25-44' 1,550 lbs			

A8559-06-74 BORING LOGS.GPJ

Figure A8,
Log of Boring 8, Page 1 of 1

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

⊠ ... DISTURBED OR BAG SAMPLE ◪ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

BORING 9

ELEV. (MSL.) **326** DATE COMPLETED **4/30/19**

EQUIPMENT **BUCKET AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK @0-5'				**AC: 3" BASE: 12"** **ARTIFICIAL FILL** Silt with Clay, firm, slightly moist, dark brown with olive brown mottles, trace gravel, trace coarse-grained sand.			
2								
4	B9@3'				**COLLUVIUM** Silt with Clay, firm, slightly moist, dark brown, plastic, some oxidation mottling.	2	106.4	19.0
6	B9@6'				- hard, some silt	5	112.0	14.4
8				CL-ML	- brown, increase in silt - firm, trace bedrock fragments			
10	B9@9'		▼		- olive brown to yellowish brown - groundwater encountered	10	109.2	18.8
12	B9@12'				- some oxidation staining, trace Silty Sand interbeds	1	108.7	19.1
14								
16	B9@15'				**PUENTE FORMATION** Siltstone, yellowish brown with olive brown mottles, poorly bedded, massive, hard, highly to completely weathered, some oxidation.	2	113.7	18.5
18								
20	B9@20'				- olive brown with brown mottles, some calcium carbonate	5	111.1	19.2
					Total depth of boring: 20.5 feet Fill to 3 feet. Perched groundwater encountered at 11 feet. Backfilled with soil cuttings and tamped. Surface patched with asphalt. *Kelley Bar Weights: 0-24' 2,400 lbs 25-44' 1,550 lbs			

A8559-06-74 BORING LOGS.GPJ

Figure A9,
Log of Boring 9, Page 1 of 1

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

⊠ ... DISTURBED OR BAG SAMPLE ◪ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A8559-06-74

BORING 10

ELEV. (MSL.) **321** DATE COMPLETED **4/30/19**

EQUIPMENT **BUCKET AUGER** BY: **RP**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK @0-5'				**ARTIFICIAL FILL** Silt with Clay, firm, slightly moist, brown with dark brown mottles, fine- to medium-grained, plastic, some fine gravel, trace glass fragments.			
2					- some interbedded bedrock fragments			
4	B10@3'					7	110.0	14.9
6	BULK @5'-10'				- yellowish brown with olive brown mottles			
8	B10@6'				- trace concrete	4	96.7	20.2
10	B10@9'		▼		**COLLUVIUM** Silt with Clay, firm, slightly moist, dark brown, plastic.	PUSH	106.8	22.7
12	B10@12'				- groundwater encountered - brown, some caliche	2	108.2	20.5
14				CL-ML				
16	B10@15'				- some sandstone fragments	3	104.6	18.2
18								
20	B10@20'				**PUENTE FORMATION** Siltstone, brown with olive brown mottles, poorly bedded, massive, hard, moderately weathered.	5	98.5	26.1
22								
24	B10@24.5					10	103.5	25.3
	B10@25'				Total depth of boring: 25.5 feet Fill to 8.5 feet. Groundwater encountered at 11 feet. Backfilled with soil cuttings and tamped. *Kelley Bar Weights: 0-24' 2,400 lbs 25-44' 1,550 lbs	19	100.2	25.5

A8559-06-74 BORING LOGS.GPJ

Figure A10,
Log of Boring 10, Page 1 of 1

SAMPLE SYMBOLS

☐ ... SAMPLING UNSUCCESSFUL ◧ ... STANDARD PENETRATION TEST ■ ... DRIVE SAMPLE (UNDISTURBED)

⊠ ... DISTURBED OR BAG SAMPLE ◥ ... CHUNK SAMPLE ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

LOGS BY GEOCON WEST, INC., APRIL 9, 2018, PROJECT NO. A9733-06-01 (5 BORINGS)

(6 PAGES)

PROJECT NO. A9733-06-01

BORING 1

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**GRAVEL** **ARTIFICIAL FILL** Silty Sand, loose, dry, brown, fine- to medium-grained, some coarse-grained, some gravel (to 1"), some brick and asphalt fragments.			
2	B1@2.5'			SM	**ALLUVIUM** Silty Sand, medium dense, slightly moist, brown, fine- to medium-grained, some coarse-grained.	51	115.7	14.4
4	B1@5'				- trace coarse-grained	15	--	--
6								
8	B1@7.5'			ML	Sandy Silt, hard, slightly moist, dark brown, fine- to medium-grained, some coarse-grained.	63	121.5	15.2
10	B1@10'			SM	Silty Sand, dense, slightly moist, dark brown, fine- to medium-grained, some coarse-grained, some gravel (to 0.5").	41	--	--
12	B1@12.5'				Silt, firm, slightly moist, brown, trace fine- to coarse-grained sand.	20	108.4	19.6
14								
16	B1@15'		▼	ML	- soft, wet, fine- to medium-grained, some coarse-grained	6	--	--
18	B1@17.5'				- firm, light brown to yellowish brown	15	103.4	24.1
20	B1@20'				- saturated, yellowish brown	11	--	--
22	B1@22.5'				**PUENTE FORMATION** Interbedded Siltstone and Sandstone, yellowish brown, fine- to medium-grained, thinly bedded, soft, highly weathered, heavy oxidation.	24	94.4	29.4
24								
26	B1@25'					14	--	--
28	B1@27.5'				- moderately hard	43	97.5	28.5

A9733-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 1 of 2

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING 1** ELEV. (MSL.) -- DATE COMPLETED **3/6/18** EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH** MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B1@30'					40	--	--
32	B1@32.5'				- hard	50 (5")	103.3	25.4
34								
36	B1@35'					51	--	--
38								
40	B1@40'					50 (6")	--	--
42								
44								
46	B1@45'					54	--	--
48								
50	B1@50'					50 (4")	--	--
52								
54								
56					Total depth of boring: 56 feet Fill to 1.5 feet. Groundwater encountered at 17 feet. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.			

A9733-06-01 BORING LOGS.GPJ

Figure A1,
Log of Boring 1, Page 2 of 2

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

BORING 2

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**GRAVEL** **ARTIFICIAL FILL** Silty Sand, loose, dry, brown, fine- to medium-grained, some coarse-grained, some gravel (to 1"), fragments of brick and asphalt.			
2	B2@2'					65	101.2	15.3
4					**ALLUVIUM** Silty Sand, dense, slightly moist, brown, fine- to medium-grained, some coarse-grained.			
6	B2@5'			SM	- decrease in medium- to coarse-grained	65	103.9	12.9
8	B2@7'				- medium dense, increase in silt content	49	112.7	20.8
10	B2@10'			ML	Sandy Silt, stiff, slightly moist, brown, fine- to medium-grained.	33	109.7	18.8
12	B2@12'				- some coarse-grained, increase in sand content	34	100.9	21.1
14	B2@15'			SM	Silty Sand, dense, slightly moist, light brown, fine- to medium-grained, some coarse-grained.	57	110.1	15.7

Total depth of boring: 15.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9733-06-01 BORING LOGS.GPJ

Figure A2,
Log of Boring 2, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

BORING 3

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0			**GRAVEL**			
			ARTIFICIAL FILL			
2	B3@2'		Silty Sand, loose, dry, brown, fine- to medium-grained, some coarse-grained, some gravel (to 1").	20	104.6	19.2
		ML	**ALLUVIUM**			
4			Sandy Silt, firm, slightly moist, dark brown, fine- to coarse-grained.			
	B3@5'		- stiff, increase in sand content	28	101.4	15.8
6			Silty Sand, medium dense, slightly moist, light brown, fine-grained, some medium-grained, trace gravel (to 0.5").			
	B3@7'			28	111.9	16.4
8		SM				
10	B3@10'		- some oxidation staining	39	109.5	20.1
12	B3@12'		Sandy Silt, stiff, slightly moist, olive brown, fine-grained, trace medium- to coarse-grained.	32	94.7	27.6
		ML				
14						
	B3@15'		- decrease in sand content	40	114.9	18.0

Total depth of boring: 15.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9733-06-01 BORING LOGS.GPJ

Figure A3,
Log of Boring 3, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

BORING 4

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**ASPHALT**			
					ARTIFICIAL FILL			
2	B4@2'			ML	Silty Sand, loose, dry, dark brown, fine- to medium-grained, some coarse-grained, some gravel (to 1").	18	104.0	16.2
					ALLUVIUM			
4					Sandy Silt, firm, slightly moist, brown, fine- to coarse-grained.			
					Silty Sand, loose, slightly moist, light yellowish brown, fine-grained.			
6	B4@5'			SM		14	112.4	19.8
	B4@7'				- some bedrock framents	14	100.8	23.7
8								
10					Sandy Silt, firm, slightly moist, dark brown, fine- to medium-grained.			
	B4@10'					17	103.7	24.5
12	B4@12'			ML	- soft, moist, some coarse-grained	8	98.7	26.7
14								
	B4@15'				- firm, brown to yellowish brown, fine- to coarse-grained	13	97.1	26.6

Total depth of boring: 15.5 feet
Fill to 1 foot.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9733-06-01 BORING LOGS.GPJ

Figure A4,
Log of Boring 4, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. A9733-06-01

BORING 5

ELEV. (MSL.) -- DATE COMPLETED **3/6/18**

EQUIPMENT **HOLLOW STEM AUGER** BY: **SRH**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ASPHALT**			
					ARTIFICIAL FILL Silty Sand, loose, dry, brown, fine- to medium-grained, some coarse-grained, some gravel (to 1").			
2	B5@2'					12	99.9	21.0
					ALLUVIUM Sandy Silt, firm, slightly moist, light brown, fine- to coarse-grained.			
4								
	B5@5'				- yellowish brown, fine- to medium-grained	14	100.9	22.1
6				ML				
	B5@7'				- dark brown	18	110.5	16.5
8								
10	B5@10'				- soft, dark yellowish brown, trace coarse-grained	9	86.7	16.8
					Sand with Silt, well-graded, medium dense, slightly moist, brown to yellowish brown, fine- to coarse-grained, some gravel (to 0.5").			
12	B5@12'			SW-SM		27	119.0	10.3
14					**PUENTE FORMATION** Interbedded Siltstone and Sandstone, yellowish brown, fine- to medium-grained, thinly bedded, soft to moderately hard, highly weathered, oxidation.			
	B5@15'					53	100.1	23.7

Total depth of boring: 15.5 feet
Fill to 1.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.
Asphalt patched.

*Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer.

A9733-06-01 BORING LOGS.GPJ

Figure A5,
Log of Boring 5, Page 1 of 1

SAMPLE SYMBOLS

- ☐ ... SAMPLING UNSUCCESSFUL
- ◧ ... STANDARD PENETRATION TEST
- ■ ... DRIVE SAMPLE (UNDISTURBED)
- ⊠ ... DISTURBED OR BAG SAMPLE
- ◪ ... CHUNK SAMPLE
- ▼ ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

LOGS BY RMA GROUP, MARCH 30, 2018, JOB NO. 18-0056-01 (3 BORINGS)

(5 PAGES)

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-1

Date Drilled:	3-6-18	Drilling Equipment:	Bucket auger
Logged By:	KRD	Boring Hole Diameter:	24"
Location:	See Site Geologic Map	Drive Weights:	0-24' – 2,400 lbs, 25'-44' – 1,550 lbs
Elevation (ft):	356'	Drop:	12"

Depth (ft)	Sample Type	Blows (blows/ft)	Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
						SM		Fill and topsoil: Gray-brown silty fine to medium sand with trace gravel, moist, medium dense, fill 3' thick on east side of boring.
								Puente formation: Gray siltstone with thin interbedded sandstone, BN85W, 52SW
5	R	5		16.8	109.6			@ 6' - Gray silty sandstone @ 6½' - Sandstone bed, BN68W, 44SW
10								@ 9½' - BN88W, 43SW @12' – ½" Thick gypsum layer, BN79W, 45SW
15	R	9		18.3	104.0			@ 16' - BN78W, 56SW @18'-19' – ¼" Thick gypsum seam along bedding
20								@ 20' - Discontinuous gypsum seams, JN2E, 59NW Gypsum Filled Joint @ 20½' - Sandstone, BN71W, 60SW Gray silty sandstone, fine grained, dense, moderately friable @ 26' - Slightly darker gray, BN79W, 62SW
25	R	20		22.8	102.4			@ 28½' - Yellow sandstone, BN78W, 46SW

Sample Types:

R - Ring Sample | Bulk Sample | Groundwater | B – Bedding attitude

T - Tube Sample | S - SPT Sample | End of Boring | J – Joint attitude

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-1

Date Drilled:	3-6-18	Drilling Equipment:	Bucket auger
Logged By:	KRD	Boring Hole Diameter:	24"
Location:	See Site Geologic Map	Drive Weights:	0-24' – 2,400 lbs, 25'-44' – 1,550 lbs
Elevation (ft):	356'	Drop:	12"

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
35	R	30/ 10"		19.5	110.7			@31' – BN86W, 50SW @32½' – Gray sandy siltstone interbeds 4" thick, BN78W, 55SW @34'-35' – Layer of gray sandy siltstone @35½' – BN75W, 54SW @35½'- 36' – Layer of gray sandy siltstone @37' – Gray sandy siltstone layer 6" thick, BN78W, 64SW @38' – Sandy siltstone, thin yellow siltstone interbeds @40½' – Siltstone, BN75W, 51SW
40								
45	R	50/8"		19.7	101.3			Total depth 45.5' No groundwater Hole Backfilled
50								

Material Description

This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.

Sample Types:

R - Ring Sample | - Bulk Sample | - Groundwater

T - Tube Sample | S - SPT Sample | - End of Boring

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-2

Date Drilled:	3-6-18	Drilling Equipment:	Bucket auger
Logged By:	KRD	Boring Hole Diameter:	24"
Location:	See Site Geologic Map	Drive Weights:	0-24' – 2,400 lbs, 25'-44' – 1,550 lbs
Elevation (ft):	355'	Drop:	12"

Depth (ft)	Sample Type	Blows (blows/ft)	Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
						SM		Topsoil: Brown silty sand
								Puente formation (Tp): Yellow sandstone, gray siltstone layers about 1' thick.
								@2½' – Yellow sandstone, calcium carbonite staining on joints, BN85W, 46SW
								@4' – JN10E, 64NW
5								@6½' – Siltstone bed BN85W, 50SW
10	R	11		15.1	105.6			@10' – Siltstone bed BN80W, 49SW
								@11' – Increase in siltstone layers, 3" thick
								@13' – Sandstone, BN80W, 44SW, siltstone below, gray, sandy
15								@16' – Siltstone bed, BN82W, 43SW
								@19½' – Few roots in joints, JN60W vertical, BN70W, 49SW, JN1E, 80SE
20	R	12		19.4	98.2			
								@22' – Sandstone bed, some gypsum, BN72W, 53SW
								@23' – Gypsum layer, BN72W, 53SW
25								@27' – Thin gypsum bed, BN76W, 50SW, ¼" thick
								@27½' – Gypsum bed
								@29' – Gypsum bed

This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.

Sample Types:

R - Ring Sample | - Bulk Sample | - Groundwater | B – Bedding attitude

T - Tube Sample | S - SPT Sample | - End of Boring | J – Joint attitude

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-2

Date Drilled:	3-6-18	Drilling Equipment:	Bucket auger
Logged By:	KRD	Boring Hole Diameter:	24"
Location:	See Site Geologic Map	Drive Weights:	0-24' – 2,400 lbs, 25'-44' – 1,550 lbs
Elevation (ft):	355'	Drop:	12"

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
35	R	10						@30' – BN80W, 54SW @31' – Small roots, gypsum bed BN70W, 51SW Total depth 35' No groundwater Hole Backfilled
40								
45								
50								

This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.

Sample Types:

R - Ring Sample | Bulk Sample - | Groundwater - | T - Tube Sample | S - SPT Sample | End of Boring -

RMA Group
Every Project Matters | www.rmacompanies.com

Exploratory Boring Log

Boring No. B-3

Date Drilled:	3-6-18	Drilling Equipment:	Bucket auger
Logged By:	KRD	Boring Hole Diameter:	24"
Location:	See Site Geologic Map	Drive Weights:	0-24' – 2,400 lbs, 25'-44' – 1,550 lbs
Elevation (ft):	348'	Drop:	12"

Depth (ft)	Samples: Sample Type	Samples: Blows (blows/ft)	Samples: Bulk Sample	Moisture Content (%)	Dry Density (pcf)	USCS	Graphic Symbol	Material Description
0						SM		Artificial fill (af): Brown silty fine to coarse sand with pieces of concrete and brick
5								Water line encountered and broken at 5'. Hole abandoned at 5' due to waterline No groundwater Hole Backfilled
10								
15								
20								
25								

This log contains factual information and interpretation of the subsurface conditions between the samples. The stratum indicated on this log represent the approximate boundary between earth units and the transition may be gradual. The log show subsurface conditions at the date and location indicated, and may not be representative of subsurface conditions at other locations and times.

Sample Types:

R - Ring Sample | Bulk Sample | Groundwater | B – Bedding attitude

T - Tube Sample | S - SPT Sample | End of Boring | J – Joint attitude

LOGS BY DAMES & MOORE, DECEMBER 13, 1993, JOB NO. 00499-038-015 (2 BORINGS)

(2 PAGES)

BORING B-1

APPROXIMATE SURFACE EL:+359 FEET (MSL)

DEPTH IN FEET	TESTS REPORTED ELSEWHERE	LABORATORY TEST DATA: ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	STRENGTH TEST DATA: TYPE OF TEST	STRENGTH TEST DATA: NORMAL OR CONFINING PRESSURE (PSF)	STRENGTH TEST DATA: SHEAR STRENGTH (PSF)	STRENGTH TEST DATA: DEVIATOR STRESS (PSF)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	BLOWS/FOOT SAMPLE TYPE	SYMBOL	DESCRIPTION
0								26		Bk	CL	8-inch thick asphaltic concrete pavement Brownish-yellow to brown silty fine sandy CLAY (stiff) [Possible Fill]
5		51	24					23	106	73 /7"		PUENTE FORMATION: Light gray siltstone with some dark brown staining, moderately weathered
10								22		75		Grades to include thin sandy interbeds
15								22		67		
20								22		61		
25										50 /5"		Grades (slightly weathered) Grades gray with occasional zones of silty sand and dark brown staining
30										50 /5"		
35	CORR									50 /4"		Grades dark gray
40										50 /4"		Boring completed to a depth of 40 feet on March 1, 1993.
45												
50												
55												
60												
65												

LOG OF BORING

LAC + USC MEDICAL CENTER REPLACEMENT PROJECT

LOS ANGELES, CALIFORNIA

For County of Los Angeles Internal Services Department

Dames & Moore

Figure A-3

GEO3FIGR USC38

BORING B-26

APPROXIMATE SURFACE EL:+375 FEET (MSL)

DEPTH IN FEET	LABORATORY TEST DATA: TESTS REPORTED ELSEWHERE	ATTERBERG LIMITS: LIQUID LIMIT (%)	ATTERBERG LIMITS: PLASTICITY INDEX (%)	STRENGTH TEST DATA: TYPE OF TEST	STRENGTH TEST DATA: NORMAL OR CONFINING PRESSURE (PSF)	STRENGTH TEST DATA: SHEAR STRENGTH (PSF)	STRENGTH TEST DATA: DEVIATOR STRESS (PSF)	MOISTURE CONTENT (%)	DRY DENSITY (PCF)	BLOWS/FOOT SAMPLE TYPE	SYMBOL	DESCRIPTION
0											SM	6-inch thick asphaltic concrete pavement
												Brown to tan silty fine SAND with trace gravel (dense) [Fill]
5								12	104	68 ■		Grades reddish-brown with medium to coarse sand (dense to very dense)
10	-200 (48)									41 ◪		PUENTE FORMATION: Reddish-brown to brownish-yellow sandstone, slightly weathered
15										78 /8" □		
20										80 /7" ◪		
25										50 /1" ◪		
30										50 /5" ◪		
35												
40										50 /4" ◪		Grades olive-brown to brown, slightly moist
45										50 /5" ◪		Grades with more silt and occasional zones of dark brown iron staining, wet
50										50 /5" ◪		Grades with light gray fine sand, bedding dips 45 degrees
55										50 /5" ◪		
60										50 /5" ◪		Boring completed to a depth of 60 feet on March 1, 1993.
65												

LOG OF BORING

LAC + USC MEDICAL CENTER REPLACEMENT PROJECT

LOS ANGELES, CALIFORNIA

For County of Los Angeles Internal Services Department

Dames & Moore

Figure A-13

GEO3FIGR USC38

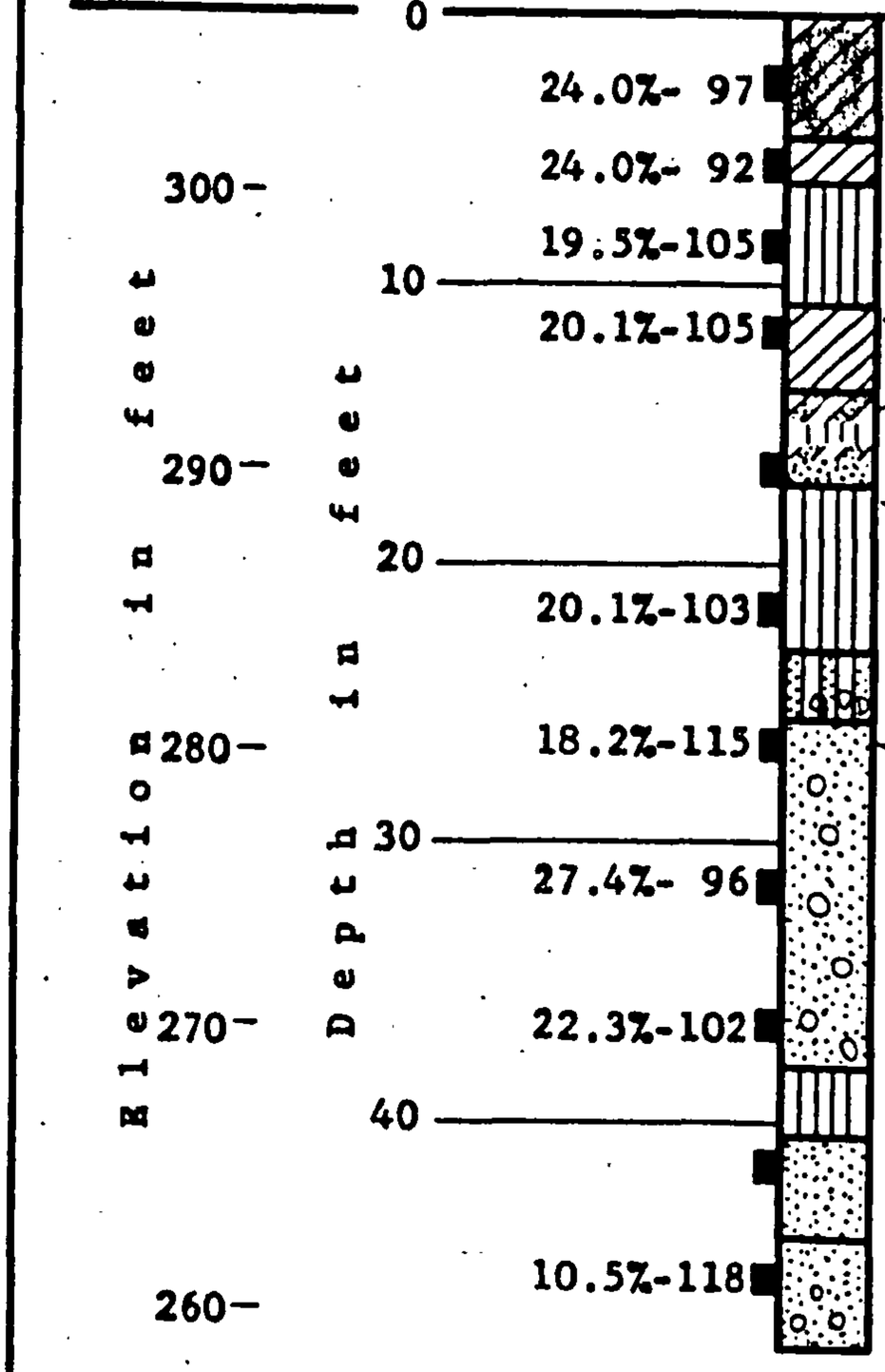
LOGS BY LEROY CRANDALL & ASSOC., SEPTEMBER 30, 1960, JOB NO. 60383 (11 BORINGS)

(9 PAGES)

JOB 60383 0 3 3 0 0 DATE 9-8-60 DRILLED BY C.M.C. SW

LOG OF BORING 1
18"-Diameter Rotary Bucket Hole
Drilled August 8, 1960

Elevation 306.6*



FILL - LEAN CLAY - brown

LEAN CLAY - contains large amount of silt, brown

SILT - contains large amount of clay, greyish-brown

Less clay, small amount of fine sand, greenish-brown

LEAN CLAY - contains large amount of silt, greenish-brown

CLAYEY SAND - sand fine, small amount of clay, greyish-brown

Lenses of POORLY GRADED SAND and SILT

SILT - contains small amount of clay and fine sand, greyish-brown

Large amount of fine sand

SILTY SAND - sand fine, medium amount of silt, bluish-grey

Large amount of gravel, few cobbles

POORLY GRADED SAND - fine to medium, large amount of fines, large amount of gravel, bluish-grey

Large amount of cobbles up to 8"

Patches of silt

Small amount of gravel and cobbles

(CHOPPING BUCKET USED AT 32½')

SILT - large amount of fine sand, bluish-grey

POORLY GRADED SAND - fine, small amount of fines, bluish-grey

Few gravel and cobbles

WELL GRADED SAND - large amount of small gravel, few cobbles, bluish-grey

NOTE: Groundwater seepage encountered at 26'; caving from 26'. to 29'.

KEY:

10.5%-118 ——— Indicates depth at which undisturbed sample obtained

Dry density in pounds per cubic foot

Field moisture content in percent of dry weight

Soils classified in accordance with the Unified Soil Classification System.

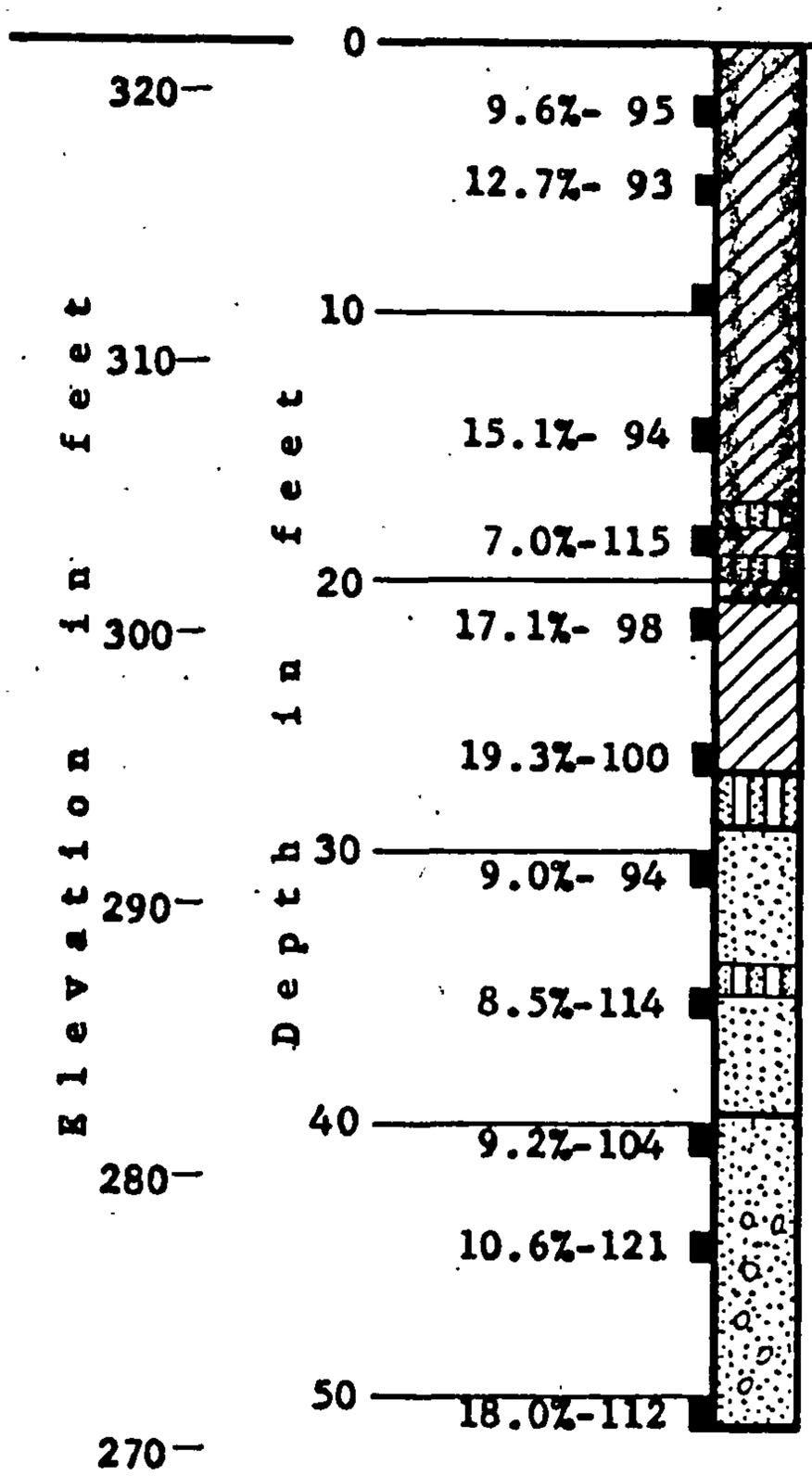
* Elevations refer to datum of reference drawing.

LEROY CRANDALL & ASSOCIATES

PLATE A-1A

JOB 60383 BY GMC
DATE 9-6-60

LOG OF BORING 2
18"-Diameter Rotary Bucket Hole
Drilled August 10, 1960



FILL - LEAN CLAY - brown
Pieces of concrete
Brown and reddish-brown mixed

Some shale

Large amount of shale, mottled yellowish-brown
Reddish-brown

Layers of SILTY SAND

LEAN CLAY - contains large amount of silt and fine sand, dark brown

SILTY SAND - sand fine, large amount of silt, small amount of clay, greyish-brown

POORLY GRADED SAND - fine, small amount of fines, yellowish-brown
Few gravel
Layer of SILTY SAND - light brown
Slightly cemented, reddish-brown
Sand fine to medium, small amount of gravel, bluish-grey

WELL GRADED SAND - small amount of fines, few small gravel, bluish-grey
Large amount of fines
Large amount of small gravel
Small amount of fines

NOTE: Groundwater encountered at 44'; small amount of caving from 44' to 50'.

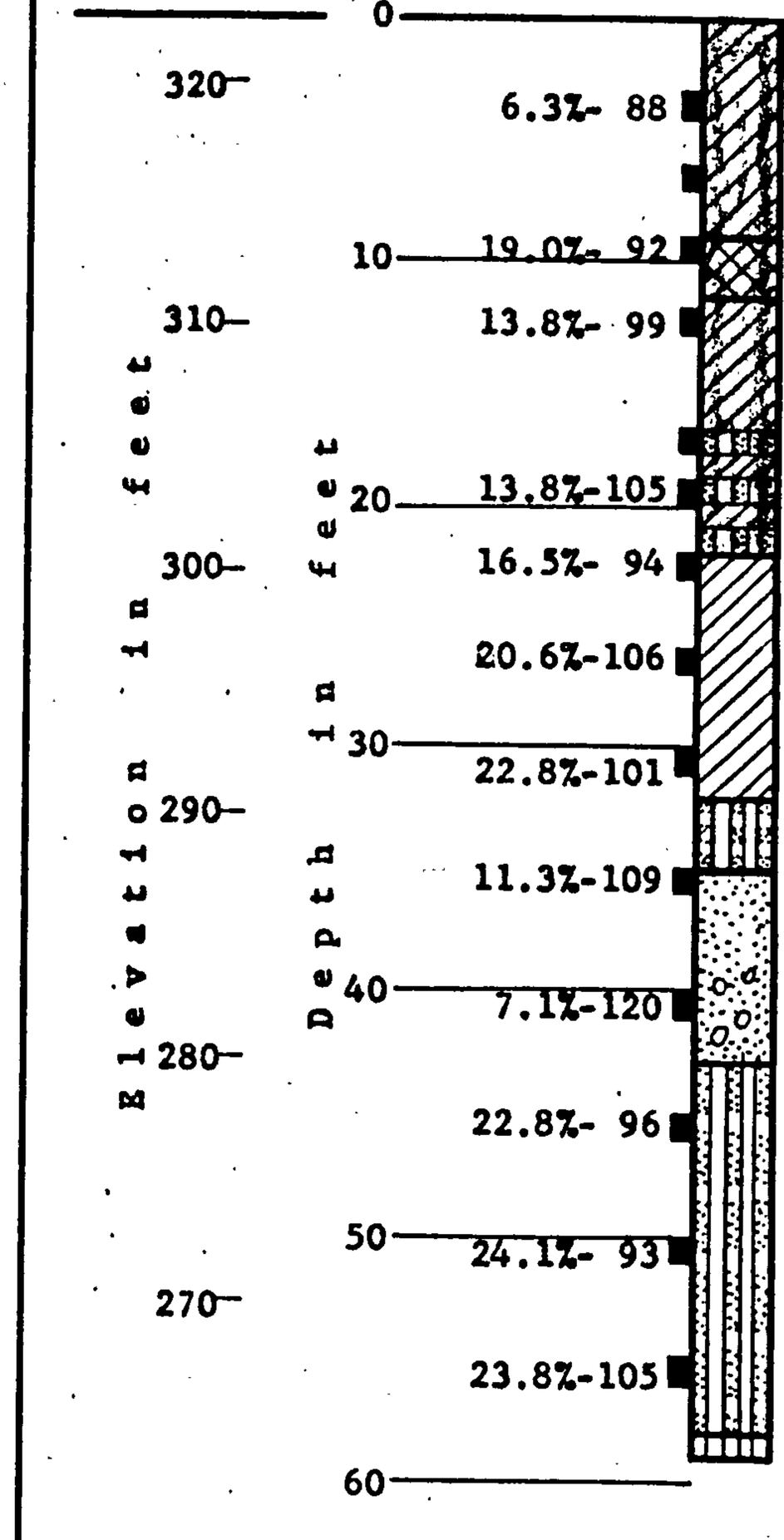
LEROY CRANDALL & ASSOCIATES

PLATE A-1B

JOB 60983 DATE 9-6-60 BY GMC SW

0 3 3 0 0 4

LOG OF BORING 3
18"-Diameter Rotary Bucket Hole
Drilled August 10, 1960



Elevation 322.8

FILL - LEAN CLAY - contains large amount of silt, brown
Pieces of concrete, some gravel
Large amount of fine sand

FILL - SHALE - yellowish-brown

FILL - LEAN CLAY - reddish-brown

Layers of SILTY SAND

Large amount of silt

LEAN CLAY - contains large amount of silt, dark brown
Contains small amount of silt, light greyish-brown
Contains large amount of sand

SILTY SAND - sand fine, small amount of silt, greyish-brown
Lenses of SILT

WELL GRADED SAND - contains patches of clay and poorly graded sand with large amount of fines, greyish-brown
(CHOPPING BUCKET USED AT 37½')
Large amount of gravel and small cobbles
Few cobbles up to 8"

SILTY SAND - sand fine, large amount of silt, light brown
Lenses of SILT
Dark bluish-grey

SILT - contains medium amount of fine sand and small amount of clay, brownish-grey

NOTE: Groundwater encountered at 39'; no caving.

LEROY CRANDALL & ASSOCIATES

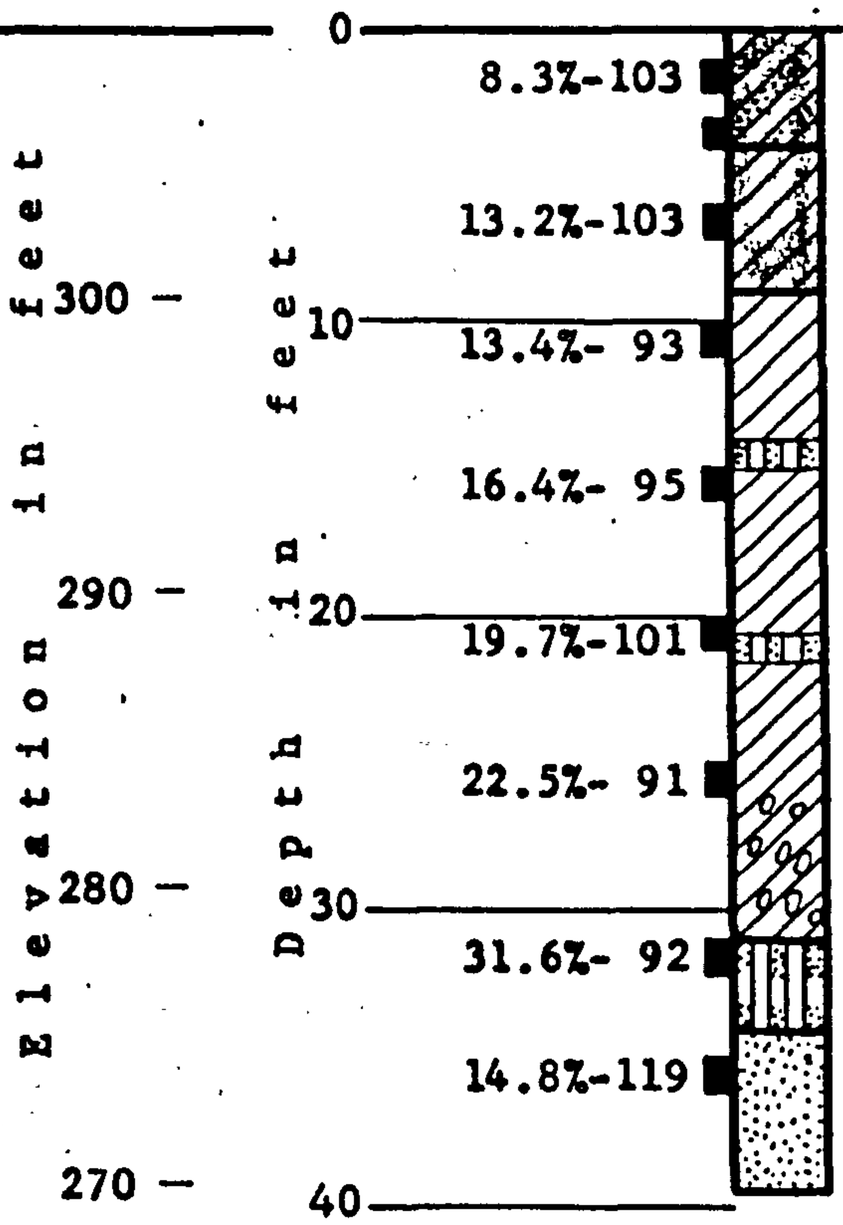
PLATE A-1C

JOB 60383 DATE 9-6-60 BY GMC Scl

0 3 3 0 0 4 0 0 0 9 0

LOG OF BORING 4
18"-Diameter Rotary Bucket Hole
Drilled August 9, 1960

Elevation 309.1



FILL - CLAYEY SAND - sand fine, large amount of clay, brown
Pieces of brick and concrete
FILL - LEAN CLAY - reddish-brown
Patches of brown

LEAN CLAY - contains small amount of silt, dark brown
Streaks of alkali
Contains large amount of silt, light brown
Layer of SILTY SAND - light brown

Layer of SILTY SAND - light brown
Contains large amount of fine sand
Contains large amount of silt, small amount of sand
(CHOPPING BUCKET USED AT 27½')
Contains large amount of gravel and cobbles
SILTY SAND - sand fine, large amount of silt, bluish-grey
Lenses of SILT - bluish-grey
WELL GRADED SAND - contains medium amount of fines, small amount of gravel and small cobbles, bluish-grey

NOTE: Groundwater encountered at 30'; caving badly below 35'.

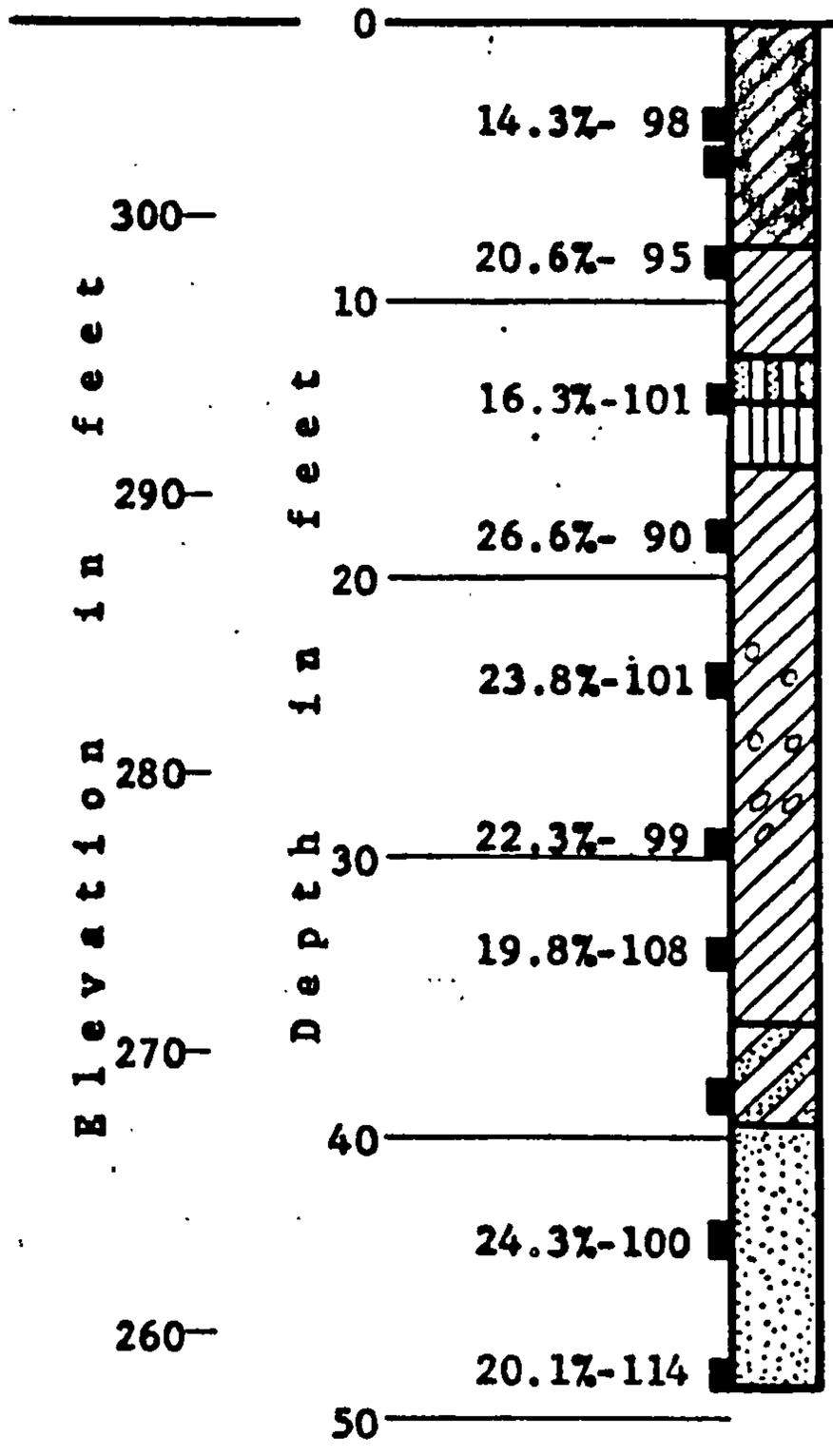
LEROY CRANDALL & ASSOCIATES

PLATE A-1D

JOB 60383 DATE 9-60 BY GMC Sec

0 3 3 0 0 4 0 9 1

LOG OF BORING 5
18"-Diameter Rotary Bucket Hole
Drilled August 8, 1960



Elevation 307.0

FILL - LEAN CLAY - brown
Pieces of concrete
Contains large amount of silt, small amount of fine sand, reddish-brown
Pieces of brick, dark brown

LEAN CLAY - contains large amount of silt, mottled brown

SILTY SAND - sand fine, medium amount of silt, light brown

SILT - contains small amount of clay, brown

LEAN CLAY - contains medium amount of silt, brown
Few small gravel
Contains large amount of fine sand, dark greyish-brown
Small amount of gravel, few cobbles
Large amount of gravel and cobbles up to 8", bluish grey

CLAYEY SAND - fine, large amount of clay, few gravel, bluish-grey

POORLY GRADED SAND - fine, large amount of fines, bluish-grey

Patches of silt

NOTE: Groundwater seepage encountered at 27'; small amount of caving at 27'.

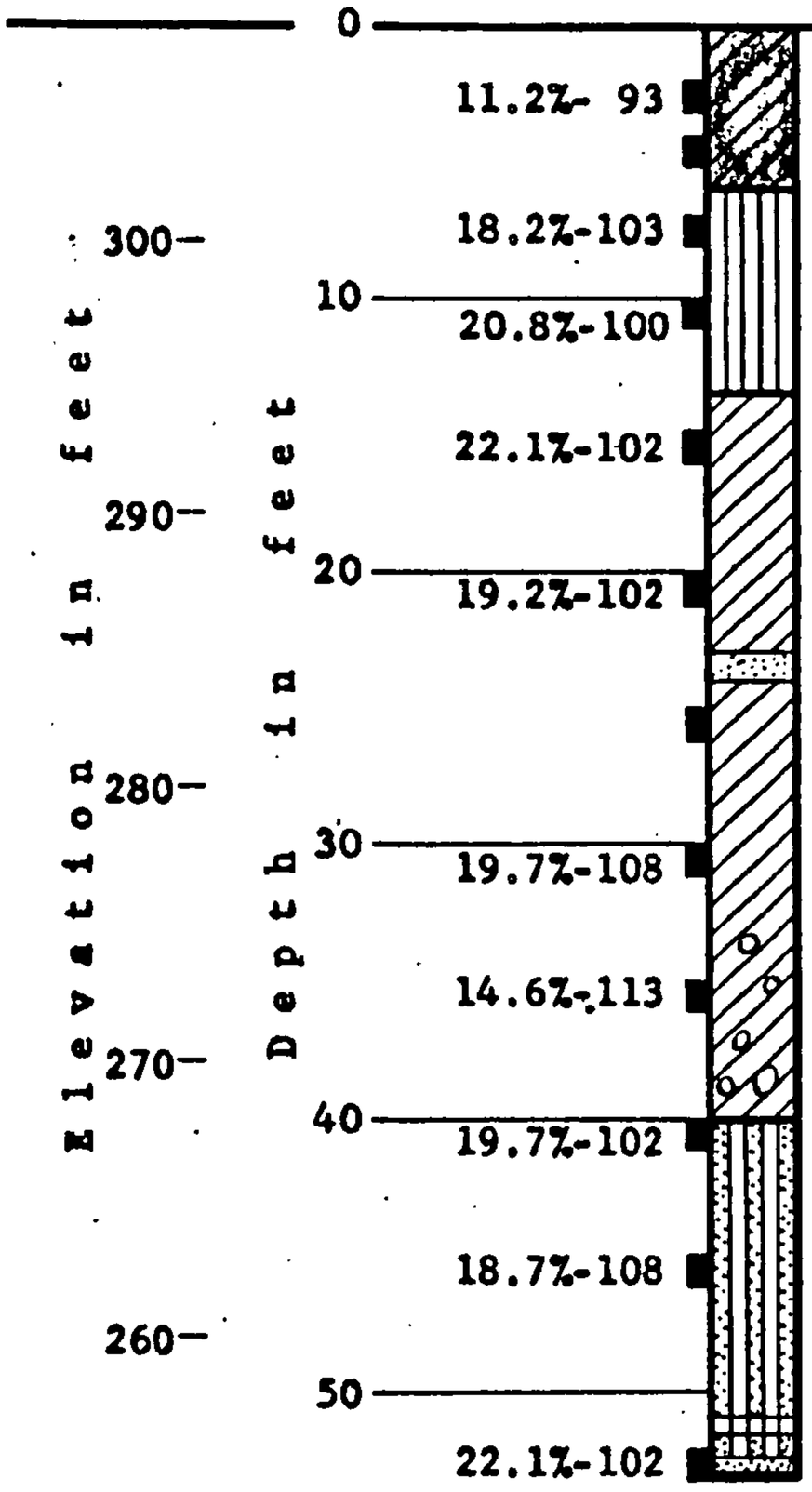
LEROY CRANDALL & ASSOCIATES

PLATE A-12

JOB 60389 DATE 9-6-60 BY GHC (Sh.)
0 3 3 0 0 4 0 2

LOG OF BORING 6
18"-Diameter Rotary Bucket Hole
Drilled August 9, 1960

Elevation 307.9



FILL - LEAN CLAY - large amount of silt, reddish-brown

SILT - contains small amount of fine sand, brown
Contains small amount of clay

LEAN CLAY - contains large amount of silt, brown

Contains small amount of fine sand

Layer of WELL GRADED SAND - brown

Occasional gravel

Bluish-grey

Occasional small cobbles

SILTY SAND - sand well graded, large amount of silt, bluish-grey
Patches of silt
Sand fine, medium amount of silt, brown

Layers of POORLY GRADED SAND and SILT

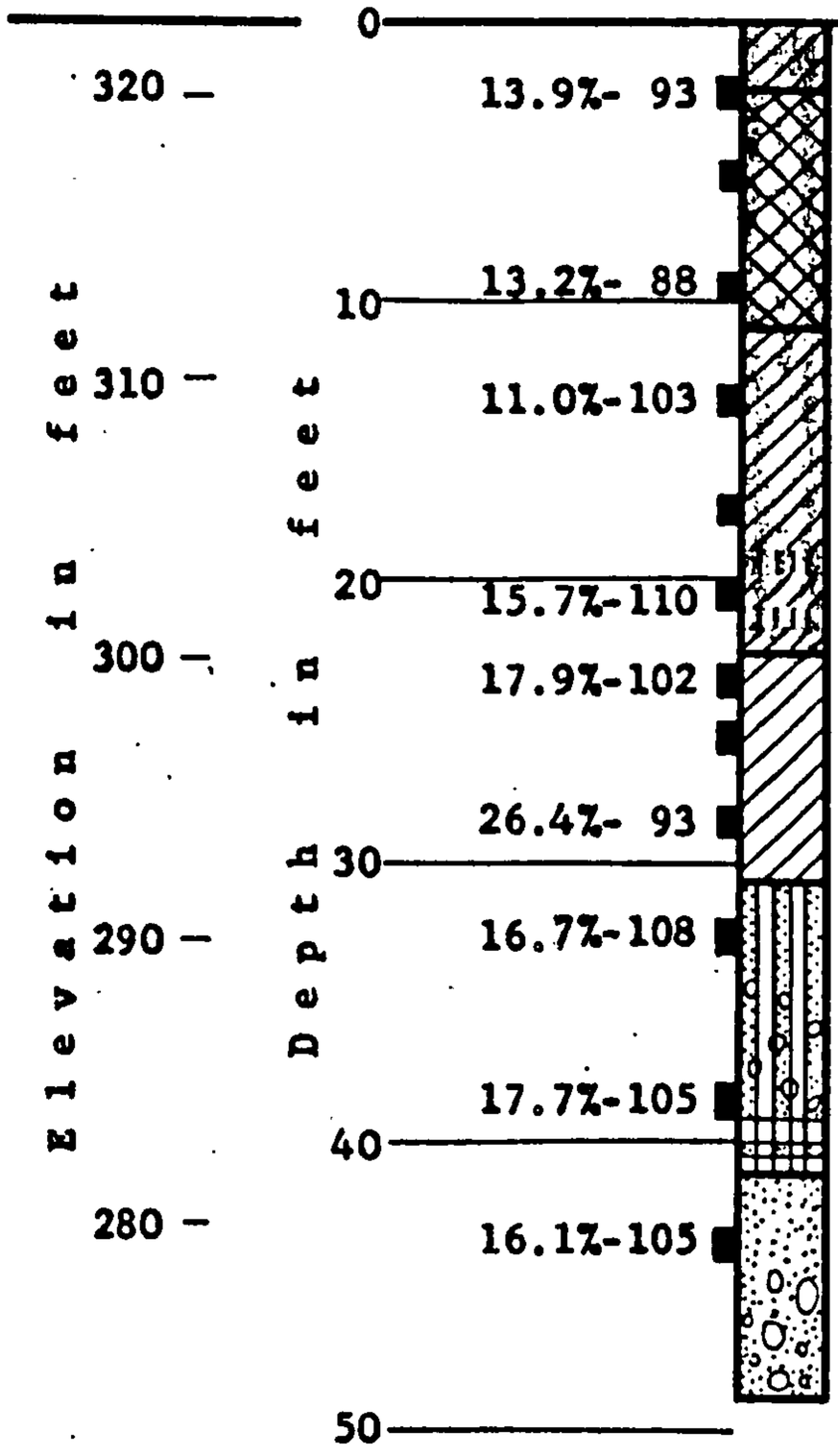
NOTE: Groundwater seepage encountered at 23'; no caving.

LEROY CRANDALL & ASSOCIATES

PLATE A-1F

JOB 60383 DATE 9.6.60 BY G.M.C. Sht 3

LOG OF BORING 7
18"-Diameter Rotary Bucket Hole
Drilled August 10, 1960



Elevation 322.8

FILL - LEAN CLAY - brown
FILL - SHALE - yellowish-brown

FILL - LEAN CLAY - reddish-brown

Lenses of SILTY SAND and SILT - greyish-brown

LEAN CLAY - contains medium amount of silt, mottled brown
Large amount of fine sand

SILTY SAND - sand fine, large amount of silt, small amount of clay, light greyish-brown
Large amount of gravel and cobbles
(CHOPPING BUCKET USED AT 37')
Layers of SILT and SILTY SAND

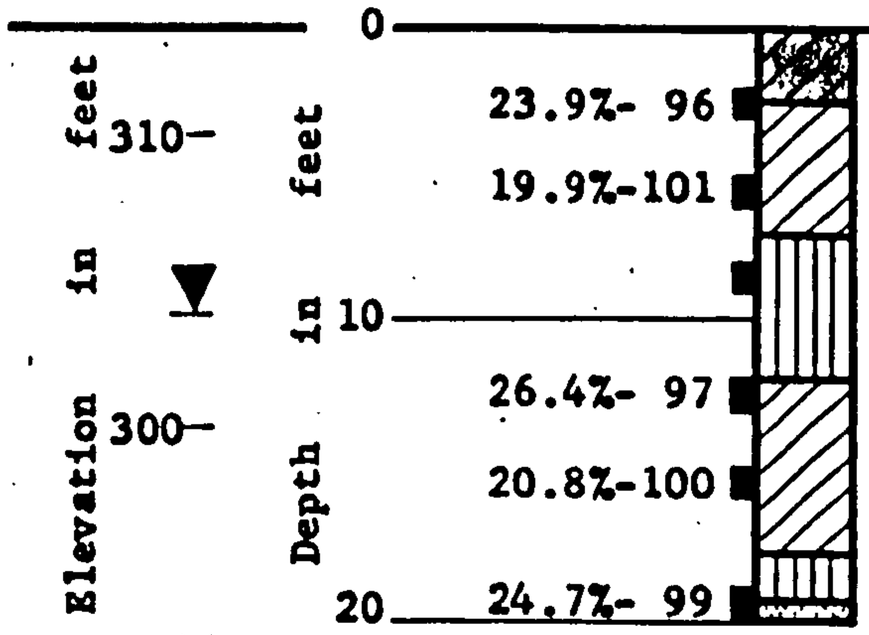
POORLY GRADED SAND - fine, large amount of fines, bluish-grey
Light brown
Few small gravel and small cobbles

NOTE: Groundwater encountered at 40'; small amount of caving from 46' to 48'.

JOB 60383 DATE 9-6-60 BY GMC SRL

0 3 3 0 0 4 1 0 0 0 9 4

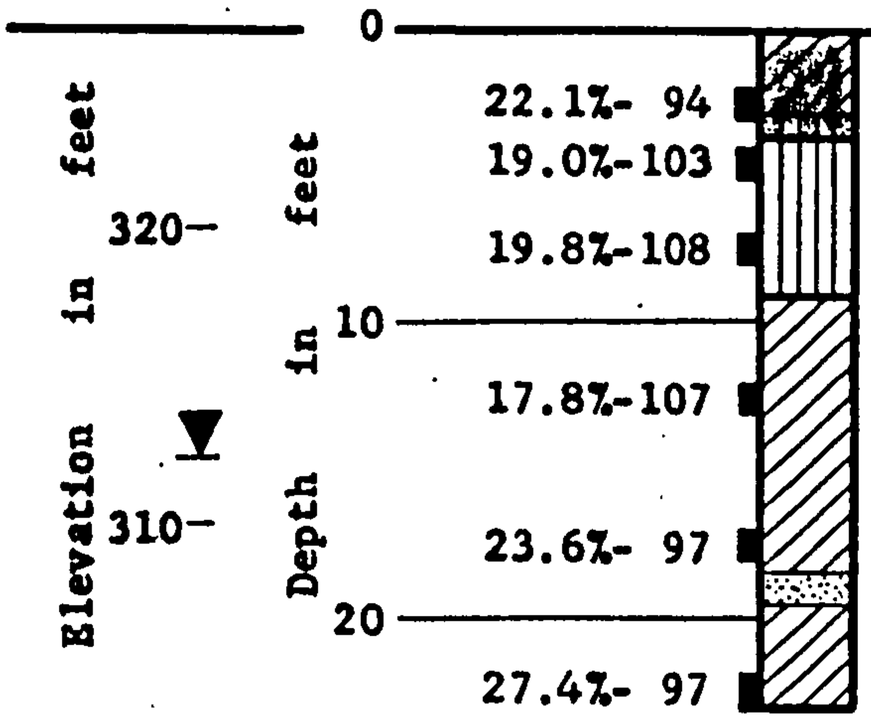
LOG OF BORING 8
18"-Diameter Rotary Bucket Hole
Drilled August 12, 1960



FILL - LEAN CLAY - mottled brown
LEAN CLAY - contains large amount of silt, dark brown
Brown
SILT - contains large amount of clay, brown
LEAN CLAY - contains small amount of silt and fine sand, mottled brown
SILT - contains large amount of clay, mottled brown
POORLY GRADED SAND - fine, clean, brown

NOTE: Groundwater seepage encountered at 16½'; no caving.

LOG OF BORING 9
18"-Diameter Rotary Bucket Hole
Drilled August 12, 1960



FILL - LEAN CLAY - brown
Layer of FILL - SILTY SAND - brown
SILT - contains small amount of clay and fine sand, brown
Contains large amount of fine sand
LEAN CLAY - contains large amount of silt, brown
Contains small amount of silt
Layer of POORLY GRADED SAND - medium, brown

NOTE: Groundwater encountered at 18½'; small amount of caving from 18½' to 19½'.

▼ Indicates approximate invert elevation

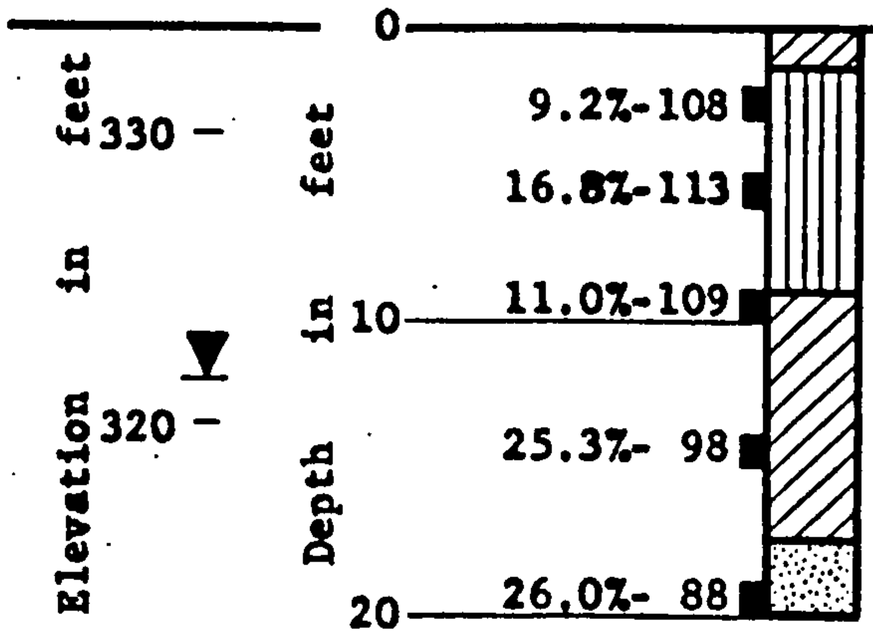
LEROY CRANDALL & ASSOCIATES

PLATE A-1H

JOB 60383 DATE 9-6-60 BY GMC Srv

0 3 3 0 0 4 0 0 0 9 5

LOG OF BORING 10
18"-Diameter Rotary Bucket Hole
Drilled August 12, 1960



LEAN CLAY - brown
SILT - contains medium amount of clay and fine sand, light brown

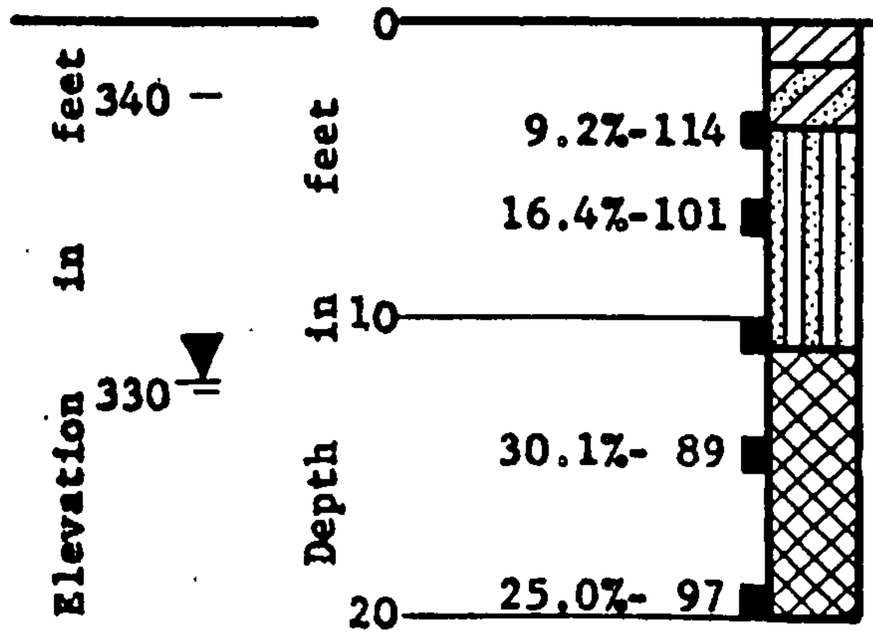
LEAN CLAY - contains large amount of silt, brown

Contains small amount of silt

POORLY GRADED SAND - fine, large amount of fines, brown

NOTE: Groundwater seepage encountered at 14'; no caving.

LOG OF BORING 11
18"-Diameter Rotary Bucket Hole
Drilled August 12, 1960



LEAN CLAY - contains small amount of fine sand, brown
CLAYEY SAND - fine to medium, small amount of clay, reddish-brown
Few gravel and small cobbles
SILTY SAND - sand fine, small amount of silt, light reddish-brown
Large amount of silt
SHALE - light brown

Mottled greyish-brown

NOTE: Groundwater encountered at 17'; no caving.

LEROY CRANDALL & ASSOCIATES

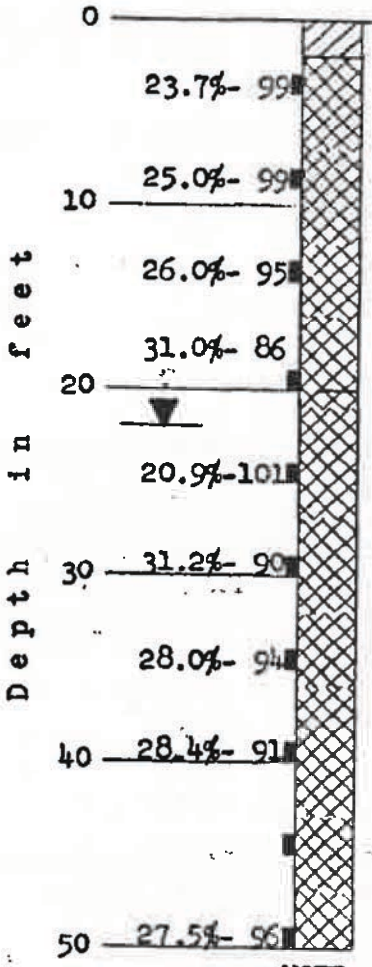
PLATE A-1I

LOGS BY LEROY CRANDALL & ASSOC., FEBRUARY 11, 1957, JOB NO. 56356 (7 BORINGS)

(7 PAGES)

JOB 56356 0330 DATE 12-29-56 BY F.S.

LOG OF BORING 1



NOTE: Water seepage encountered at 20'; no caving

KEY:

27.5%- 96 ——— Indicates depth at which undisturbed sample obtained

——— Dry density in lbs./cu.ft.

——— Field moisture content in percent of dry weight

Soils classified in accordance with the United States Bureau of Reclamation system.

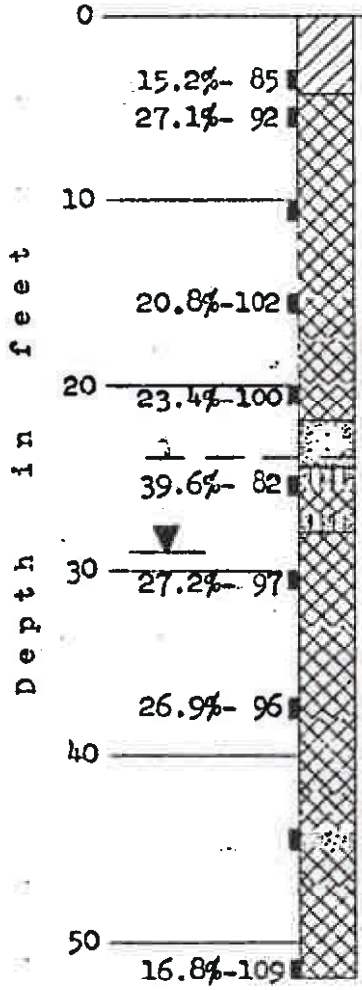
▼ Indicates recommended footing elevation.

LEROY CRANDALL & ASSOCIATES

PLATE A-1A

JOB 56356 DATE 12-22-56 BY F.S.

LOG OF BORING 2



LEAN CLAY - contains large amount of silt, porous, brown

SHALE - jointed, fractured, yellowish-brown

Less fractured

Lenses of sandstone

POORLY GRADED SAND - fine, contains small amount of silt, greyish-brown

Heavy water seepage encountered

SILTY SAND and SHALE - sand predominantly fine to very fine, contains large amount of silt, shale is very plastic

SHALE and SANDSTONE - mottled yellowish-brown

Lens of POORLY GRADED SAND - predominantly fine, moderately clean, brown

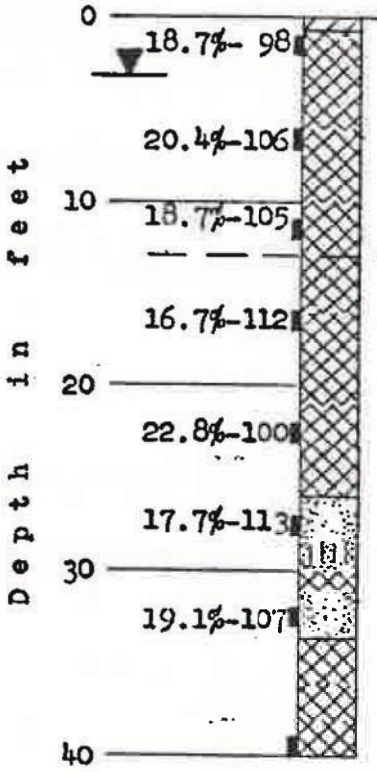
NOTE: Heavy water seepage encountered at 24'; no caving

LEROY CRANDALL & ASSOCIATES

PLATE A-1B

JOB 56356 DATE 12-29-56 BY RS

LOG OF BORING 3



Elevation 343

LEAN CLAY - brown

SANDSTONE - contains laminations of shale, yellowish-brown

Water seepage encountered

Layer of highly cemented sandstone Yellowish-brown and grey

Layers of shale

POORLY GRADED SAND - predominantly fine, contains trace of silt, brown

Layers of SILTY SAND, SHALE, and POORLY GRADED SAND

SANDSTONE and SHALE - yellowish-brown

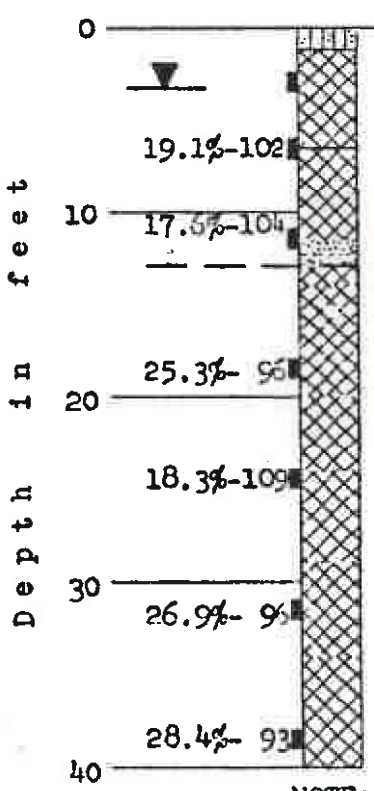
NOTE: Water seepage encountered at 13'; no caving

LEROY CRANDALL & ASSOCIATES

PLATE A-1C

JOB 56356-3-0 DATE 12-8-56 BY FS

LOG OF BORING 4



FILL - SILTY SAND and CLAY - Brown
SHALE - jointed, fractured, yellowish-brown

SANDSTONE - yellowish-brown and grey

Layer of POORLY GRADED SAND - fine
Water seepage encountered
Lenses of shale

Predominantly shale, layers of sandstone

Predominantly sandstone, layers of shale

Laminations of shale

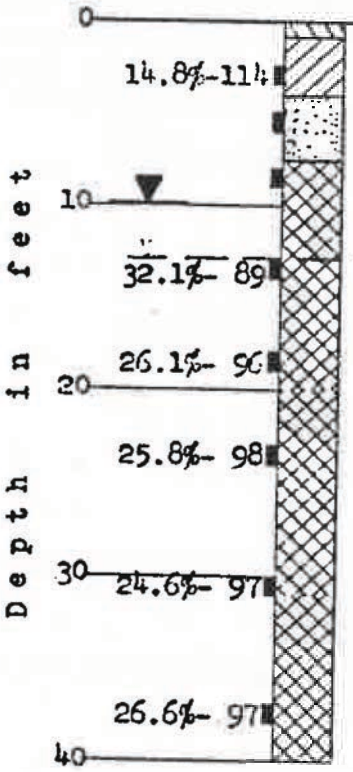
NOTE: Water seepage encountered at 13', water at 9' after 24 hours; no caving

LEROY CRANDALL & ASSOCIATES

PLATE A-1D

JOB 56855 DATE 12-74-20-0-BY FS
0 3 3 0 0 1 0

LOG OF BORING 5



FILL
LEAN CLAY - contains large amount of very fine sand, medium amount of silt, reddish-brown
WELL GRADED SAND - contains some silt, large amount of gravel and small cobbles, reddish-brown
SHALE - laminated, yellowish-brown and grey
— Water seepage encountered

NOTE: Water seepage encountered at 13'; no caving

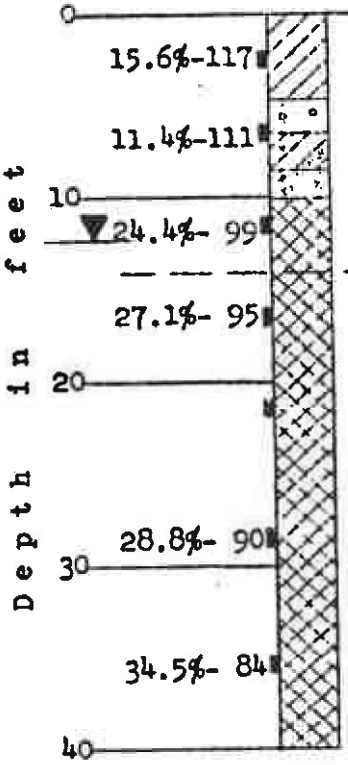
LEROY CRANDALL & ASSOCIATES

PLATE A-1E

JOB 56356 DATE 11/5/56 BY AHK

LOG OF BORING 6

Elevation 347



LEAN CLAY - dark brown to reddish-brown, contains medium amount of very fine sand

WELL GRADED SAND - few gravel, brown

CLAYEY SAND - large amount of clay, reddish-brown

WELL GRADED SAND - few gravel, brown

SHALE - laminated, yellowish-brown

Water seepage encountered

Yellowish-brown and grey

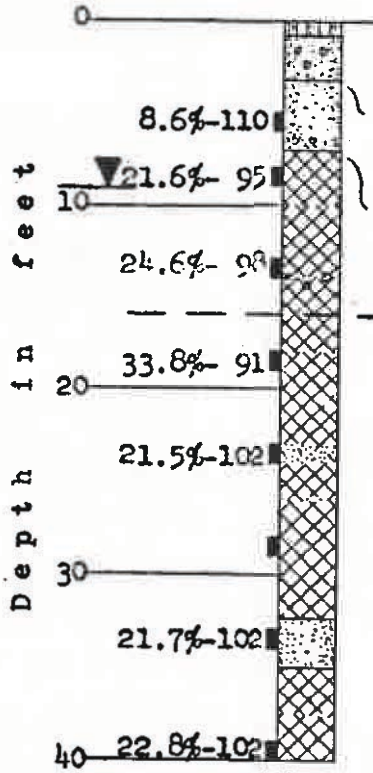
NOTE: Water seepage encountered at 14'; no caving

LEROY CRANDALL & ASSOCIATES

PLATE A-1F

JOB 56356 DATE 12-29-56 BY FS

LOG OF BORING 7



SILTY SAND - fine, large amount of silt

WELL GRADED SAND - contains some silt, few gravel, yellowish-brown

POORLY GRADED SAND - predominantly fine to medium, contains some silt, reddish-brown
Cemented; few small cobbles

SHALE and SANDSTONE - yellowish-brown and grey

Few highly cemented shale cobbles

— — — Water seepage encountered

Lens of POORLY GRADED SAND - fine
Predominantly silty sand, silt, and layers of firm to very firm shale, yellowish-brown and grey
Sandstone

Layer of POORLY GRADED SAND - fine, some silt, brown

NOTE: Water seepage encountered at 16'; no caving

LEROY CRANDALL & ASSOCIATES

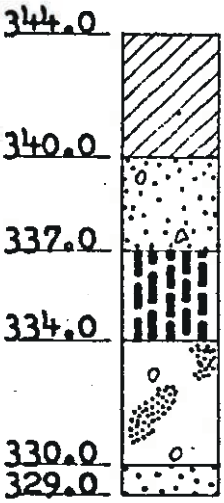
PLATE A-10

LOGS BY L.T. EVANS, SEPTEMBER 8, 1950

(7 TEST HOLES)

(7 PAGES)

LOG OF TEST HOLE NO. 1



Dark brown clayey sand fill

Dark brown disintegrated granite fill

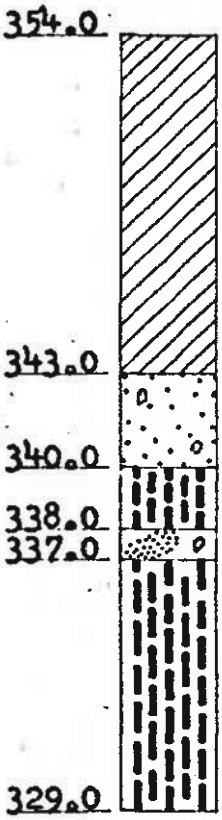
Grayish tan siltstone fill

Grayish brown fine sand to medium gravel fill

Light gray fine to medium sand fill

0330040001

LOG OF TEST HOLE NO. 2



Dark brown clayey sand fill

Dark brown disintegrated granite fill

Grayish tan siltstone fill

Grayish brown fine sand to medium gravel fill

Tannish gray siltstone

0330041012

LOG OF TEST HOLE NO. 3



Dark brown clayey sand fill

Brown fine to medium sand-Trace of clay fill

Dark brown disintegrated granite fill

Light brown medium sand fill

Grayish brown fine sand to medium gravel fill

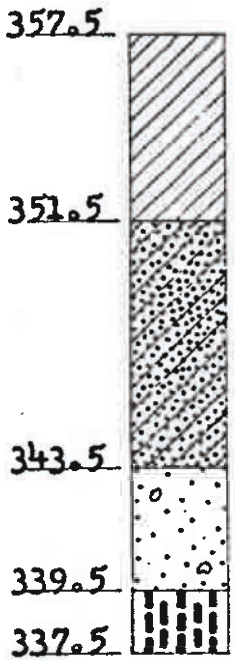
Tannish gray siltstone

Dark gray siltstone

0330040013

3

LOG OF TEST HOLE NO. 4



Dark brown clayey sand fill

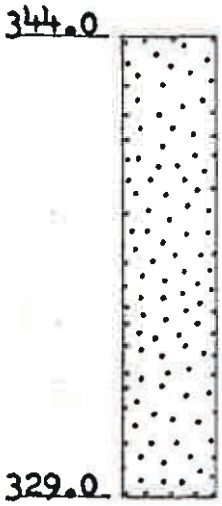
Rust brown clayey fine sand fill

Dark brown disintegrated granite fill

Tannish gray siltstone

0330040001-4

LOG OF TEST HOLE NO. 5



Sandy fill

0330040011 5

LOG OF TEST HOLE NO. 6



Dark brown clayey sand fill

Sand and gravel fill

Light grayish brown medium sand

Tannish gray siltstone

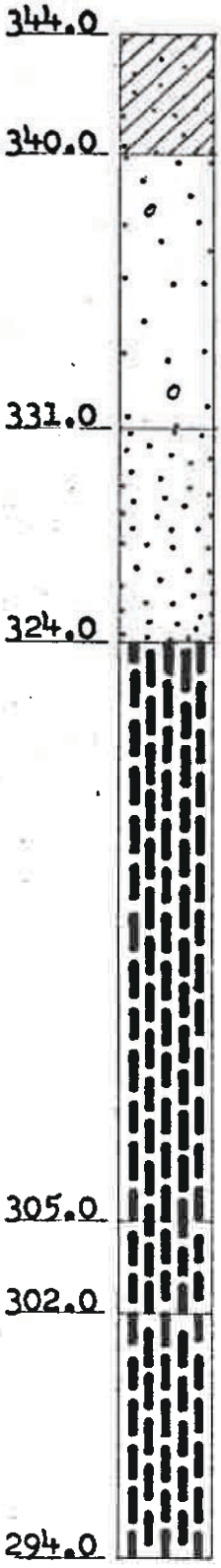
Gray siltstone

Dark gray siltstone

0330040011 6

6

LOG OF TEST HOLE NO. 7



Dark brown clayey sand fill

Sand and gravel fill

Light grayish brown medium sand

Tannish gray siltstone

Gray siltstone

Dark gray siltstone

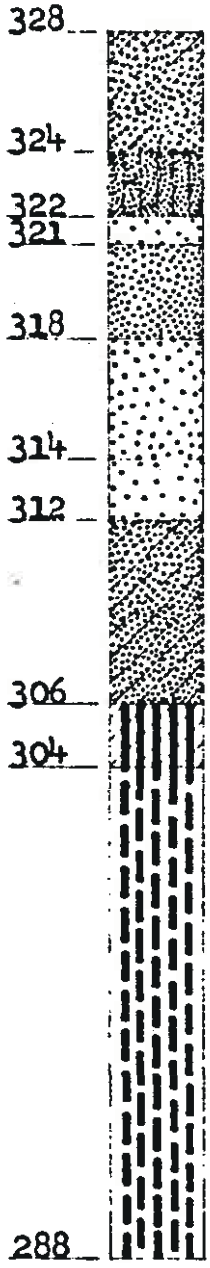
0330040011 7

7

LOGS L.T. EVANS, AUGUST 2, 1950 (6 TEST HOLES)

(6 PAGES)

LOG OF TEST HOLE NO. 2



Rust brown clayey very fine to fine sand

Brown silty very fine sand-Trace of clay

Rust brown fine to medium sand-Few roots

Light brown fine sand

Brown fine to medium sand-Trace of clay

Brown fine to medium sand

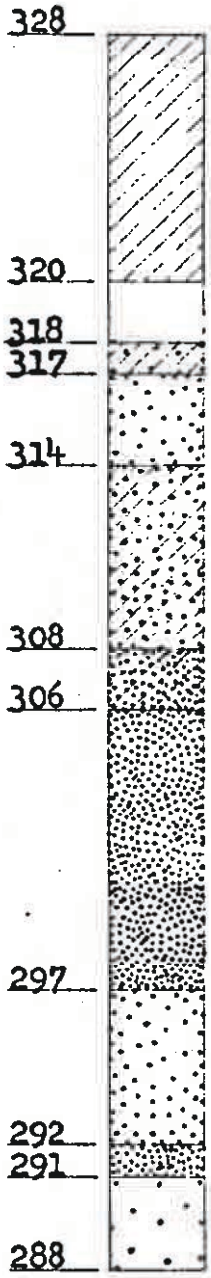
Brown clayey fine sand

Brown clayey silt

Tannish gray siltstone

0330040139

LOG OF TEST HOLE NO. 3



Clayey fill

Dark brownish gray top soil

Dark brown clayey medium to coarse sand

Grayish brown medium sand-Trace of clay

Rust brown clayey fine to medium sand

Brownish gray clayey fine sand

Brown fine sand-Trace of clay

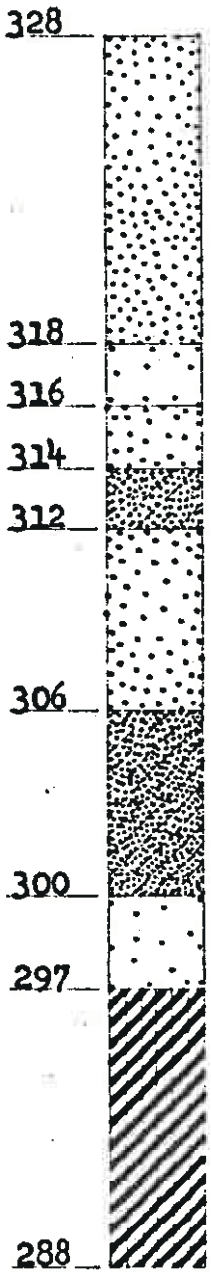
Brown medium sand

Brownish gray fine sand

Brownish gray coarse sand

0330040'0'0140

LOG OF TEST HOLE NO. 4



Rust brown fine to medium sand-Few roots

Dark brown clayey medium to coarse sand

Rust brown fine to medium sand-Trace of clay

Brown clayey fine sand

Brown fine to medium sand-Trace of clay

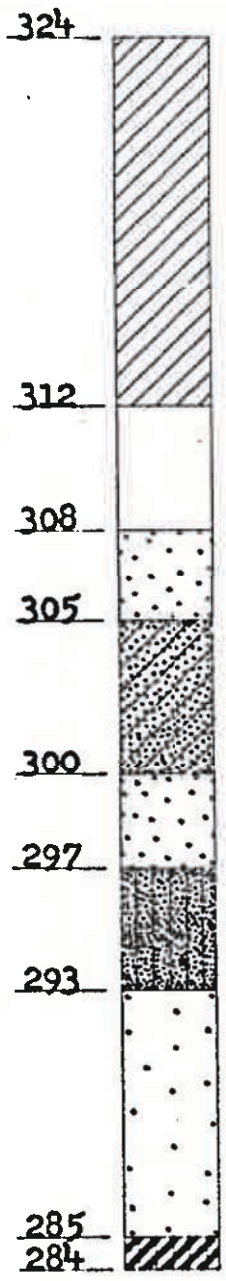
Brown clayey very fine sand

Brownish gray coarse sand

Brown silty clay

0330040041

LOG OF TEST HOLE NO. 5



Clayey fill

Dark brownish gray top soil

Brown medium sand

Brown clayey very fine to fine sand

Brown medium sand

Brownish gray silty very fine sand

Brownish gray medium to coarse sand

Brown silty clay

0330040012

0 3 3 0 0 4 0 0 1 4 3

LOG OF TEST HOLE NO. 6



Dark brownish gray top soil

Brown fine to medium sand-Trace of clay

Brown clayey very fine sand

Grayish brown clayey very fine sand

Light brownish gray coarse sand to medium gravel

Grayish brown silty clay

Dark gray clayey very fine sand

Dark gray fine to medium sand

LOGS BY L.T. EVANS, JULY 27, 1948 (5 TEST HOLES)

(5 PAGES)

LOG AND PROPERTIES

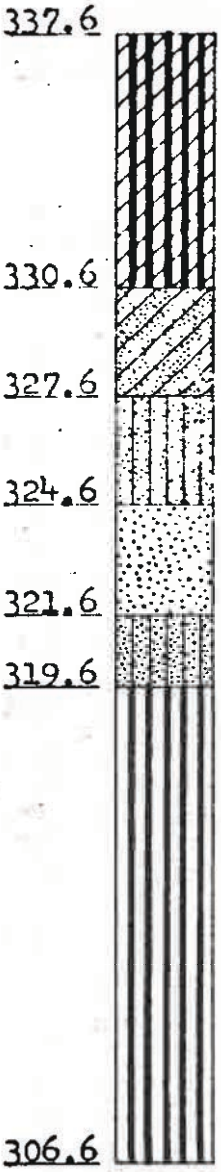
HOLE NO. 1



0330041 0167

LOG AND PROPERTIES

HOLE NO. 2



0330040168

LOG AND PROPERTIES

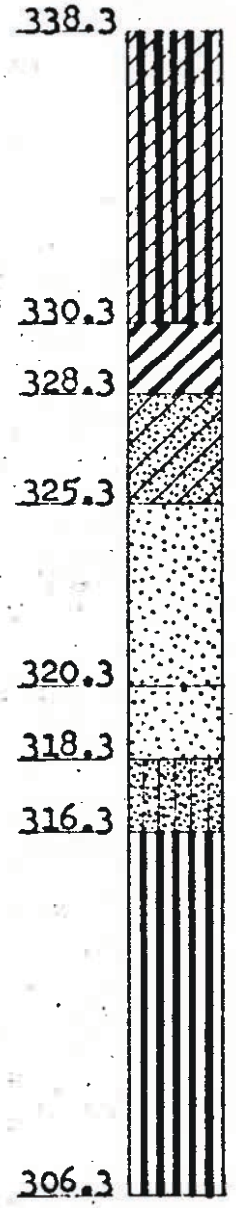
HOLE NO. 3



0330041069

LOG AND PROPERTIES

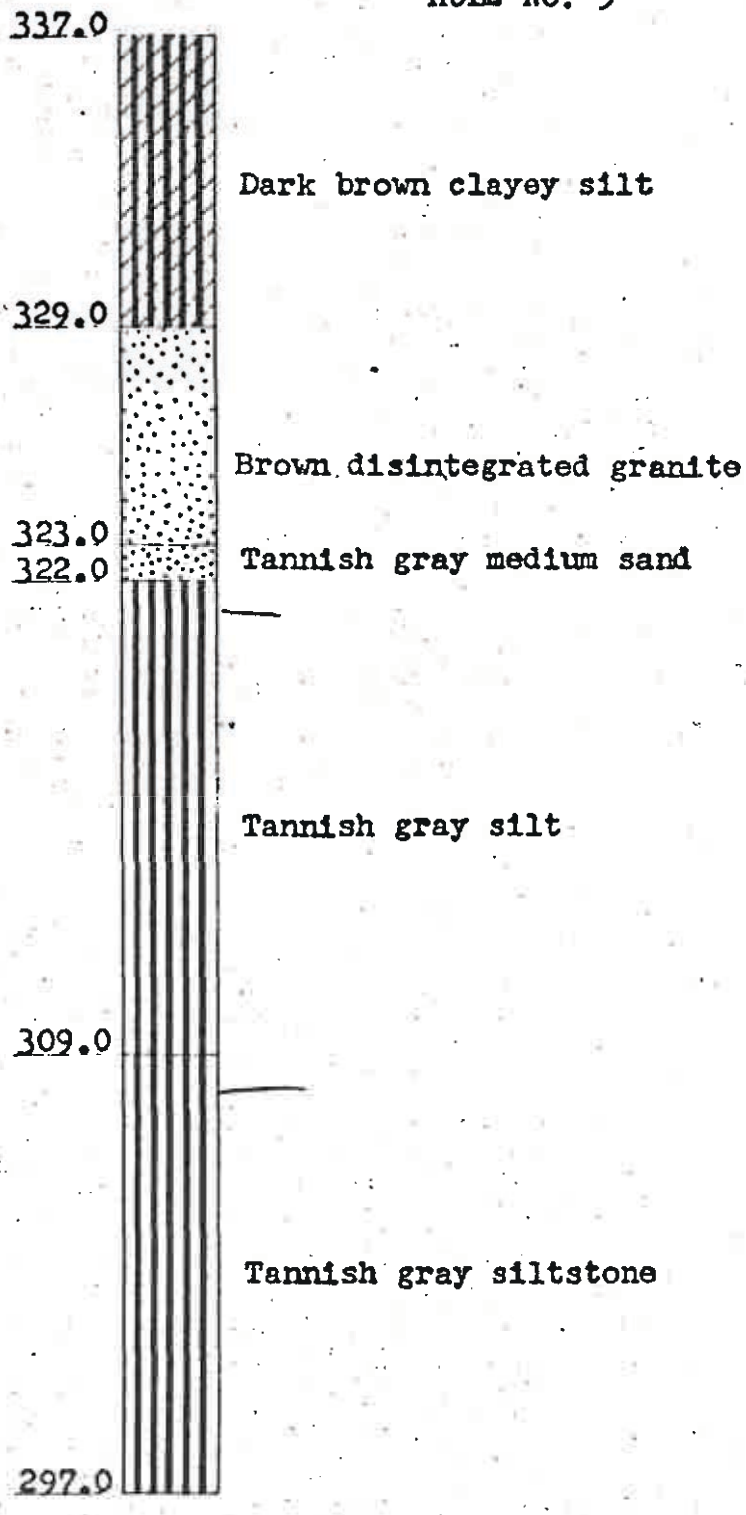
HOLE NO. 4



0330041017 0

LOG AND PROPERTIES

HOLE NO. 5



0330041171