APPENDIX C
Preliminary Geotechnical Investigation

SOILS ENGINEERING, INC.
Consulting Geotechnical Engineers

REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION

Proposed New Residential Development

1007 East Victoria Street

City of Carson, California

Prepared For:

Brandywine Homes

16580 Aston Street
Irvine, California 92606

Project No. 6816.18 **July 24, 2018**

2860 Walnut Avenue, Signal Hill CA 90755 * Tel: (562) 426-7990 * Fax: (562) 426-1842

SOILS ENGINEERING, INC.
Consulting Geotechnical Engineers

July 24, 2018
Project No. 6816.18

BRANDYWINE HOMES
16580 Aston Street
Irvine, California 92606

Attention: Mr. Matt Ashton

Subject: **Report of Preliminary Geotechnical Investigation**
Proposed New Residential Development
1007 East Victoria Street
City of Carson, California

Gentlemen:

Presented herewith is the Report of Preliminary Geotechnical Investigation (the Soils Report) prepared by Associated Soils Engineering, Inc. (ASE) for the proposed new residential development (the Buildings) planned for construction at the above subject address, in the City of Carson, California (the Site). This work was conducted in accordance with ASE's Proposal No. P18-080, dated June 7, 2018, and your subsequent authorization.

The subject geotechnical investigation was planned and performed based on the relevant development information provided by your office. Provided information included a Concept Civil Plan, Sheet 1 of 2, prepared by KES Technologies, Inc., date unknown, on which was shown planned building locations and access roadways within the new residential development.

The purpose of this study was to evaluate the subsurface soils conditions at the Site, followed by assessment of site geologic/seismic hazards, performance of engineering analyses, and formulation/assembly of recommendations for the geotechnical design and construction pertinent to the Buildings. ASE's study has concluded that construction of the Buildings is geotechnically feasible provided that the recommendations and design guidelines with respect to ground preparation and foundation construction presented in the Soils Report are incorporated in the project plans and design, and implemented during construction. This Soils Report also presents 1) the findings of the geotechnical field investigation, 2) the summary of potential geological/seismic hazard assessment, and 3) the results of laboratory tests performed.

We at ASE appreciate the opportunity to provide our professional services on this important project, and look forward to assisting you during construction phase of the Buildings.

If you have any questions or require additional information, please contact the undersigned.

Respectfully submitted,
ASSOCIATED SOILS ENGINEERING, INC.

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1.0 INTRODUCTION

This Soils Report presents the results of ASE's geotechnical investigation for the proposed development of new residential buildings (the Buildings) to be located on an approximately 1.57-acre lot located at 1007 East Victoria Street, in the City of Carson, California. The approximate location of the Site is shown on Figure 1, Site Location Map. The purpose of this investigation was to evaluate the general subsurface soil conditions at the Site and provide geotechnical recommendations for the design and construction of the Buildings. This Soils Report presents the summary of the data collected, the results of ASE's engineering evaluations/analyses, and the pertinent geotechnical conclusions and recommendations.

1.1 Project Outline

The project information below is understood to be applicable at the time of this Soils Report preparation.

1.1.1 Building/Development Scope:

Based on the project information, ASE understands that the planned development is to consist of thirty-eight (38) two- to three-story townhomes (the Buildings) across the Site. It is assumed that the Buildings will be of frame, masonry and concrete construction. Appurtenant construction will include new parking, hardscaping, landscaping and associated utility connections.

1.1.2 Structural Loading for Geotechnical Analyses:

In the absence of structural loading information from the Structural Consultant at the time of this Soils Report preparation, and for the purpose of relevant foundation analyses, ASE had assumed that the Buildings will be supported by isolated pad footings and continuous spread footings, with maximum concentrated column load (D + L) on the order of 80 kips, and with a maximum line load (D + L) not exceeding 4,000 pounds per linear foot. In response, tolerable total and differential settlements resulted from the aforementioned structural loadings have been assumed to be on the order of one (1) inch and one-third (1/3) inch over a 30-foot linear distance, respectively.

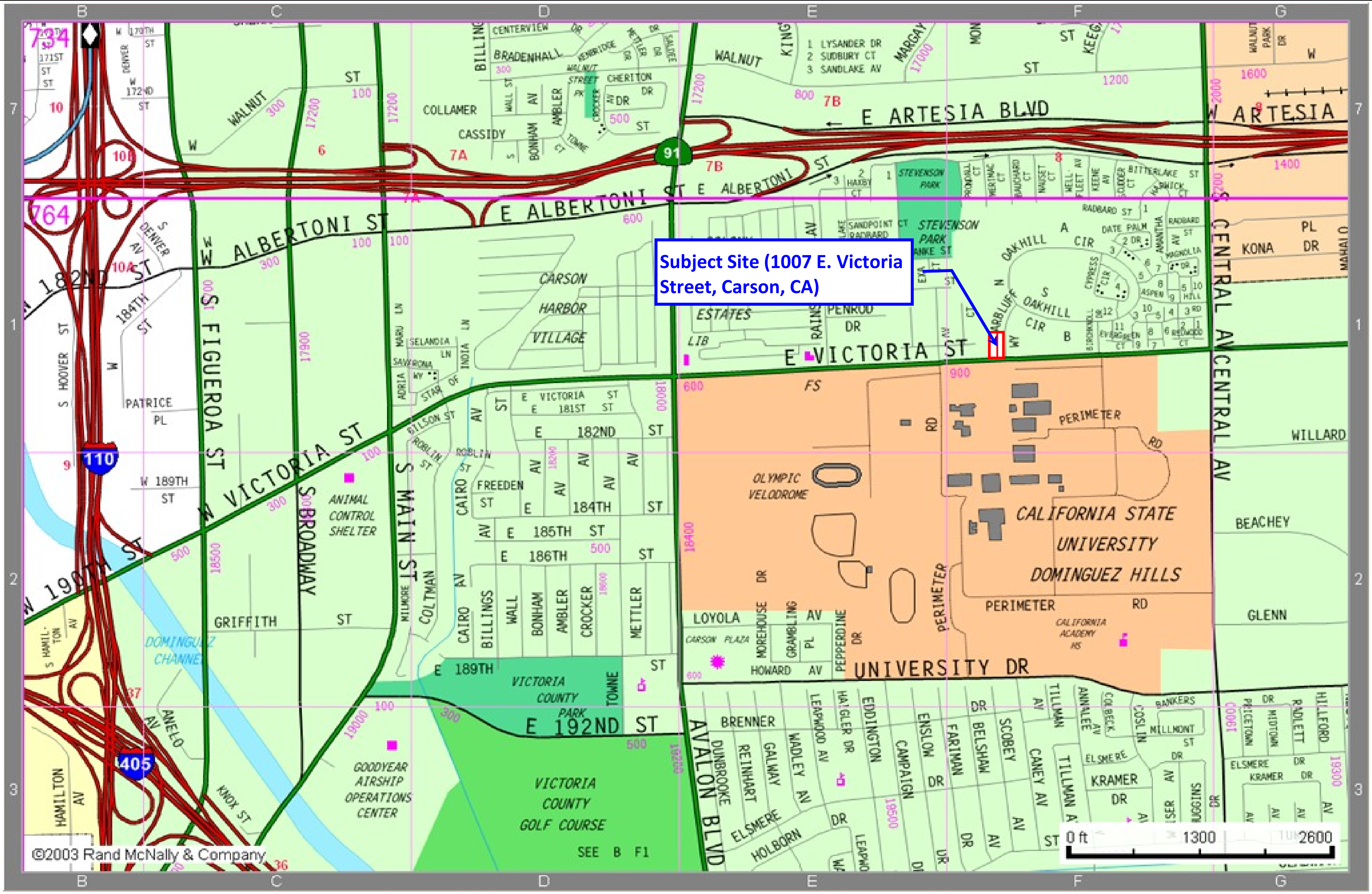
1.2 Scope of Exploration

In accomplishing the subject investigation, ASE's staff had performed the following geotechnical tasks:

A. Review of readily available background information, including in-house geotechnical data, geologic maps, seismic hazard maps, and literature relevant to the subject Site.

B. A geotechnical site reconnaissance to observe the general surficial soil conditions at the Site and to select and mark boring locations, followed by notification to Underground Service Alert of the planned boring locations 72 hours prior to field exploration.

C. Field exploration consisting of drilling six (6) exploratory borings to depths ranging from 5 feet 11 inches to 35 feet 6 inches below respective existing grades. ASE staff logged and sampled representative soils



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Site Location Map

Proj. Name:	Brandywine Homes Prop. New Residential Development 1007 E. Victoria Street, Carson, CA
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Date:	July, 2018

Figure 1

encountered in each exploratory boring. Locations of the exploratory borings on site are shown on the Boring Location Plan, Plate A, in Appendix A.

D. Field percolation testing at two (2) pre-selected test locations to measure infiltration rate of site soils as part of the requirements for the planning and design of on-site stormwater BMP system.

E. Laboratory testing on retrieved representative soil samples for classification and for determination of pertinent engineering properties.

F. Engineering analyses of data obtained from literature review, site investigation and laboratory testing covering the following aspects:

- Evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials.
- Assessment of geologic/seismic hazards based on the pertinent criteria required by the California Geological Survey (CGS).
- Determination of the seismic design parameters in accordance with Chapters 16 and 18 of the California Building Code, 2016 Edition (2016 CBC).
- Evaluation of the suitability of on-site soils for foundation support and establishment of qualification criteria for on-site or imported fill material.
- Recommendations for site remedial grading and subgrade preparation for the Buildings and appurtenant improvements.
- Recommendations for excavation and shoring.
- Recommendations for subgrade preparation and minimum design parameters for slab-on-grade, flatwork and hardscape support.
- Recommendations for design of shallow footing foundations, including allowable soils bearing capacity, estimated settlement, and lateral resistance, and alternative Post-Tensioned (PT) slab foundations.
- Evaluation of the corrosion and expansion potential of the on-site materials.
- Measurement of percolation rate of site soils required for the planning and design of on-site stormwater BMP system planning and design.

G. Preparation of this Soils Report presenting the work performed and data acquired, as well as summarizing our conclusions and geotechnical recommendations for the design and construction of foundation supporting the Buildings. The calculated design percolation rate is also presented in the Soils Report.

Please note that ASE's geotechnical investigation did not include any evaluation or assessment of hazardous or toxic materials which may or may not exist on or beneath the Site. ASE does not consult in the field of potential site contamination/mitigation.

2.0 SITE AND SUBSURFACE CONDITIONS

2.1 Location

The Buildings are to be located on an approximately 1.57-acre lot located at 1007 East Victoria Street, in the City of Carson, California. The following information pertaining to site conditions was logged during the course of ASE’s field work.

2.2 Boundary Conditions and Existing Development

The approximate 1.57-acre Site is bound to the south by East Victoria Street, with Cal State Dominguez Hills beyond. Cedarbluff Way is east of the Site, with townhome development beyond. Residential development is north and west of the Site.

The Site is presently undeveloped, but has been graded in the past. Site surface exhibits a westerly gradient, with an approximate four (4) to six (6) feet elevation differential from east (higher) to west (lower). The southerly approximate one-half (1/2) of the Site exhibits graded pads stepped downwards from east to west, with elevation differentials of one (1) to two (2) feet between the graded pads.

The Site surface at the time of ASE’s field investigation was covered with a light to moderate, scattered growth of wild grasses and weeds. A masonry wall “Dominguez Hills Village" development monument is located at the southeast corner of the Site. Several palm trees are present around the sign and along Cedarbluff Way east of the Site. Landscaping planter fronts the sign, with some shrubs continuing along Cedarbluff Way.

2.3 Subsurface Conditions

The subsurface soils descriptions presented hereunder have been interpreted from conditions exposed during the field investigation and/or information inferred from the reviewed geologic literature. As such, it is likely that not all of the subsurface conditions at the Site could be captured or represented. It is therefore essential that the Geotechnical Consultant’s engineer or geologist should be on site during grading and foundation construction such that information/recommendations deciphered during geotechnical investigation phase could be verified and, if necessary, amended as appropriate.

2.3.1 Artificial Fill (af):

Artificial fill was observed in all of ASE’s exploratory borings ranging from a minimum of approximately 4.5 feet deep in Borings B-1 and B-6, to a maximum of 10.5 feet deep in Boring B-4.

The encountered artificial fill generally consists of light olive brown silty fine sands with clay, light yellowish brown to brown clayey silts with sand, brown to dark brown silty clays, and dark brown to yellowish brown to grayish brown to brown silty clays with sand, with some asphaltic concrete (AC), concrete and paper pieces, and has been classified as "undocumented fill" and evaluated accordingly due to the lack of documentation substantiating prior compaction effort.

2.3.2 Older Alluvium Deposits (Qoa):

Native site soils consisting of late Pleistocene-age older alluvium deposits were encountered beneath the artificial fill in ASE's exploratory borings to the maximum explored depth of 35 feet 6 inches in Boring B-2. Per Reference 5, the older alluvium deposits are characterized as unconsolidated sand, silt and clay deposits associated with the raised and incised Torrance Plain. In specific, the on-site alluvium deposits consist of silty sands, sands, sandy silts, sandy silts with clay, silts, silts with sand, silts with clay, clayey silts, clayey silts with sand, and silty clays.

Blow counts recorded from advancing Standard Penetration Test (SPT) sampler and Modified California barrel sampler empirically indicate that the granular, sandy strata of on-site older alluvium deposits soils are in a medium dense to very dense condition, while fine-grained soils exhibit stiff to hard consistencies. Site subsurface soils were, in general, in a damp to moist condition within the explored depth at the time of ASE's site investigation.

More detailed descriptions of soils encountered and conditions observed during the subsurface exploration are shown in the Field Logs of Borings B-1 through B-6 in Appendix A, together with information including soil classifications, depths and types of soil samples, blow counts, field dry densities and moisture contents, and corresponding laboratory tests performed.

2.4 Groundwater and Caving

During field exploration, groundwater was not encountered to the maximum explored depth of 35 feet 6 inches below existing grade in Boring B-2. Published data in Seismic Hazard Zone Report 035 for the Torrance 7.5-Minute Quadrangle, Los Angeles County, California by CGS (1998) is inconclusive as there are no groundwater contours in the vicinity of the subject Site. A search on Google Earth indicates that the Site is approximately 97 to 105 feet above Mean Sea Level (MSL).

Information from the State of California Water Resources Control Board Geotracker website (http://geotracker.waterboards.ca.gov) indicates that the groundwater elevation in groundwater monitoring wells MW-3 and MW-11, on the west side of Avalon Boulevard between East 184th Street and East 186th Street (Thrifty #267 (Former): 18523 South Avalon Boulevard - approximately 0.7 mile southwest of the Site), was 16.2 feet below grade on August 26, 2014. The ground surface elevation at this site location (taken from Google Earth images) is 24 to 25 feet above MSL, which is approximately 72 to 81 feet lower than site grades.

Information available from the Los Angeles County Public Works Hydraulic/Water Conservation Records Division website (www.ladqw.org) indicates that the historic high groundwater level in Well No. 833, located west of Avalon Boulevard and north of Victoria Street, (within Carson Harbor Village approximately 0.65 mile west of the Site), was 51.2 feet below ground surface elevation on April 8, 1935. The ground surface elevation at this location (taken from Google Earth images) is 42 feet above MSL, which is approximately 55 to 63 feet lower than site grades. The depth to groundwater for the most recent reading in this well taken on May 1, 1952, was 71.6 feet below ground surface elevation.

Generally, seasonal and long-term fluctuations in the groundwater may occur as a result of variations in subsurface conditions, rainfall, run-off conditions and other factors. Therefore, variations from our observations may occur, as ASE’s exploratory borings were not installed for long-term groundwater monitoring purpose. Also the use of hollow-stem augers during drilling precluded observation of potential caving conditions which may have otherwise occurred in an uncased hole. Caving and/or sloughing were not measured during the extraction of auger stem at the completion of boring operations. However, caving and/or soil sloughing may be likely in excavations greater in dimension than our exploratory borings.

2.5 Utilities

No overhead or underground utilities were encountered or disturbed during the course of ASE's on-site exploration. However, overhead and underground utilities are present along site perimeter streets. Other utilities, though not known at the time of this report preparation, may be present on site, and should be located and incorporated into site development plans accordingly.

3.0 GEOLOGY

3.1 Regional Geologic Setting

The Site is located in the Central Block of the Los Angeles Basin. The Los Angeles Basin is a large northwest trending synclinal depression at the southern end of the Transverse Ranges and at the northwestern end of the Peninsula Range geomorphic Provinces of California. The Central Block is bounded by the active Newport-Inglewood Fault Zone (located 0.5 mile (0.8 km) southwest of the Site) and the active Whittier Fault Zone (approximately 15.8 miles (25.5 km) northeast of the Site).

3.2 Geologic and Soil Units

3.2.1 General:

The native geologic and soil units encountered beneath the artificial fill in each of the borings, and extending within the explored depths on the Site, consist predominantly of late Pleistocene Age alluvium deposits associated with the elevated Torrance Plain. The following section discusses the on-site alluvium soils in more detail.

3.2.2 Late Pleistocene-Age Older Alluvium Deposits (Qoa):

In accordance with CGS (1998; Reference 5), soils within the unit of older alluvium deposits were deposits associated with the raised and incised Torrance Plain. Soils within the unit were found to predominantly consist of sand, silt, and clay. In specific, on-site alluvium soils predominantly contain silty sands, sands, sandy silts, sandy silts with clay, silts, silts with sand, silts with clay, clayey silts, clayey silts with sand, and silty clays. Figure 2, Local Geologic Map, excerpt from CGS (Reference 5), shows Quaternary geology in the Site vicinity.

4.0 FAULTING AND SEISMICITY

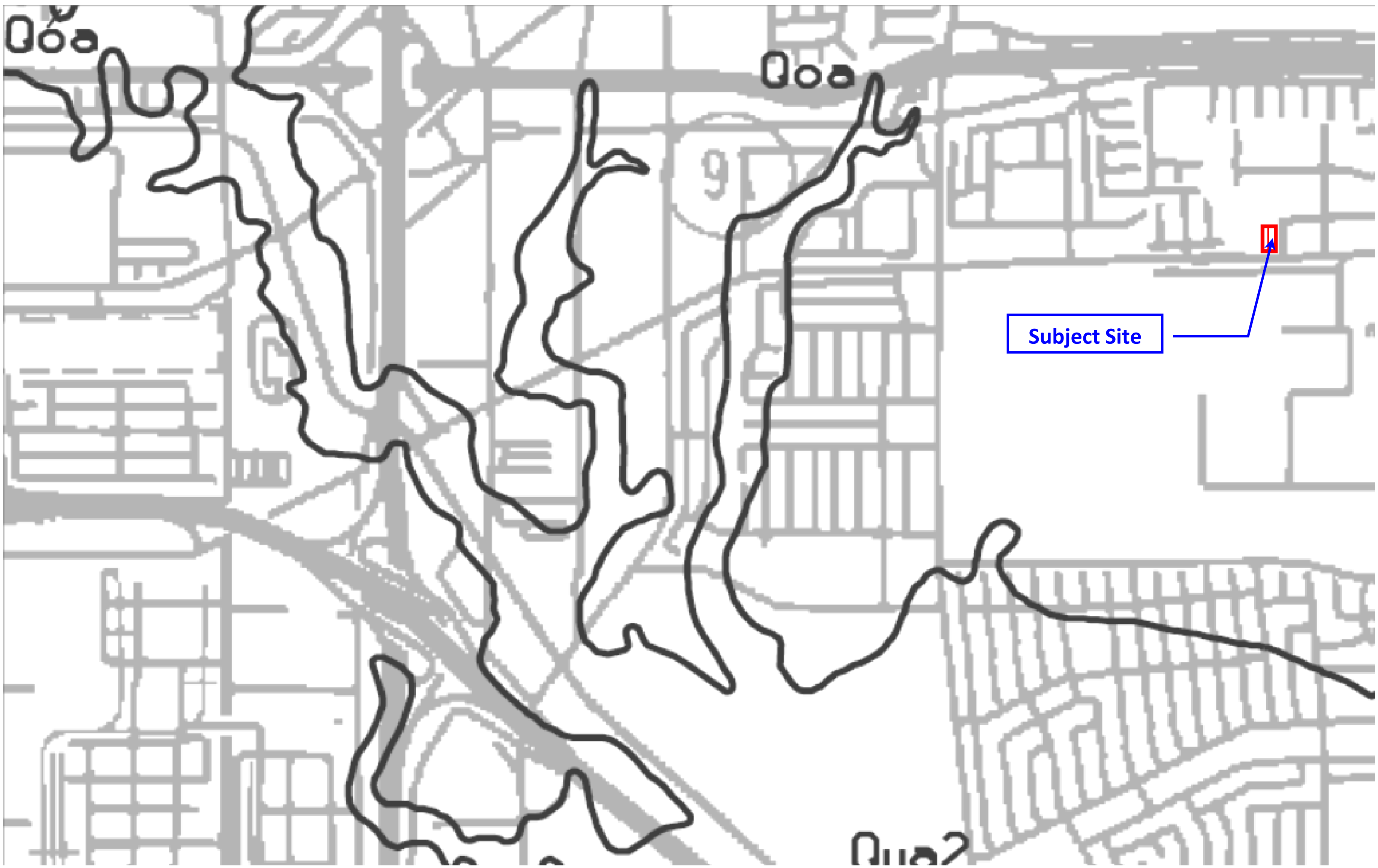
Carson, like the rest of southern California, is located within a seismically active region as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional faults such as the San Andreas, San Jacinto, Newport-Inglewood and Whittier-Elsinore fault zones.

By the definition of CGS, an active fault is one which has had surface displacement within the Holocene Epoch (roughly the last 11,000 years). The CGS has defined a potentially active fault as any fault which has been active during the Quaternary Period (approximately the last 1,600,000 years). These definitions are used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazard Zones Act of 1972 and as subsequently revised in 1997 as the Alquist-Priolo Earthquake Fault Zoning Act and Earthquake Fault Zones. The intent of the act is to require fault investigations on sites located within Special Studies Zones to preclude new construction of certain inhabited structures across the trace of active faults. The subject Site is not located within the Alquist-Priolo Earthquake Fault Zone. In addition, the Site is not located within a seismic hazard zone per CGS's mapping.

Several sources were researched for information pertaining to site seismicity. The majority of data was obtained from the program, EQFAULT, by Blake (2000) that allows for an estimation of peak horizontal ground acceleration (PGA) using a data file of approximately 150 digitized California faults. This program compiles information including the dominant type of faulting within a particular region, the maximum earthquake magnitude each fault is capable of generating, the estimated slip-rate for each fault, and the approximate location of the fault trace. Printouts of the results of the fault search for the Site are shown as Plates I-1 and I-2 in Appendix B.

4.1 Deterministic Analysis

Based on the referenced literature and deterministic analysis performed with the EQFAULT software, the



(Partial Excerpt of the Quaternary Geologic Map of the Torrance 7.5- Minute Quadrangles, California Division of Mines and Geology, Seismic Hazard Zone Report 035, 1998)

LEGEND

- Approximate Site Location
- Q_{oa} Late Pleistocene-age older alluvium consisting of dense to very dense sand and silty sand

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Figure 2	**Local Geologic Map**		
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Newport-Inglewood (L.A. Basin) Fault, approximately 0.5 mile (0.8 km) from the Site, would probably generate the most severe site ground motions. A Maximum Probable Earthquake (MPE), i.e. the maximum earthquake that is considered likely to occur during a 100-year time interval, of 7.1 Mw (moment magnitude as per USGS) has been assessed along the Newport-Inglewood (L.A. Basin) Fault. As shown on Plate I-2 in Appendix B, estimated PGA resulting from a MPE event on the Newport-Inglewood (L.A. Basin) Fault is on the order of 0.555g should this event occur at the fault's closest approach to the Site. Other nearby active faults include the Palos Verdes Fault and the Puente Hills Blind Thrust Fault, located approximately 6.9 miles (11.1 km) and 9.2 miles (14.8 km) away, respectively. In sum, approximately 44 active or potentially active faults have been found within 62 miles (100 km) of the Site.

4.2 Probabilistic Analysis

The seismicity of the Site was evaluated utilizing probabilistic analysis available from CGS (www.quake.ca.gov/gmaps/PSHA/psha_interpolator.html). The Maximum Probable Earthquake (MPE) and the Maximum Considered Earthquake (MCE) that carry 10 percent and 2 percent exceedance probabilities, respectively, in 50 years have been considered. Based on a typical damping ratio of 5% and V_s^{30} value of 387 m/sec, derived from the "Set Site Parameters for Web Services"' function as part of the "Hazard Spectrum Calculator (Local)" application available from the "OPENSHA" website, three spectral acceleration values representing peak ground acceleration (PGA), spectral acceleration for structural period of 0.2 second (Sa – 0.2 sec; typical of low-rise buildings) and spectral acceleration for structural period of 1.0 second (Sa – 1.0 sec; typical of multi-story buildings) have been analyzed and are tabulated below.

Seismic Acceleration Values from CGS's Ground Motion Interpolator (2008)

Latitude	Longitude	V_s^{30} (m/sec)	Scenario	Acceleration (g)		
				PGA	Sa – 0.2 sec	Sa – 1.0 sec
N 33.8681°	W 118.2555°	387	MPE [1]	0.380	0.875	0.390
			MCE [2]	0.681	1.558	0.765

1. MPE scenario carries a 10% exceedance probability in 50 years.
2. MCE scenario carries a 2% exceedance probability in 50 years.

4.3 2016 CBC Seismic Design Parameters

The earthquake design requirements listed in 2016 CBC and other governing standards account for faults classified as "active", in accordance with the most recent fault listing as per the United States Geological Survey (USGS) or the CGS. The seismic design of the proposed structures should be implemented in accordance with the applicable provisions stipulated in 2016 CBC unless otherwise specified by the governing authority having jurisdiction over the project.

The 2016 CBC seismic design criteria for the Site based on a Site Class of "D", a Risk Category II and a scenario of Risk-Targeted Maximum Considered Earthquake (MCE_R) that carries a 2% exceedance probability in 50 years had been determined utilizing the U.S. Seismic Design Maps web-application available from the Seismic Design Maps and Tools webpage on the website of Earthquake Hazard Program of USGS (http://earthquake.usgs.gov/hazards/designmaps/usdesign.php). Summaries of the seismic coefficients for the Site are tabulated below.

2016 CBC SEISMIC DESIGN PARAMETERS					
Site Latitude:	N 33.8681°	**Site Longitude:**	W 118.2555°	**Risk Category** [a]	II
Seismic Parameter				**Recommended Value**	
Site Class [b]				D	
Soil Profile Name [b]				Stiff Soil Profile	
Site Coefficient, Fa [c]				1.0	
Site Coefficient, Fv [d]				1.5	
0.2-Second Spectral Response Acceleration, S_S [e]				1.653g	
1.0-Second Spectral Response Acceleration, S_1 [f]				0.613g	
Adjusted 0.2-Second Spectral Response Acceleration, S_{MS} [g]				1.653g	
Adjusted 1.0-Second Spectral Response Acceleration, S_{M1} [h]				0.919g	
Design 0.2-Second Spectral Response Acceleration, S_{DS} [i]				1.102g	
Design 1.0-Second Spectral Response Acceleration, S_{D1} [j]				0.613g	
Long -Period Transition Period, T_L [k]				8 sec	
Mapped MCE_G Geometric Mean Peak Ground Acceleration, PGA [l]				0.622g	
Site Coefficient, F_{PGA} [m]				1.0	
MCE_G Peak Ground Acceleration adjusted for Site Class Effect, PGA_M [n]				0.622g	
Risk Category		**I or II or III**		**IV**	
Seismic Design Category based on SD_S [o]		D		D	
Seismic Design Category based on SD_1 [p]		D		D	

a Per 2016 CBC Table 1604.5
b Per 2016 CBC Section 1613.3.2
c Per 2016 CBC Table 1613.3.3(1)
d Per 2016 CBC Table 1613.3.3(2)
e Per 2016 CBC Figure 1613.3.1(1)
f Per 2016 CBC Figure 1613.3.1(2)
g Per 2016 CBC Equation 16-37
h Per 2016 CBC Equation 16-38
i Per 2016 CBC Equation 16-39
j Per 2016 CBC Equation 16-40
k Per ASCE 7-10 Figure 22-12
l Per ASCE 7-10 Figure 22-7
m Per ASCE 7-10 Table 11.8-1
n Per ASCE 7-10 Equation 11.8-1 = PGA x F_{PGA}
o Per 2016 CBC Table 1613.3.5(1)
p Per 2016 CBC Table 1613.3.5(2)

Please note that conformance to the 2016 CBC seismic design criteria does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not take place during the occurrence of a MCE_R event. The primary goal of seismic design is to protect life and not to avoid all damage, since such design may be economically prohibitive. Following a major earthquake, a building may be damaged beyond repair, yet not collapse. The Structural Consultant should review the pertinent parameters to evaluate the seismic design.

5.0 GEOLOGIC HAZARDS

5.1 Surface Fault Rupture and Ground Shaking

The Site is not located within an Alquist-Priolo Earthquake Fault Zone. No known active or potentially active faults are shown crossing the Site on published maps reviewed. No evidence for active faulting was encountered in the exploratory excavations performed during this evaluation. The risk of surface rupture at the Site is considered very low. Also, being in close proximity to several known active and potentially active faults, severe ground shaking should be expected during the life of the proposed development.

5.2 Seismic Hazards

5.2.1 Liquefaction:

As evidenced in Figure 3, Local Seismic Hazard Map, the Site is not within an area identified as having a potential for soil liquefaction when subject to a MPE or MCE event.

The term "liquefaction" describes a phenomenon in which a saturated cohesionless soil loses strength and acquires a degree of mobility as a result of strong ground shaking during an earthquake. The factors known to influence liquefaction potential include soil type and depth, grain size, relative density, groundwater level, degree of saturation, and both the intensity and duration of ground shaking. The soils to the maximum explored depth of 35 feet 6 inches generally consist of medium dense to very dense granular soils, and stiff to hard fine-grained soils.

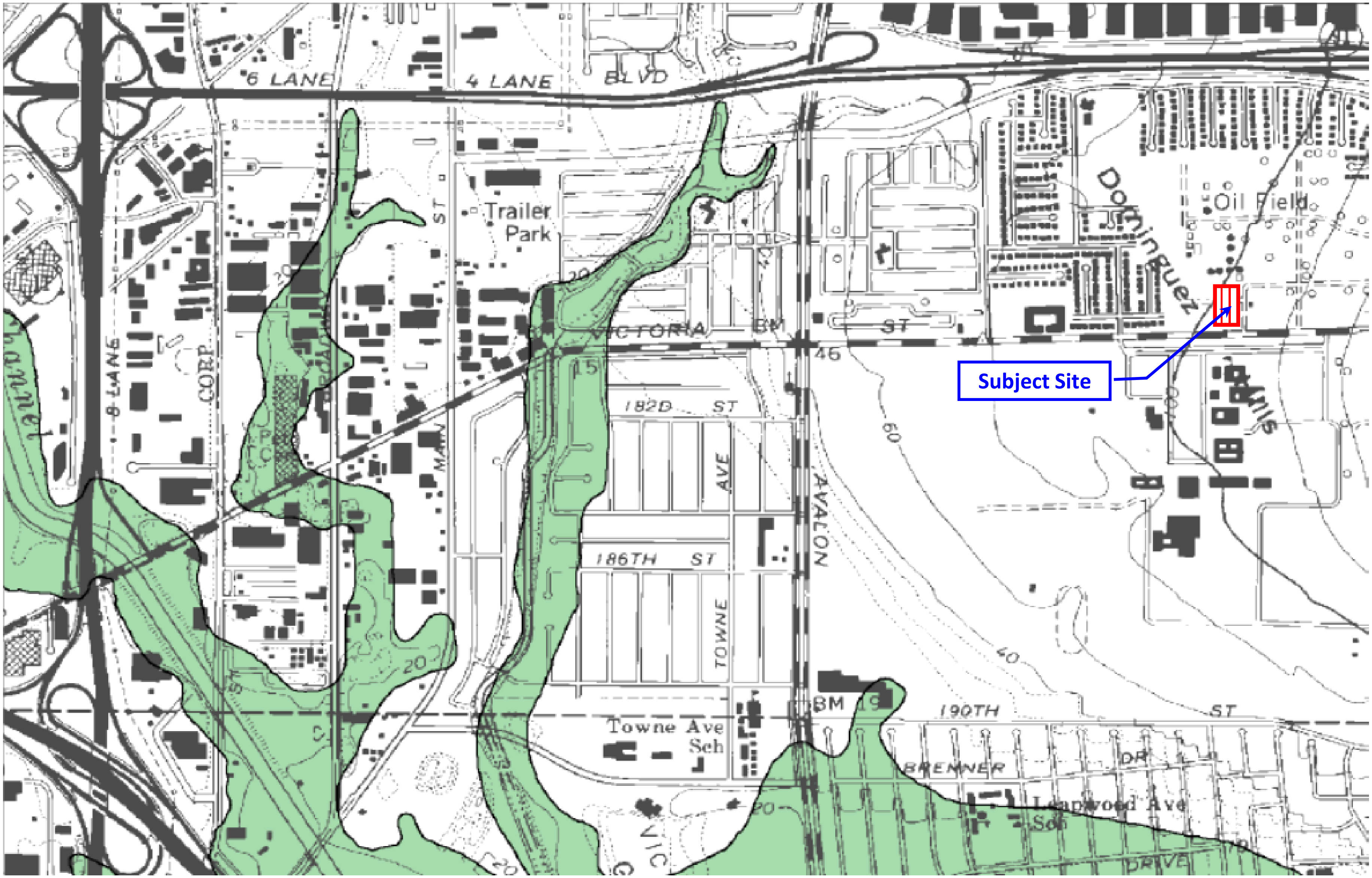
During ASE's field exploration, groundwater was not encountered to the maximum explored depth of 35 feet 6 inches below grade in Boring B-2. Per the referenced CGS (1998) historic high groundwater in the vicinity of the Site is inconclusive. According to the information from the Los Angeles County Public Works Department Hydraulic/Water Conservation Division website, historic high groundwater in a well located approximately 2/3 mile west of the Site is approximately 51.2 feet deep.

Considering that 1) groundwater was not encountered in Boring B-2 to a maximum explored depth of 35 feet 6 inches below existing grade, 2) historic high groundwater in a well near the vicinity of the Site is in excess of 50 feet below site grade based on ASE's literature and County website review, and 3) the existing native granular soils are in a medium dense to very dense state and fine-grained soils exhibit stiff to hard consistencies, the likelihood of occurrence of seismically-induced liquefaction at the Site is deemed negligible.

5.2.2 Seismic Settlements:

Ground accelerations emitted from a seismic event can cause densification of loose soils both above and below the groundwater table that may result in settlements on ground surface due to volumetric compression of soil mass. This phenomenon is often referred to as seismic settlement and commonly takes place in relatively clean sands, as well as soils with low plasticity and less fines.

While the site soils encountered consist predominantly of medium dense to very dense granular native soils, and stiff to hard fine-grained native soils, and are considered non-liquefiable due to the



(Partial Excerpt of the State of California Seismic Hazard Zones Torrance Quadrangle Official Map dated March 25, 1999)

LEGEND

- Approximate Site Location
- Potential Landslide Hazard Area
- Potential Liquefaction Hazard Area

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	Figure 3	**Local Seismic Hazard Map**	
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deep groundwater elevation beneath the Site, the earth materials on site above the groundwater level may still undergo seismically-induced densification, which is estimated not to exceed 1/4 or 1/2 inch, when subject to a MPE or MCE seismic event, respectively.

5.2.3 Earthquake-Induced Landslides:

There is no indication that recent landslides or unstable slope conditions exist on or adjacent to the project Site that would otherwise result in an obvious landslide hazard to the proposed development or adjacent properties.

ASE's review of the same geohazard map that was based upon for the production of Figure 3 indicates that the Site is not located within an area identified as having a potential for earthquake-induced landslides. Due to the lack of significant unretained relief on or adjacent to the Site, the potential for earthquake induced landslides in the future is considered nil.

5.2.4 Lateral Spreading:

Lateral spreading, a phenomenon associated with seismically-induced soil liquefaction, is a display of lateral displacement of soils due to inertial motion and lack of lateral support during or post liquefaction. It is typically exemplified by the formation of vertical cracks on the surface of liquefied soils, and usually takes place on gently sloping ground or level ground with nearby free surface such as drainage or stream channel. Since there is no presence of free surface on or near the Site, and since it is unlikely that seismically-induced liquefaction is to take place on or near the Site, the potential for the occurrence of liquefaction-induced lateral spreading is deemed unlikely on the Site.

5.2.5 Tsunamis and Seiches:

Due to the elevation of the Site and absence of nearby waterfront, hazard from tsunami is considered very low.

Seiches are rhythmic movements of water within a lake or other enclosed or semi-enclosed body of water, generally caused by earthquakes. Since no lakes or other enclosed bodies of water lie on or near the Site, the hazard from seiches is not present at the Site.

5.2.6 Flood Hazards:

The Site was located on the ESRII/FEMA Hazard Awareness site. The Site is not located within the limits of the 100 year flood plain per FEMA Flood Insurance Rate Map (Map No. 06037C1935F, map revised September 26, 2008), and is located outside an area of 0.2-percent-annual-chance flood.

6.0 GEOTECHNICAL CONSIDERATIONS AND RECOMMENDATIONS

Based on the results of field exploration, laboratory testing, and engineering analysis, it is ASE's geotechnical opinion that the Buildings may be constructed as planned, provided that the ground preparation and foundation design criteria recommended herein are incorporated into the project plans and specifications and implemented during construction.

The major geotechnical factors affecting the design and construction of the Buildings include the following:

1. Soil disturbances as a result of site demolition, clearing and excavation operations.
2. Presence of undocumented fills across the entirety of the Site.
3. Presence of soils that exhibit "Medium" expansion potential at shallow depth that may heave or shrink significantly and unevenly upon saturation and drying, respectively, resulting in potentially excessive uneven displacement of overlying foundations, structural improvements, flatworks, and pavements.

In consideration of the above factors, it is ASE's opinion that overexcavation and backfilling with properly compacted fill in the pad areas of the Buildings, as recommended herein, will be essential to reduce unfavorable foundation and slab displacement caused by static settlements of underlying soils, including shrinkage or heaving of "Medium" expansive site soils, and to provide satisfactory bearing stratum for the Buildings. The grading recommendations provided herein should be reviewed when final grading plans become available. It is assumed that the finish grades will be close to existing site grades ($\pm$ two feet).

Conventional shallow foundations, i.e. continuous spread footings and isolated pad footings bearing on properly compacted fill, combined with slab-on-grade, is deemed feasible for supporting the Buildings and appurtenant construction. Alternatively, PT slab foundation is also deemed feasible, especially in the presence of "Medium" expansive site soils.

6.1 Site Preparation

6.1.1 Existing Improvements:

Prior to grading operations, it will be necessary to remove designated existing improvements, including any remaining buried obstructions, which may be in the areas of proposed construction. Structure removal should include foundations. Concrete flatwork and asphalt pavement should also be removed from the areas of proposed construction. Concrete and asphalt fragments from site demolition operations should be disposed of off-site.

6.1.2 Surface Vegetation:

Surface vegetation should be stripped from areas of proposed construction. Stripping should penetrate six (6) inches into surface soils. Any soil contaminated with organic matter (such as root systems or strippings mixed into the soil) should be disposed of off-site or set aside for future use in non-structural landscaped areas. Removal of trees and shrubs should include rootballs and attendant root systems.

6.1.3 Underground Utilities:

Any underground utilities to be abandoned within the zone of proposed construction should be cut off a minimum of five (5) feet from the area of the new structures. The ends of cut-off lines should be plugged a minimum of five (5) feet with lean concrete exhibiting minimum shrinkage characteristics to prevent water migration to or from hollow lines. Capping of lines may also be required should the plug be subject to any line pressure.

Alternatively, deep hollow lines may be left in place provided they are filled with lean concrete or 2-sack control density fill (slurry fill). No filled line should be permitted closer than two (2) feet from the bottom of future footings, unless it has been evaluated and approved by the Geotechnical Consultant. However, local ordinances relative to abandonment of underground utilities, if more restrictive, will supersede the above minimum requirements.

6.2 Site Grading

In view of minimizing the potential adverse effects associated with the development of excessive total or differential settlement/heave underneath the Buildings, as well as to ensure uniform bearing competency for the foundations and slabs, preparation of on-site soils are recommended in the following sections.

6.2.1 Undocumented Fill/Disturbed Native Soils:

All undocumented fill soil, as discussed in Section 2.3.1 above, encountered during site grading in the area of the Buildings, as well as any native soils disturbed during demolition and clearing operations, should be excavated full depth under the observation and confirmation by the Geotechnical Consultant. Lateral extent of overexcavation beyond Building perimeters, where possible, should be to a minimum distance equal to the depth of undocumented fill/disturbed soil encountered or five (5) feet, whichever is greater.

For other secondary improvements such as free-standing walls or hardscape, the lateral extent of removal should be to a minimum distance equal to the depth of undocumented fill/disturbed soils encountered or one (1) foot, whichever is greater.

The exposed excavation bottom should be scarified/reworked to a minimum one (1) foot depth and recompacted to at least 90 percent relative compaction with a minimum moisture content of two (2) percentage points above optimum moisture content, prior to backfilling with approved soils as specified in Section 6.2.9. Unless otherwise stated, the measurement of relative compaction in this report should always refer to ASTM D1557-12 Test Method.

6.2.2 Expansive Soils:

Laboratory testing results on a near surface soil sample indicate a “Medium” soil expansion potential (i.e. Expansion Index, EI = 73 per ASTM D4829-11 Test Method) as defined in 2016 CBC. For slabs and structural elements supported by approved fill materials complying with criteria stipulated in Sections 6.2.3 and 6.2.6 below, the overall soil expansion is anticipated to be reduced from the present state. Nonetheless, it may be desirable that the soil expansion potential be re-evaluated through additional testing during or after rough grading operations to verify the design adequacy of foundation or slab-on-grade against the re-tested soil expansion potential as heterogeneity within soil mass is not uncommon.

Lightly loaded structural elements such as shallow foundations and slabs are likely to undergo significant movements due to the “Medium” expansion potential of site surficial soils. Such magnitude of movement could potentially cause distress such as cracks, deformation and/or misalignments to the overlying foundations, slabs, or walls. It should be noted that design provisions, such as the use of “Very Low” to “Low” expansive fill beneath lightly loaded structural elements, adequate reinforcements, deeper foundations or other measures, as presented in Sections 6.2 and 6.3 of this Soils Report, may help to alleviate the effects of “Medium” soils expansion on the foundations and structures but may not completely eliminate the problem. Future Buildings maintenance and repair as a result of the presence of “Medium” expansive soils could not be totally ruled out.

6.2.3 Remedial Grading:

In areas where the existing fill soils on-site extend to depths of four (4) feet or less, to provide acceptable support for building foundations and slabs, it is recommended that on-site fill and underlying native soils within the footprint of the Buildings be overexcavated and removed uniformly to a minimum depth of four (4) feet below existing grade, or two (2) feet below the bottom of the lowest footing, whichever is lower, and replaced with properly compacted fill such that the building foundations and slabs are supported on a re-engineered, compacted fill layer. The excavation bottoms should be near uniform. The overexcavation should extend laterally to a minimum distance of five (5) feet beyond Buildings perimeters, where possible.

The upper two (2) feet of compacted fill should consist of "Very Low" to “Low” expansive, fill material (Expansion Index, "EI" per ASTM D4829-11 test procedures not greater than 50), compacted to at least 90 percent relative compaction with minimum moisture content of two (2) percentage above optimum moisture content. On-site subgrade soils at their present state generally exhibit an EI exceeding the preferred value and, thus, are not deemed suitable for re-use as fill within the upper two (2) feet from finish grade. As such, suitable import sources to be used as fill for the upper two (2) feet of compacted fill should be identified, pre-tested and pre-approved prior to the onset of site rough grading. Alternatively, blending of “Very Low” expansive soil (EI ≤ 20) or crushed/pulverized PCC or AC material with the excavated on-site soils to lessen the resultant EI may be considered. The mixed soil should comply with the fill criteria stipulated in Section 6.2.8 below. The blending should ensure a thorough mixture of various soils and materials, and should be subject to testing and approval by the Geotechnical Consultant prior to use.

Soils exposed at excavation bottoms to a depth of one (1) foot should be scarified, reworked and recompacted to exhibit a minimum 90 percent relative compaction with a minimum moisture content of two (2) percentage points above the optimum moisture content prior to receiving fill placement. The exposed excavation bottom should be observed, tested, and approved by the Geotechnical Consultant prior to placing compacted fill. In case of the presence of localized loose soils, the overexcavation needs to be deepened accordingly to delete the loose soil condition. However, this deepened overexcavation may be terminated when the exposed native, undisturbed soils exhibit a natural relative compaction greater than 85 percent, subject to the testing and inspection by the representative from the Geotechnical Consultant.

If PT slab foundation is adopted for supporting the Buildings, the use of "Very Low" to “Low” expansive (EI ≤ 50) fill material within the upper two (2) feet of subgrade, as recommended in the preceding paragraph, may be omitted, as PT slabs can be structurally designed and built to withstand the site “Medium” expansive soils.

The Geotechnical Consultant should be provided with appropriate foundation details and staking during grading to verify that depths and/or locations of the recommended overexcavation are adequate. For areas on site that grading recommendations stipulated in both Sections 6.2.1 and 6.2.3 apply, the more stringent grading criteria between the two sections should govern.

The depth of overexcavation should be reviewed by the Geotechnical Consultant during the actual construction. Any subsurface obstruction, buried structural elements, and unsuitable material encountered during grading, should be immediately brought to the attention of the Geotechnical Consultant for proper exposure, removal and processing, as recommended.

6.2.4 Pumping and Heaving of Subgrade Soils:

Some existing site native soils within the zone of recommended remedial grading were found to contain relatively high moisture content at the time of ASE's site investigation, as evidenced in the respective Field Logs of Boring in Appendix A. If soils exhibiting high moisture content are encountered during site overexcavation and grading, the grading contractor may need to use track-mounted equipment in lieu of rubber tire mounted, wherever possible.

If excavated site soils are highly moist or wet at the time of site grading, the excavated soils should be evenly spread and left aerated for a period of time to reduce the moisture content prior to being re-used. If project schedule does not allow prolonged drying of any encountered wet site soils, alternative measures such as using or blending with site drier soils, imported "Very Low" expansive fill (EI ≤ 20), or crushed/pulverized AC or PCC material, if available, may be considered. The Geotechnical Consultant should be consulted at the time of site grading for the assessment of such alternative measures.

When pumping and heaving conditions are observed with soils exposed at the overexcavation bottom, it will be necessary to deepen the overexcavation within the area of observed pumping and heaving an additional 18 inches, under the observation and evaluation of the Geotechnical Consultant's representative. The area of the deepened overexcavation should subsequently be backfilled with 3/4" to 1" open-graded crushed rocks (per Section 200-1.2 of the latest edition of the Standard Specifications for Public Works Construction, i.e. the Greenbook) rammed tightly to form a stiffened platform to facilitate the ensuing fill placement and compaction. The crushed rock layer should be sandwiched top and bottom by a layer of filtering geofabric (such as Mirafi 140N or equivalent) for proper separation from overlying fill soils and underlying subgrade soils, respectively. Alternative, site-specific recommendations for the stabilization of excavation bottoms may be provided by the Geotechnical Consultant based on a review of soil conditions exposed during site grading.

6.2.5 Temporary Excavation:

Excavations of site soils 4 feet or deeper should be temporarily shored or sloped in accordance with Cal OSHA requirements.

a) Temporary Sloping:

In areas where excavations deeper than 4 feet are not adjacent to existing structures of public right-of-ways, sloping procedures may be utilized for temporary excavations. It is recommended that temporary slopes in both fill and native soils be graded no steeper than 1.5:1 (H:V) for excavations up to 20 feet in depth. The above temporary slope criteria is based on level soils conditions behind temporary slopes with no surcharge loading (structures, traffic) within a lateral distance behind the top of slope equivalent to the slope height.

It is recommended that excavated soils be placed a minimum lateral distance from top of slope equal to the height of slope. A minimum setback distance equivalent to the slope height should be maintained between the top of slope and heavy excavation/grading equipment. Should running sand conditions be experienced during excavation operations, flattening of cut slope faces, or other special procedures may be required to achieve stable, temporary slopes.

Soil conditions should be reviewed by the Geotechnical Consultant as excavation progresses to verify acceptability of temporary slopes. Final temporary cut slope design will be dependent upon the soil conditions encountered, construction procedures and schedule.

b) Temporary Shoring:

Temporary shoring will be required for those excavations where temporary sloping as specified above is not feasible.

Temporary cantilever shoring, if used, should be designed to resist an active earth pressure of 48 pounds per cubic foot (pcf) equivalent fluid pressure (EFP) for level soil conditions behind shoring. The resultant lateral deflection of shoring and surficial settlement immediately behind shoring are estimated to be on the order of one (1) to one and one half (1 ½) percent of the shored excavation depth. Should this ground deformation be intolerable to the existing structure, ASE should be consulted for more detailed analysis and further recommendations.

The design shoring should also include surcharge loading effects of existing structures and anticipated traffic, including delivery and construction equipment, when loading is within a distance from the shoring equal to the depth of excavation.

In addition to the above, a minimum uniform lateral pressure of 100 pounds per square foot (psf) in the upper ten (10) feet of shoring should be incorporated in the design when normal traffic is permitted within ten (10) feet of the shoring.

6.2.6 Exterior Slab-on-Grade/Concrete Flatwork/Hardscape/Pavement Support:

For the purpose of reducing future unsightly and uneven movements and cracks of any newly re-constructed exterior slab-on-grade, concrete flatwork, hardscape, or pavement, it is recommended that the upper eighteen (18) inches of subgrade soils below the bottom of and eighteen (18) inches laterally beyond the footprint of exterior concrete slab-on-grade/concrete flatwork/hardscape/pavement should be overexcavated and replaced with approved fill soils consisting predominantly of "Very Low" to "Low" expansive, suitable site, import or blended material (EI ≤ 50), compacted to at least 90 percent relative compaction with a minimum moisture content of two (2) percentage points above optimum moisture content. Prior to placement of the above recommended fill layer, the upper six (6) inches of exposed native subgrade should be reworked to at least 90 percent relative compaction and moisture conditioned to at least two (2) percentage points above optimum moisture content. From geotechnical viewpoint, new landscape area with only softscape is not subject to subgrade preparation and remedial grading requirements mentioned in Sections 6.2.1, 6.2.3 and 6.2.6.

6.2.7 New Fills for Grade Alteration:

If any non-structural area is to receive new fills as part of any planned grade alteration, the upper eighteen (18) inches of site soils should be reworked and recompacted to a minimum 90 percent relative compaction with a moisture content of two (2) percentage points above optimum moisture content prior to receiving any new fills, where required, to achieve finish grade elevations outside building pad areas.

6.2.8 Suitable Soils and Imported Soils:

Unless otherwise noted (such as subgrade for PT slab foundation), any soil re-used or imported as fill for the completion of subgrade preparation within two (2) feet from finish subgrade should consist of predominantly "Very Low" to “Low” expansive, granular material exhibiting an EI not greater than 50, and should be exhibiting a relatively uniform gradation, free of debris, particles greater than 4 inches in maximum dimension, organic matter or other deleterious materials. For the excavated on-site soils to be blended such that the resultant EI is not exceeding 50, a general rule-of-thumb would be blending one (1) part excavated site soils with one (1) part of imported “Very Low” (EI ≤ 20) expansive soils or base material. There is no limitation to the re-use of excavated site soils as fill for depth greater than two (2) feet from finish subgrade.

Unless otherwise approved by the Geotechnical Consultant, the fill materials should also comply with the following soil corrosivity criteria with respect to the desired concrete and reinforcement protection.

Corrosivity Criteria for Select Fill and General Fill			
Soluble Sulfate (% by weight) (1)	**Soluble Chloride (ppm) (2)**	**Resistivity Value (ohm-cm) (3)**	**pH-Value (4)**
≤ 0.1	≤ 500	≥ 2000	7.0 ~ 8.8

(1) California Test Method 417. (2) California Test Method 422. (3) ASTM G187-12a Test Method. (4) California Test Method 532.

6.2.9 Backfilling and Compaction Requirements:

Existing site soils at their present state and composition, unless indicated otherwise (such as subgrade for PT slab foundation), are considered only suitable for re-use as fill during site grading below designated depths, i.e. deeper than 2 feet from finish subgrade beneath Building interior slabs (see Section 6.2.3), and deeper than 18 inches from finish subgrade in areas of exterior slab-on-grade, concrete flatwork, hardscape, and pavement (see Section 6.2.6), non-structural or landscape areas (no depth limit), and backfilling of utility trenches (no depth limit), provided they 1) free of debris, particles greater than 4 inches in maximum dimension, organic matter or other deleterious materials, 2) are not environmentally contaminated, and 3) adequately moisture conditioned to permit achieving the required compaction. No nesting of large particles (2 to 4-inch size) should be permitted during backfilling operations.

On-site soils, blended soils and import materials approved for use as fill should be placed in horizontal lifts not exceeding 8 inches in loose thickness, moisture conditioned to a minimum of two (2) percentage points above optimum moisture content and compacted to a minimum 90 percent relative compaction, per ASTM D1557-12 Test Method, unless otherwise stated.

6.2.10 Shrinking and Subsidence:

The volume change of excavated on-site materials upon excavation and placement as engineered fill will vary with soil type, depth, location and compactive effort. However, for planning purposes, a shrinkage factor on the order of ten (10) percent should be considered for earthwork calculations. Subsidence due to scarification and recompaction of the exposed ground surfaces within removal areas has been estimated to be approximately on the order of one (1) inch.

6.2.11 Tests and Observations:

All subgrade preparation, compaction, and backfill operations should be performed under the observation of and testing by the Geotechnical Consultant's field representative. An adequate number of field tests should be taken to ensure compliance with this report and local ordinances.

If it is determined during grading that site soils require overexcavation to greater depths for obtaining proper support for the proposed structures, this additional work should be performed in accordance with the recommendations of the Geotechnical Consultant.

Imported fill soils or base materials should be examined by a representative of this office, and tested as necessary for evaluating their suitability for use as fill prior to being hauled to the Site. Final acceptance of any imported soil will be based upon review and testing of the soil actually delivered to the Site. All blended soils to be used as fill must be tested and approved by the Geotechnical Consultant prior to being used for fill placement.

6.3 Foundation Design

It is ASE’s opinion that either continuous spread footings and isolated pad footings or PT slab foundation bearing on approved compacted fill soils may be used to provide foundation support for the Buildings, provided that the site preparation recommendations presented in Section 6.2 above are incorporated in project planning and design, and implemented during site construction. Presented below are the recommended geotechnical design and construction criteria for conventional footing foundation and PT slab foundation.

6.3.1 Conventional Shallow Footing Foundation:

a) Minimum Footing Dimension and Reinforcement:

In order to mobilize sufficient soils bearing capacity supporting the new footings for the Buildings, it is recommended that the following tabulated minimum footing embedments, widths and reinforcements for various footing types be considered.

Building Height	Minimum Footing Dimension & Reinforcement					
	Continuous Spread Footing/Strip Footing			Isolated Pad Footing		
	Depth [1] (in)	Width (in)	Reinforcement [2]	Depth [1] (in)	Width (in)	Reinforcement [2]
2-story	24	15	Four #5 bars – two near the top and two near the bottom	24	30 square	Four #5 bars – two near the top and two near the bottom, applied bi-axially
3-story		18			36 square	

(1) Footing embedment measured from the nearest adjacent lowest soils grade.
(2) Based strictly from geotechnical point of view.

Grade beams having a cross section of at least 12-inch square should be provided across large entrances/openings exceeding 15 feet in span and tried to the adjoining footings to minimize differential movements that might otherwise be experienced by the slabs. Grade beams should also be designed to tie any isolated pad footings together minimize potential differential movements that might otherwise be associated with soil heave/settlements. The same minimum reinforcement recommended for the adjacent footings should be applied to the grade beam as well

Foundation design details such as concrete strength, reinforcements, etc. should be established by the Structural Consultant.

b) Allowable Soils Bearing Capacity:

For footings complying with the minimum dimension requirements stipulated in Section 6.3.1 a) above, the allowable soils bearing capacities, inclusive of both dead and live loads, should be as per tabulated below.

Allowable Soils Bearing Capacity (psf)		Increase per 12-inch Increment in Footing Width (psf)	Increase per 12-inch Increment in Footing Depth (psf)	Maximum Composite Ceiling Value (psf)
Continuous Spread Footing/Strip Footing	Isolated Pad Footing			
2,200	2,200	100	400	4,000

The above allowable bearing capacities may be increased by one-third (1/3) when subject to short-term, transient loading induced by wind or seismic activities.

c) Lateral Resistance:

Resistance to lateral loads can be assumed to be provided by passive lateral earth pressure and by friction acting on structural components in permanent contact with the subgrade soils.

For site preparation implemented as per recommended in the above Section 6.2, lateral resistance on the sides of foundations may be computed using a passive lateral earth pressure of 200 pcf EFP for footings embedded into approved compacted fill soils, subject to a maximum of 2,000 psf. An ultimate coefficient of friction on the order of 0.3 may also be used for structural dead load acting between the footing bottom and the supporting soils. The above passive lateral earth pressure may be used in conjunction with the ultimate coefficient of friction in calculating composite lateral resistance, provided the passive lateral earth pressure value is reduced by one-third (1/3). The composite lateral resistance may be increased by one-third (1/3) under transient wind or seismic loading.

d) Static Settlements/Heaves:

Total static settlements/heaves resulting from compression/expansion of subgrade soils for conventional footings designed and constructed in accordance with the above criteria, and supporting maximum assumed dead plus live (D+L) column and wall loads mentioned in Section 1.1.2 above, are not anticipated to exceed <u>two-third (2/3) inch</u>, upon implementation of site preparation as per recommended in Section 5.2 above. A static differential settlement/heave on the order of <u>one-third (1/3) inch</u> over a distance of <u>30 feet</u> is anticipated between similarly loaded adjacent isolated pad footings, as well as for continuous wall footings over a distance of approximately <u>30 feet</u>.

Please be reminded that the Geotechnical Consultant should be contracted for further evaluation and recommendations, as necessary, should final design structural loads exceed the maximum loads assumed in the above analyses by more than ten (10) percent.

6.3.2 Alternative PT Slab Foundation:

If adopted, PT slabs should be designed based on the latest PTI design method (Reference 25), which is stipulated in References 23 and 24. Both of References 23 and 24 are part of the PT-slab design procedures per 2016 CBC. The slabs should be designed for at least one (1.0) inch of differential settlement over the width of each Building.

Based on review of laboratory data for the on-site materials, the average soil modulus of subgrade reaction, K-value, to be used for design is 80 pounds per cubic inch (pci). The post-tensioned slabs may also be designed based on a surface soils bearing value of 800 psf. Specific recommendations for design following *Post-Tensioning Institute (3rd Edition)* methodology are as follows.

Post-tensioned slabs should have sufficient stiffness to resist differential movement of the corner, edge or center of slab due to non-uniform swell and shrinkage of subgrade soils and fluctuation of subgrade soil moisture content. Based on the specifications of the PTI method, 3rd Edition, the potential for differential movement can be evaluated.

The following table presents suggested minimum coefficients to be used for soils with different degrees of expansion potential. For the encountered site soils, a "**Medium**" soil expansion category should be considered for PT slab design:

PTI METOD (3RD EDITION) DESIGN PARAMETERS				
Thornswaite Index -20	**Soil Fabric Factor, Ft** 1.0		**Equilibrium Suction, pF** 3.909	
Surface Equilibrium Wet Suction, $(pF)_{wet}$ 3.0 [a]		**Surface Equilibrium Dry Suction, $(pF)_{dry}$** 4.5 [a]		
Soil Expansion Classification	Very Low (EI $\leq$ 20)	Low (20 < EI $\leq$ 50)	Medium (50 < EI $\leq$ 90)	High (90 < EI $\leq$ 130)
(e_m) edge lift (ft) [b]	5.20 [c]	4.64 [c]	4.13 [c]	3.78 [c]
(e_m) center lift (ft) [b]	9.00 [c]	9.00 [c]	8.55 [c]	7.34 [c]
(y_m) edge lift (in)	0.25	0.62	1.11	1.61
(y_m) center lift (in)	0.10	0.25	0.43	0.60

a. Values per PTI recommendations.

b. Soil parameters such as Atterberg Limits and % passing #200 have been derived from typical values available from Day (2005), Bowles (1996), and PTI (2004).

c. Governed by soil aspect consideration.

The above tabulated coefficients are considered minimums and may not be adequate to represent worst-case conditions such as adverse drainage and/or improper landscaping and maintenance. The above parameters are applicable provided structures have gutters and downspouts and positive drainage is maintained away from structures. For the Buildings, the "Medium" expansion classification should be utilized for design pending supplemental testing at the completion of rough grading of the building pads.

Deepened footings/edges around the slab perimeter must be used to minimize non-uniform surface moisture migration (from an outside source) beneath the slab. An edge depth of at least 12 inches should be considered for soils with "Low" to "Very Low" EI's and 18 inches for soils with "Medium" to "High" EI's. The bottom of the deepened footing/edge should be designed to resist tension, using cable or reinforcement per the Structural Engineer.

The entirety of each Building interior slab should be underlain by a minimum 10-mil polyvinyl chloride membrane vapor barrier with a minimum overlap of 12 inches in all directions. In order to comply with the 2016 CRC Section R506.2.3, the interior slab should be placed in direct contact with the vapor retarder, underlain by four (4) inches of clean sand or aggregate as a capillary break. The concrete slab shall consist of a concrete mix design which will address bleeding, shrinking and curling. As an alternative from a geotechnical viewpoint, the impermeable visqueen layer may be sandwiched by at least 2 inches of clean sand top and bottom prior to casting of the slab.

If the City of Carson is requiring compliance with the latest CALGreen Code, then the entire slab within each Building should be underlain by an impermeable vapor barrier (minimum 15-mil-thick visqueen) per 2016 CRC Section R506.2.3. A minimum 12-inch overlap between visqueen sheets should be ensured during placement. All visqueen sheets should be puncture free prior to slab construction.

PT slabs often develop a “dishing” or “arching” characteristic due to the fluctuation of soil moisture content underlying the perimeter and center slab. All areas to receive concrete should be presaturated to a depth of 18 inches such that the soil within this zone is at optimum moisture or higher. The Geotechnical Engineer should verify the subgrade was presaturated within 24 hours of placing the moisture barrier.

6.3.3 Retaining Walls:

Cantilevered retaining walls should be designed for an “active” lateral earth pressure value tabulated on the next page for approved granular backfill soils and level backfill conditions, whereas an “At-rest” lateral earth pressure value for approved granular backfill and level backfill conditions tabulated on the next page should be used for top-restrained retaining walls. Should site clayey/silty soils be used as backfill behind retaining walls and prolonged moisture inundation behind retaining walls is anticipated, then added lateral earth pressure accounting for soils expansion should be considered. In this regard, it is recommended that cantilevered and top-restrained retaining walls should be designed for lateral earth pressure equivalent to the “at-rest” and “passive” states tabulated below for site soils, respectively. Retaining walls subject to uniform surcharge loads should be designed for an additional uniform lateral pressure equal to one-third (1/3) and one-half (1/2) of the anticipated surcharge pressure over the full retained height of the retaining wall (measuring from the top of wall to the heel of wall footing) for cantilevered and top-restrained wall fixity conditions, respectively, as shown in Figure 4, Nearby Building Surcharge Consideration and Retaining Wall Drainage Details. Any retaining wall with a retained height exceeding six (6) feet should additionally be designed to resist seismic lateral earth pressure. It is our understanding that walls in excess of 6 feet in height are not currently planned for this Site. The Geotechnical Consultant should be consulted if this condition exists, or if the local governing agency requires the retaining wall to be designed for seismic lateral earth pressure regardless of the retained height. Footings should be reinforced as recommended to by Structural Consultant. Appropriate back drainage should be provided to avoid build-up of excessive hydrostatic pressure.

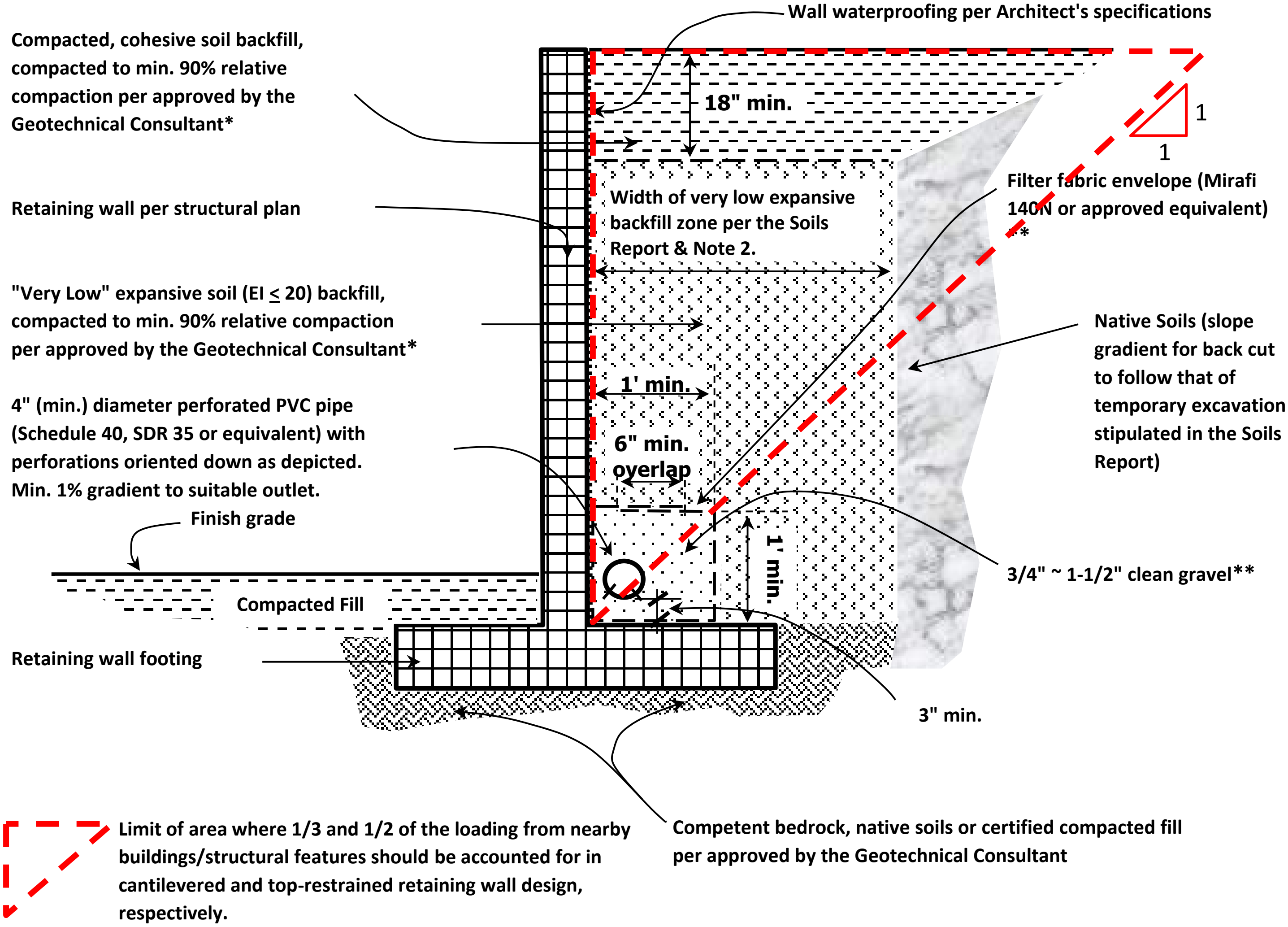
The Geotechnical Consultant should be on-site during temporary back cut and retaining wall construction to inspect and evaluate the stability of cuts and, if necessary, to provide additional remedial or mitigative recommendations.

Retaining Wall Design Parameter	Value
Allowable Soils Bearing Capacity	2,200 psf (1)(2)
Active Pressure [granular backfill (3)/site soils: level]	35/48 pcf EFP
At-rest Pressure [granular backfill (3)/site soils: level]	55/70 pcf EFP
Passive Pressure (per foot of depth)	200 pcf (4)
Coefficient of Friction	0.3 (4)
Minimum Footing Depth	24 inches
Minimum Footing Width	15 inches
Minimum Reinforcement	Four No. 5 rebar - 2 near top and 2 near bottom

(1) Based on compliance with earthwork recommendations per Section 6.2 of this Soils Report.
(2) Allowable soils bearing capacity increase for larger retaining wall footings should be as per Section 6.3.1b).
(3) Design values assuming a drained condition with “Very Low” expansive materials (EI ≤ 20) within the backfill zone and no surcharge loading conditions.
(4) Passive lateral resistance may be combined with frictional resistance provided the passive lateral earth pressure is reduced by 1/3. See Section 6.3.1c.

Preferably, the backfill should consist of approved "Very Low" expansive (EI ≤ 20) material, and should be compacted to a minimum relative compaction of 90 percent. In order to be able to utilize the active or at-rest lateral earth pressure values for granular soil backfill as listed in the following table, the extent of the “Very Low” expansive (EI ≤ 20) backfill zone should be as per the red-dotted triangular wedge depicted in Figure 4. Flooding or jetting of backfill should not be permitted. Granular backfill should be capped with 18 inches (minimum) of relatively impervious fill such as native site soils to seal the backfill and prevent saturation. Figure 4 illustrates the general configuration and requirements for retaining wall drainage. Should any conflict noticed between recommendations stated in this report and those shown in Figure 4, the fore should govern. Other retaining wall drainage alternatives such as CONTECH C-Drain system or MIRADRAIN may be considered but should first be reviewed and approved by the Geotechnical Consultant prior to implementation.

Should the space behind the new retaining wall be too tight to implement the above recommended backfill effort, as an alternative, 2-sack control density fill (slurry fill) may be used in lieu of regular soil backfill, provided that the integrity and functionality of wall backdrain is protected and maintained.



SPECIFICATIONS FOR CALTRANS CLASS II PERMEABLE MATERIAL

U.S. STANDARD 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
Sand Equivalent > 75	

* Based on ASTM D-1557-02

** If Caltrans Class II permeable material (see gradation to left) is used in place of 3/4" ~ 1-1/2" gravel, filter fabric may be deleted. Caltrans Class 2 permeable material should be compacted to minimum 90 percent relative compaction. Unless otherwise specified, a minimum of 1 cubic foot of gravel should be used for each 1 foot run of drain.

Note 1: Composite drainage products such as Contech C-Drain, Miradrain or J-Drain may be used as alternative to gravel or Class II. Installation should be performed in accordance with manufacturer's specifications.

Note 2: Width of " Very Low" expansion backfill equals 1/2 of retained height, or distance from back of wall to heel of footing, whichever is greater.

Schematic Not To Scale

Associated Soils Engineering, Inc.

2860 Walnut Avenue

Signal Hill, CA 90755

Tel (562) 426-7990 Fax (562) 426-1842

Project:	Brandywine Homes - Prop. New Residential Development, 1007 E. Victoria Street, Carson, CA
Figure 4	Nearby Building Surcharge Consideration & Retaining Wall Drainage Details
Proj. No.: 6816.18	Date: July, 2018

It should be noted that the use of heavy compaction equipment in close proximity to retaining structures can result in wall pressures exceeding design values and corresponding wall movement greater than that normally associated with the development of active or at-rest conditions. In this regard, the contractor should take appropriate precautions during the backfill placement.

6.3.4 Footing/Foundation Observation:

All footing/foundation excavations should be observed by the Geotechnical Consultant's representative to verify minimum embedment depths and competency of bearing soils. Such observations should be made prior to placement of any reinforcing steel or concrete.

6.4 Slabs-on-Grade

Concrete floor slabs in the Buildings and exterior concrete flatwork/hardscape should be supported on properly compacted soils as recommended in the Site Grading section (i.e. Section 6.2) of this report. The slab subgrade soils should also be proof-rolled just prior to construction to provide a firm, unyielding surface, especially if the subgrade has been disturbed or loosened by the passage of construction traffic. Final compaction and testing of slab subgrade should be performed just prior to placement of concrete.

Per Section 1808.6.2 of the 2016 CBC, when the WRI/CRSI Design of Slab-on-Ground Foundations is adopted, the Effective Plasticity Index (EPI) should be utilized for slab-on-grade design. The EPI defined as the weighted plasticity index (WPI) x Cs x Co. The preliminary EPI's reflecting the use of either site native soils or "Very Low" to "Low" expansive (EI ≤ 50) imported fill or blended soils for subgrade are tabulated below. However, it is of essential importance that the EPI for final slab-on-grade design should be based on additional laboratory tests performed on soils samples retrieved from rough graded building pad.

Area	WPI	C_S (1)	C_0 (2)	EPI
Compacted native soils	27	1.0	1.0	27
With either 2' or 18" of imported or blended "Very Low" to "Low" (EI ≤ 50) expansive fill soil from finish subgrade(3)	22	1.0	1.0	22

(1) For essentially flat ground, C_s = 1.0.
(2) For over-consolidated foundation materials with unconfined compressive strength not exceeding 6000 psf, C_0 effect is deemed less significant.
(3) A PI of 0 is assumed for "Very Low" to "Low" expansive fill soil. Refer to Sections 6.2.3.a) and 6.2.6 for applicable "Low" expansive fill depths.

For structural design of concrete slabs, a modulus of subgrade reaction ("k-value") on the order of 80 pounds per square inch per inch ("psi/in") and an allowable bearing capacity of 800 psf may be used for slab constructed on recompacted site soils. For slabs supported by two (2) feet of approved "Very Low" to "Low" expansive fill (EI ≤ 50) soils, a k-value on the order of 100 psi/in, and an allowable bearing capacity of 900 psf may be used. Interior and exterior slabs should be properly designed and reinforced for the construction and service loading conditions, as well as considering the soil expansion potential. To minimize slab distress due to soil expansion, geotechnically, it would be prudent to provide a minimum actual slab thickness of four and one-half (4.5) inches with minimum reinforcement consisting of number 4 reinforcing bars spaced maximum 18 inches on centers each way for slabs supported by subgrade soils prepared as per recommended in Sections 6.2.3 or 6.2.6 above. The structural details, such as slab thickness, concrete strength, amount and type of reinforcements, joint spacing, etc., should be established by the Structural Consultant in accordance with pertinent sections in 2016 CBC.

The entirety of any new slabs within the Buildings should be underlain by an impermeable vapor barrier (minimum 10-mil-thick visqueen) per 2016 CRC Section R506.2.3. A minimum 12-inch overlap between visqueen sheets should be ensured during placement. All visqueen sheets should be puncture free prior to slab construction, and should be sandwiched top and bottom by two (2) inches of clean sand (Sand Equivalent, SE, ≥ 30 per ASTM D2419-14 Test Method. Alternatively, the sand layers may be eliminated with the use of Stego Wrap 15-mil Class A Vapor Barrier. If the City of Carson is requiring compliance with the latest CALGreen Code, then the entire slab within the Buildings should be underlain by an impermeable vapor barrier (minimum 15-mil -thick visqueen) per 2016 CRC Section R506.2.3. The interior slab should be placed in direct contact with the vapor barrier, underlain by a minimum of 4 inches of clean sand or aggregate as a capillary break. The concrete slab shall consist of a concrete mix design which will address bleeding, shrinking and curling.

Exterior slabs should be properly jointed to limit the number of concrete shrinkage cracks. For long/thin sections, such as sidewalks, expansion or control joints should be provided at spacing intervals equal to the width of the section. Slabs between 5 and 10 feet in minimum dimension should have a control joint at centerline. Slabs greater than 10 feet in minimum dimension should have joints such that unjointed sections do not exceed 10 feet in maximum dimension. Where flatwork adjoins structures, it is recommended that a foam joint or similar expansion material be utilized. Joint depth and spacing should conform to the ACI recommendations. It is, however, cautioned that uneven heaving of exterior slabs may develop in the future when prolonged irrigation or seepage permeates the subgrade soil, especially in areas that expansive soil pockets exist due to inadequate control or inspection of earthwork construction.

6.5 Asphaltic Concrete (AC) Flexural Pavement Design

For preliminary pavement design purposes, a laboratory tested R-Value of 5 has been utilized considering the site soils as subgrade soils. Three (3) traffic indices ("TI") of 4.5, 5.5 and 7.0, together with the tested R-Value, have been utilized for the development of preliminary recommendations for the pavement sections. Analyses performed in accordance with the current edition of the Caltrans Highway Design Manual, and assuming compliance with site preparation recommendations, it is recommended that the AC pavement structural sections tabulated on next page be considered. However, please be reminded that the following preliminary pavement section recommendations have been established based purely on procedures stipulated in Caltrans Manual. Governing authority should be consulted for minimum pavement section requirements and, if more stringent than that recommended by ASE, be complied with.

Traffic Index (TI)	Pavement Section Alternatives		Remark
	AC [1] (inches)	AB [2] (inches)	
4.5	3.0	8.5 / 5.0 [3]	For auto parking stalls.
	4.0	6.0 / 4.0 [3]	
5.5	3.0	12.0 / 8.0 [3]	For auto circulation aisles.
	4.0	10.0 / 6.5 [3]	
7.0	5.0	13.5 / 9.0 [3]	For fire lanes and truck access ways/entry and exits.
	6.0	11.5 / 7.5 [3]	

(1) Asphaltic Concrete.
(2) CAB or CMB, Greenbook sections 200-2.2 and 200-2.4, respectively, compacted to at least 95% relative compaction.
(3) Base reinforced by placement of a layer of geogrid (Tensar BX-1100, Mirafi BXG-11 or equivalent) at bottom of base layer.

It is recommended that R-Value testing be performed on representative soil samples after rough grading operations on the upper 2 feet to confirm/modify applicability of the above pavement sections.

The aggregate base should conform to the Crushed Aggregate Base (CAB) per Section 200-2.2 of the Greenbook requirements. The base course should be compacted to a minimum relative compaction of 95% at a minimum of one (1) percentage point above the optimum moisture content. Field testing should be used to verify compaction, aggregate gradation, and compacted thickness.

The asphalt concrete pavement should be compacted to 95% of the unit weight as tested in accordance with the Hveem procedure. The asphalt concrete material shall conform to Type III, Class C2 or C3, of the Greenbook. All subgrade and aggregate base materials should be proof-rolled by heavy rubber tire equipment to verify that the subgrade and base grade are in a non-yielding condition.

If the paved areas are to be used during construction, or if the type and frequency of traffic is greater than assumed in the design, the pavement section should be re-evaluated for the anticipated traffic.

6.6 Portland Cement Concrete (PCC) Pavements

The concrete pavement sections tabulated on next page are based on load safety factors of 1.0 and 1.1, and a modulus of subgrade reaction ("k" Value) of 80 pounds per cubic inch for site soils compacted as subgrade material, and the design procedures presented in the Portland Cement Association bulletin "Thickness Design for Concrete Highway and Street Pavements" (EB109.01P), 1984. A design service life of 20 years was assumed for the design of the Portland cement concrete pavement section.

The Structural Consultant should establish the design details of the concrete pavement section, including reinforcements, concrete strength, and joint and load transfer requirements.

Concrete Flexural Strength (psi) (1)	Pavement Thickness (in) (2), (4)	Pavement Thickness (in) (3), (4)
600	6.0	7.0
650	5.5	6.5

(1) Represents 90-day flexural strength. Based on Figure 10 of Reference 18, concrete with 28-day unconfined compressive strength values of 4000 to 4500 psi typically correlates to 90-day flexural strength values of 600 and 650 psi, respectively.
(2) Load Safety Factor = 1.0 (Auto Parking Stalls)
(3) Load Safety Factor = 1.1 (Fire Lanes/Truck Traffic Areas/Entry and Exits)
(4) Assumes no PCC shoulder or curb.

It is recommended that edges of concrete pavements which are not adjacent to existing buildings, or are adjacent to planter areas, be downturned a minimum of 12 inches or be constructed with curbing to prevent water infiltration to subgrade soils. If edges are downturned or curbing is constructed, the above pavement thicknesses should be decreased by 1/2 inch.

Subgrade soils should exhibit a firm, unyielding surface in addition to the recommended compaction. Final compaction and testing of pavement subgrade should be performed just prior to placement of aggregate base and/or concreting. Other pertinent subgrade preparation measures stipulated in the "Thickness Design for Concrete Highway and Street Pavements" (EB109.01P), 1984, or required by the jurisdictional municipal authorities should be followed accordingly.

6.7 Site Drainage

Per Section 1804.4 of 2016 CBC, a minimum 5% descending gradient away from the Buildings for a minimum distance of 10 feet should be incorporated for earth grade placed adjacent to the foundation. This descending gradient may be reduced to 2% for any impervious areas, such as concrete paved walkways, within the 10-foot zone. For areas where the 10-foot drainage distance is not attainable, alternative measure such as concrete-lined swales having a minimum 2% gradient may be adopted to divert the water away from the Buildings, provided that a minimum 5% gradient is maintained in the distance between the building footprint and the diversion measure such as swales. For more specific site drainage guidelines, the Project Civil Consultant should refer to the pertinent sections in 2016 CBC.

Any planter areas to be placed adjacent to structure perimeters should be provided with impervious bottoms and a drainage pipe, or should be planted with drought-tolerant plants, to divert water away or minimize moisture infiltration from foundation and slab subgrade soils. Excessive moisture variations in site soils could result in significant volume changes and movement.

6.8 Soil Corrosivity Evaluation

Soils corrosivity tests were performed on a representative sample of site soil. These tests are meant to determine the corrosive potential of on-site soils to proposed concrete foundations/flatwork and underground metal conduit. The soils corrosivity test results are presented in Appendix A.

6.8.1 Concrete Corrosion:

Disintegration of concrete may be attributed to the chemical reaction of soils sulfates and hydrated lime and calcium aluminate with the cement. The severity of the reaction resulting in expansion and disruption of the cement is primarily a function of the concentration of soluble sulfates and the water-cement ratio of the concrete.

A soluble sulfate content of 0.060% by weight has been recorded from testing per California Test Method (CTM) 417 conducted on on-site soils, as indicated in Appendix A. As per Table 4.2.1 of ACI 318-14, soils exhibiting soluble content less than 0.1% by weight are classified as having “Not Applicable” sulfate exposure and “S0” sulfate exposure category. As such, for structural features to be in direct contact with on-site soils, the requirements regarding type of Portland cement or water cement ratio pertinent to the tested “S0” sulfate exposure category as per stipulated in Table 4.3.1 of ACI 318-14 should be followed.

6.8.2 Metal Corrosion:

In the evaluation of soil corrosivity to metal, the hydrogen ion concentrates (pH) and the electrical resistivity of the site and backfill soils are the principal variables in determining the service life of ferrous metal conduit. The pH of soil and water is a measure of acidity or alkalinity, while the resistivity is a measure of the soils resistance to the flow of electrical current. Currently available design charts indicate that corrosion rates decrease with increasing resistivities and increasing alkalinities. It can also be noted that for alkaline soils, the corrosion rate is more influenced by resistivity than by pH.

The tested resistivity value of 660 ohm-cm per ASTM G187-12a Test Method coupled with a pH-value of 8.15 per CTM 643, classifies the on-site soils tested to be very corrosive to buried ferrous metals. Based on CTM 643, the year to perforation for 18-gauge steel in contact with soils of similar resistivities and pH-values is <u>21</u> years for the very corrosive soils. In lieu of additional testing, alternative piping materials, i.e. plastic piping, may be used instead of metal if longer service life is desired or required. The resistivity values of on-site soils may also have implications to other building materials and depths of embedment for steel reinforcement etc. Therefore it might be desirable that a qualified corrosion consultant be engaged to review the building plans.

A soluble chloride content of 21 ppm was recorded in our laboratory tests per CTM 422. Per Caltrans guidelines and specifications (References 16 and 17), soils exhibiting soluble chloride contents exceeding 500 ppm are considered “corrosive”. The soils are thus classified as “non-corrosive” per Caltrans criterion. In addition, special measure in terms of rebar protection against chloride corrosion under Exposure Class “C0” stipulated in Tables 4.2.1 and 4.3.1 of ACI 318-14 may be required as a result of the soluble chloride content tested.

6.9 Utility Trenches

All trenches should be backfilled with approved fill material compacted to relative compaction of not less than 90 percent per ASTM D1557-12 Test Method. Care should be taken during backfilling to prevent utility line damage.

The on-site soils may be used for backfilling utility trenches from one (1) foot above the top of pipe to the surface, provided the material is free of organic matter and deleterious substances. Any soft and/or loose materials or fill encountered at pipe invert should be removed and replaced with properly compacted fill or adequate bedding material.

Site soils are <u>not</u> considered to be suitable for bedding or shading of utilities. Imported soils for pipe bedding should consist of non-expansive granular soils. Bedding materials should consist of sand with a SE value not less than <u>30</u>.

If sandy soils are used for trench backfill, the backfill should be topped with a minimum 2-foot thick cap of compacted fine-grained, cohesive soil. Also, a minimum 10-foot length of trench at the entrance and exit points of buildings should be backfilled with fine-grained soils to serve as a plug to prevent water migration into structure foundation support zones.

The walls of temporary construction trenches may not be stable when excavated nearly vertical due to the potential for caving. Shoring of excavation walls or flattening of slopes will be required if excavation depths greater than 4 feet are necessary. Trenches should be located so as not to impair the bearing capacity of soils or cause settlement under foundations. As a guide, trenches parallel to foundations should be clear of a 45-degree plane extending outward and downward from the edge of the foundations. All work associated with trenches, excavations and shoring must conform to the State of California Safety Code (CAL-OSHA).

6.10 Plan Review, Observations and Testing

Once foundation and grading plans are completed, they should be forwarded to the Geotechnical Consultant for review of conformance with the intent of these recommendations and criteria presented in the pertinent sections of this report.

All excavations should be observed by a representative of this office to verify minimum embedment depths, competency of bearing soils and that the excavations are free of loose and disturbed materials. Such observations should be made prior to placement of any fill, reinforcing steel or concrete. All grading and fill compaction should be performed under the observation of and testing by a Geotechnical Consultant or his representative.

7.0 FIELD PERCOLATION TEST DATA

Initial observations in the borings after overnight pre-soaking (i.e. water remains in boring) indicated the time interval between readings should be 30 minutes. The percolation tests were performed using the County of Los Angeles Public Works Publication GS200.1 procedures modified to test the cross sectional zone of typical soils within the level of anticipated storm water infiltration (e.g. approximately 1 foot to 5 feet below existing grade for Boring B-6, and 5 feet to 10 feet below existing grade for Boring B-5).

Field percolation testing was conducted on July 10, 2018. Stabilized field percolation test data indicates preadjusted percolation test rates of approximately 8.0 and 120.0 minutes per inch (mpi) for clean water at the locations of Borings B-5 and B-6, respectively. Field percolation test data is presented on Plates F-1 and F-2 attached in Appendix A of this Soils Report.

Tabulated below are the results of percolation testing conducted at the locations of Borings B-5 and B-6, including the infiltration rates derived from the procedures outlined in L.A. County Publication GS200.1.

	Percolation Test Rate	**Infiltration Rate***
Boring No.	**(Minutes/Inch)**	**(Inches/Hour)**
B-5	8.0	0.46
B-6	120.0	0.018

*Infiltration Rate for vertical flow derived from L.A. County Public Works Publication GS200.1 procedures for Boring Percolation Testing.

The rates presented above are anticipated to be the fastest rates that can be absorbed by the site soils at the respective boring locations. However, with time and depending on the degree of saturation of soils and other factors, the percolation rates may reduce which is typical for sewage disposal or stormwater dispersal fields.

Please be informed that during installation of on-site storm water dispersal system, the following factors should be noted:

- The degree of compactive effort in the upper 1 to 1.5 feet of soils above any filter material should be between 90 and 92 percent relative compaction. As any greater compactive efforts in the soil strata of water retention system construction may cause the percolation rates to reduce substantially, it is not advisable to impose significant structural loading in these areas, from a geotechnical viewpoint.
- The rate of water transmission from the filter material to the soil will be limited the porosity characteristics of the fabric wrap around the filter material.

8.0 CLOSURE

This report has been prepared for the exclusive use of **Brandywine Homes** (the Client) and their design consultants for use in the design and construction of the proposed new residential development (the

Buildings) at 1007 East Victoria Street in the City of Carson. The report has not been prepared for use by other parties, and may not contain sufficient information for purposes of other parties.

The Client or their representatives are responsible for ensuring the information and recommendations contained in this report are brought to the attention of the project engineers and architects, incorporated into the project plans, and implemented by project contractors. This report should be reflected on project grading plans as a part of the project specifications.

ASE requests and recommends proper notification from the Client should any of the following occur:

1. Final plans for site development indicate utilization of areas not originally proposed for construction.
2. Structural loading conditions vary from those utilized for evaluation and preparation of this report.
3. The Site is not developed within 12 months following the date of this report.
4. Change of ownership of property occurs that would render the property development indicated in this report irrelevant or significantly different.

If changes or delays do occur, this office should be notified and provided with finalized plans of site development for our review to enable us to provide the necessary recommendations for additional work and/or updating of the report. Any charges for such review and necessary recommendations would be at the prevailing rate at the time of performing review work.

The findings contained in this report are based upon our evaluation and interpretation of the information obtained from the limited number of test borings and the results of laboratory testing and engineering analysis. As part of the engineering analysis it has been assumed, and is expected, that the geotechnical conditions existing across the area of study are similar to those encountered in the test excavations. However, no warranty is expressed or implied as to the conditions at locations or depths other than those excavated. Should conditions encountered during construction differ significantly from those described in this report, this office should be contacted immediately for recommendations prior to continuation of work.

Our findings and recommendations were obtained in accordance with generally accepted current professional principles and local practice in geotechnical engineering and reflect our best professional judgment. We make no other warranty, either express or implied.

These recommendations are, however, dependent on the aforementioned assumption of uniformity and upon proper quality control of engineered fill and foundations. Geotechnical observations and testing should be provided on a continuous basis during grading at the site to confirm preliminary design assumptions and to verify conformance with the intent of our recommendations. If parties other than ASE are engaged to provide geotechnical services during construction, they must be informed that they will be required to assume complete responsibility for the geotechnical phase of the project by either concurring with the recommendations in this report or providing alternative recommendations.

This concludes our scope of services as indicated in ASE's proposal dated June 7, 2018, however, our report is subject to review by the controlling authorities for the project. Any further geotechnical services that may be required of our office to respond to questions/comments of the controlling authorities after their review of the report will be performed on a time-and-expense basis as per our current fee schedule. We would not proceed with any response to report review comments/questions without authorization from your office.

We at ASE appreciate your business and are prepared to assist you with construction-related services.

APPENDIX A

The following Appendices contain the substantiating data and laboratory test results to complement the engineering evaluations and recommendations contained in the report.

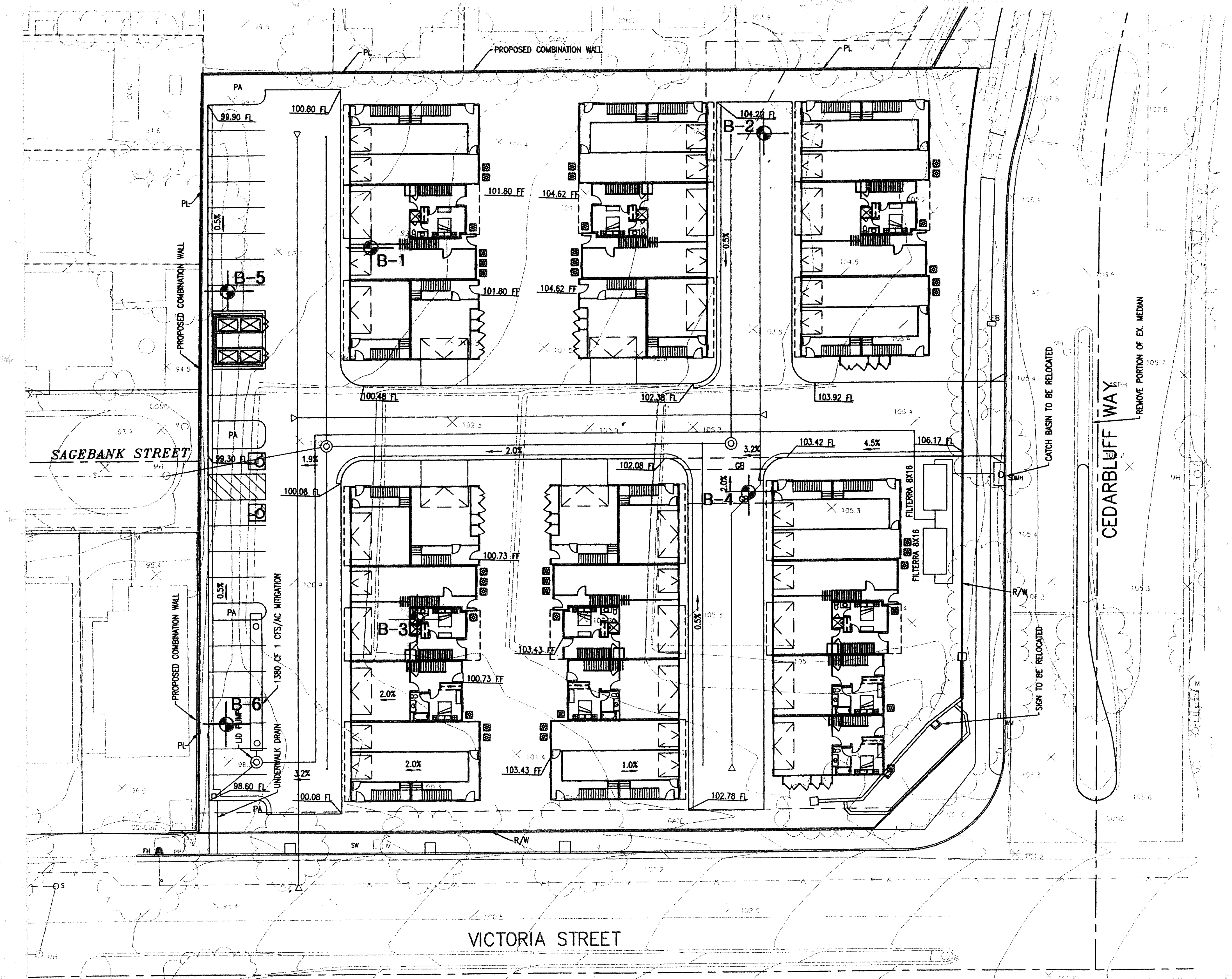
Site Exploration

On July 9, 2018, field explorations were performed by drilling six (6) test borings at the approximate locations indicated on the attached Boring Location Plan, Plate A. The exploratory borings were drilled by Choice Drilling, Inc., utilizing a truck mounted, CME75 rotary drilling rig equipped with 8-inch diameter continuous flight, hollow-stem rotary augers. The borings extended to depths of 5 feet 11 inches to 35 feet 6 inches from the existing grades.

Continuous observations of the materials encountered in the borings were recorded in the field. The soils were classified in the field by visual and textural examination and these classifications were supplemented by obtaining bulk soil samples for future examination in the laboratory. Relatively undisturbed samples of soils were extracted in a Modified California barrel sampler lined with 2.416-inch diameter by one-inch high rings and tipped with tapered cutting shoe. Additional samples were obtained in a Standard Penetration sampler in accordance with specifications outlined in ASTM D1586-11 Test Method. All samples were secured in moisture-resistant bags immediately after retrieval from exploratory boring to minimize the loss of field moisture, followed by timely transportation to ASE's laboratory for ensuing testing. Upon completion of exploration, the borings were backfilled with excavated materials and compacted by tamping.

Description of the soils encountered, depth of samples, field density and moisture content of tested samples, respective laboratory tests performed, as well as Standard Penetration Test ("N" Values) and Modified California barrel sampler blow counts are presented in the attached Field Logs of Borings ("B" Plates).

Plate A	Boring Location Plan
Plates B-1 through B-6	Field Logs of Borings



Approximate Scale: 1"= 37.5'

Denotes Approximate Boring Location

SOURCE:CONCEPT CIVIL PLAN,Sheet 1 of 2,prepared by KES Technologies Inc.,undated.

Associated Soils Engineering, Inc.	BORING LOCATION PLAN	Project# 6816.18	Plate A

FIELD LOG OF BORING B-1

ASSOCIATED SOILS ENGINEERING, INC.

Project:	*Proposed New Residential Development-Carson*
Location:	*1007 East Victoria Street*
Project No.	*6816.18*

Dates(s) Drilled:	**7/9/2018**	Logged By:	**Gary L. Martin**
Drilled By:	**Choice Drilling,Inc.**	Total Depth:	**25 Feet**
Rig Make/Model:	**CME 75**	Hammer Type:	**Automatic**
Drilling Method:	**Hollow-stem Auger**	Hammer Weight/Drop:	**140 Lb./±30 In.**
Hole Diameter:	**8 Inches**	Surface Elevation:	**N/A**

Comments: Groundwater not encountered. Backfill not determined.

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: BULK / DRIVE / TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0			SM	SILTY SAND WITH CLAY: FILL-Light olive brown,damp,fine-grained sand,with gravel				
		20/6" (Ring)				114.6	4.3		
		25/6" (Ring)		CL	SILTY CLAY WITH SAND: FILL-Dark brown with yellowish brown,moist,fine-grained sand,with some gravel and asphaltic concrete (AC) pieces	123.9	9.2		
5	5	24/6" (Ring)		CL	SILTY CLAY: NATIVE-Dark yellowish brown,moist	120.0	13.4		
		30/6" (Ring)		ML	CLAYEY SILT: Brown to yellowish brown,moist	118.0	13.2		CONSOL,SHEAR
		11(SPT)		ML	same as above,yellowish brown to light olive brown		19.1		
10	10	15/6" (Ring)		ML	same as above,light olive brown,very moist	99.3	21.2		
		21/6" (Ring)		ML	SANDY SILT: Pale olive,trace yellowish brown,moist,fine-grained sand	113.2	17.3		
				SP	SAND: Light gray with brownish yellow,damp,fine-grained sand				
15	15	85(Ring)				101.2	3.1		CONSOL

FIELD LOG OF BORING B - 1

ASSOCIATED SOILS ENGINEERING, INC.

Project: ***Proposed New Residential Development-Carson***

Location: ***1007 East Victoria Street***

Project No. ***6816.18***

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: BULK	SAMPLE INTERVALS: DRIVE	TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
						SM	SILTY SAND: Pale olive with yellowish brown,damp,fine-grained sand				
20	20			20(SPT)					4.7		
				75(Ring)		SM	same as above,pale olive	102.1	5.3		
25	25										

FIELD LOG OF BORING B-2

ASSOCIATED SOILS ENGINEERING, INC.

Project:	*Proposed New Residential Development-Carson*
Location:	*1007 East Victoria Street*
Project No.	*6816.18*

Dates(s) Drilled:	**7/9/2018**	Logged By:	**Gary L. Martin**
Drilled By:	**Choice Drilling,Inc.**	Total Depth:	**35 Feet 6 Inches**
Rig Make/Model:	**CME 75**	Hammer Type:	**Automatic**
Drilling Method:	**Hollow-stem Auger**	Hammer Weight/Drop:	**140 Lb./±30 In.**
Hole Diameter:	**8 Inches**	Surface Elevation:	**N/A**

Comments: Groundwater not encountered. Backfill not determined.

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0			CL	SILTY CLAY WITH SAND: FILL-Brown to grayish brown,with light olive brown,damp to moist,fine-grained sand,with some gravel				R-VALUE
		72(Ring)		CL		127.3	8.1		
5	5	38(Ring)		CL	same as above,becoming brown with light olive brown,pale yellow and brownish yellow	122.5	11.7		CONSOL,SHEAR
		15(SPT)		ML	CLAYEY SILT WITH SAND: NATIVE-Yellowish brown with light yellowish brown,moist,fine-grained sand		18.3		
10	10	9/6"(Ring)		CL	SILTY CLAY: Yellowish brown,moist	103.1	18.3		
		16/6" (Ring)		ML	CLAYEY SILT WITH SAND: Yellowish brown to light olive brown,moist,fine-grained sand	113.7	16.1		
				ML	SANDY SILT: Yellowish brown and pale olive,moist,fine-grained sand				
15	15	17/6" (Ring)				89.6	20.4		
		32/6" (Ring)		ML	SANDY SILT WITH CLAY: Pale olive,moist,fine-grained and	104.3	19.4		

SOILS ENGINEERING, INC.

FIELD LOG OF BORING B - 2

Project: ***Proposed New Residential Development-Carson***

Location: ***1007 East Victoria Street***

Project No. ***6816.18***

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
			SP	SAND: Light gray with light olive brown,damp,fine-grained sand				
20	20	90/11" (Ring)			100.8	3.1		
			SM	SILTY SAND: Light gray,pale olive and light olive brown,damp,fine-grained sand,with thin layers Sand,Silt and Sandy Silt				
25	25	11/6" (SPT)				6.6		
		50/6" (SPT)	ML	SANDY SILT: Pale olive with brownish yellow,moist,fine-grained sand,trace clay		21.2		
			SM	SILTY SAND: Pale olive to pale yellow,damp,fine-grained sand				
30	30	35/6" (Ring)			94.2	4.9		
		50/5" (Ring)	SM	SANDY SILT: Pale olive,moist,very fine-grained sand,with some gravel-size particles	102.4	9.5		
			ML	SILT WITH CLAY: Light olive brown,very moist				
		16/6" (SPT)				33.3		
35	35	31/6" (SPT)	ML	CLAYEY SILT: Olive,trace yellowish brown,moist		25.0		

FIELD LOG OF BORING B-3

ASSOCIATED SOILS ENGINEERING, INC.

Project:	***Proposed New Residential Development-Carson***
Location:	***1007 East Victoria Street***
Project No.	***6816.18***

Dates(s) Drilled:	**7/9/2018**	Logged By:	**Gary L. Martin**
Drilled By:	**Choice Drilling,Inc.**	Total Depth:	**25 Feet 6 Inches**
Rig Make/Model:	**CME 75**	Hammer Type:	**Automatic**
Drilling Method:	**Hollow-stem Auger**	Hammer Weight/Drop:	**140 Lb./±30 In.**
Hole Diameter:	**8 Inches**	Surface Elevation:	**N/A**

Comments: Groundwater not encountered. Backfill not determined.

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0			ML	CLAYEY SILT WITH SAND: FILL-Light yellowish brown,dry to damp,fine-grained sand				MAX DENSITY, EXPANSION, REMOLD SHEAR, ATTERBERG LIMITS, CORROSIVITY TESTS
				CL					
		82(Ring)			SILTY CLAY WITH SAND: FILL-Brown with grayish brown,moist,fine-grained sand,with some gravel	110.6	15.6		
5	5	43(Ring)		CL	same as above,with AC and concrete pieces	126.6	10.3		
		18(SPT)		ML	CLAYEY SILT: NATIVE-Yellowish brown,moist		14.3		
10	10	47(Ring)		ML	same as above	112.6	14.9		CONSOL
				ML	SILT: Pale olive,moist				
15	15	30/6" (Ring)				96.4	18.8		
		40/6" (Ring)		SM	SILTY SAND: Light yellowish brown,damp to moist,fine-grained sand	113.8	7.9		
20	20								

ASSOCIATED SOILS ENGINEERING, INC.

FIELD LOG OF BORING B - 3

Project: ***Proposed New Residential Development-Carson***

Location: ***1007 East Victoria Street*** | Project No. ***6816.18***

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: BULK	SAMPLE INTERVALS: DRIVE	TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
20	20		■	76/11" (Ring)		SM	SILTY SAND: Pale olive and brownish yellow to light gray with brownish yellow,moist,fine-grained sand	101.7	10.5		
			■	59(SPT)		SM	same as above,pale olive with yellowish brown,with layer Fine Sandy Silt (ML)		9.3		
25	25										

ASSOCIATED SOILS ENGINEERING, INC.

FIELD LOG OF BORING B-4

Project:	*Proposed New Residential Development-Carson*
Location:	*1007 East Victoria Street*
Project No.	*6816.18*

Dates(s) Drilled:	**7/9/2018**	Logged By:	**Gary L. Martin**
Drilled By:	**Choice Drilling,Inc.**	Total Depth:	**35 Feet**
Rig Make/Model:	**CME 75**	Hammer Type:	**Automatic**
Drilling Method:	**Hollow-stem Auger**	Hammer Weight/Drop:	**140 Lb./±30 In.**
Hole Diameter:	**8 Inches**	Surface Elevation:	**N/A**

Comments: Groundwater not encountered. Backfill not determined.

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: BULK	SAMPLE INTERVALS: DRIVE	TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0					ML	CLAYEY SILT WITH SAND: FILL-Light yellowish brown to brown,damp,fine-grained sand,with some gravel				
				56(Ring)				110.3	7.0		CONSOL,SHEAR
						CL	SILTY CLAY: FILL-Brown to dark brown,moist,with some gravel,with AC pieces				
5	5			43(Ring)				122.9	11.1		
10	10			7/6"(Ring)		CL	same as above,brown with yellowish brown	112.6	15.0		
				11/6" (Ring)		ML	CLAYEY SILT: NATIVE-Yellowish brown to light olive brown,moist	106.1	19.0		
15	15			55(Ring)		ML	SILT WITH SAND: Light olive brown,trace yellowish brown,moist,fine-grained sand,trace clay	109.8	14.9		
20	20										

FIELD LOG OF BORING B - 4

ASSOCIATED SOILS ENGINEERING, INC.

Project: ***Proposed New Residential Development-Carson***

Location: ***1007 East Victoria Street*** | Project No. ***6816.18***

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
20	20	27(SPT)		SM	SILTY SAND: Light gray with light olive brown,damp,fine-grained sand		3.1		
25	25	15/6" (Ring)		SM	same as above	88.1	6.5		
		50/6" (Ring)		ML	SILT: Pale olive and light olive brown,moist *insufficient sample for density	*	18.1		
				SM	SILTY SAND: Pale olive,damp,fine-grained sand				
30	30	22(SPT)					6.4		
				ML	CLAYEY SILTSTONE: Light olive brown to pale yellow,with yellowish brown,moist to very moist				
		92/11" (Ring)				90.0	31.6		
35	35								

FIELD LOG OF BORING B-5

Sheet 1 of 1

ASSOCIATED SOILS ENGINEERING, INC.

Project: *Proposed New Residential Development-Carson*

Location: *1007 East Victoria Street* | Project No. *6816.18*

Dates(s) Drilled:	**7/9/2018**	Logged By:	**Gary L. Martin**
Drilled By:	**Choice Drilling,Inc.**	Total Depth:	**11 Feet 3 Inches**
Rig Make/Model:	**CME 75**	Hammer Type:	**Automatic**
Drilling Method:	**Hollow-stem Auger**	Hammer Weight/Drop:	**140 Lb./±30 In.**
Hole Diameter:	**8 Inches**	Surface Elevation:	**N/A**

Comments: Groundwater not encountered. Backfill not determined.

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	OTHER TESTS
0	0		ML	CLAYEY SILT WITH SAND: FILL-Light yellowish brown,damp,fine-grained sand,with some gravel			
		53(Ring)	CL	SILTY CLAY WITH SAND: FILL-Pale brown to brown with light yellowish brown,damp to moist,fine-grained sand,with some gravel,with paper pieces	122.0	8.7	
5	5	36(Ring)	CL	same as above	121.3	12.4	
			ML	CLAYEY SILT: NATIVE-Light yellowish brown,moist			
		17(SPT)	ML	CLAYEY SILT: Pale olive,trace yellowish brown,moist,with layers Fine Sandy Silt		22.0	
10	10						
				NOTE:Boring backfilled to 10 feet 3 inch (10' 3") depth with gravel mixed with some site soils. Ten feet three inch (10' 3") length of 6-inch I.D. slotted PVC pipe (two pieces:upper 5 feet 3 inches solid,lower 5 feet slotted) placed in boring with the annular area backfilled with pea gravel to surface. Percolation test performed after overnight presoaking.			
15	15						

SOILS ENGINEERING, INC.

FIELD LOG OF BORING B-6

Sheet 1 of 1

Project: *Proposed New Residential Development-Carson*

Location: *1007 East Victoria Street* | Project No. *6816.18*

Dates(s) Drilled:	**7/9/2018**	Logged By:	**Gary L. Martin**
Drilled By:	**Choice Drilling,Inc.**	Total Depth:	**5 Feet 11 Inches**
Rig Make/Model:	**CME 75**	Hammer Type:	**Automatic**
Drilling Method:	**Hollow-stem Auger**	Hammer Weight/Drop:	**140 Lb./±30 In.**
Hole Diameter:	**8 Inches**	Surface Elevation:	**N/A**

Comments: Groundwater not encountered. Backfill not determined.

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: BULK	SAMPLE INTERVALS: DRIVE	TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0					ML	CLAYEY SILT WITH SAND: FILL-Light yellowish brown,dry to damp,fine-grained sand				
				29(Ring)		CL	SILTY CLAY WITH SAND: FILL-Brown,trace yellowish brown and grayish brown,damp to moist,medium to coarse-grained sand,with gravel		9.3		
				+		CL	same as above +moisure sample from upper 6" of SPT		12.7		
5	5			10(SPT)		CL	SILTY CLAY: NATIVE-Yellowish brown,moist		16.3		
							NOTE:Boring backfilled to 5 feet 3 inch (5' 3") depth with gravel. Five feet three inch (5' 3") length of 6-inch I.D. slotted PVC pipe (one piece) placed in boring with the annular area backfilled with pea gravel to surface. Percolation test performed after overnight presoaking.				

Laboratory Tests

After samples were visually classified in the laboratory, a testing program aimed at generating sufficient data for subsequent evaluation was established and implemented.

- Moisture Content and Density Tests

The undisturbed soil retained within the rings of the Modified California barrel sampler was tested in the laboratory to determine in-place dry density and moisture content. Test results are presented on the Field Logs of Boring, “B” Plates.

- Consolidation and Direct Shear Tests

Consolidation (ASTM D 2435-11 Test Method) and direct shear (ASTM D 3080-11 Test Method) tests were performed on selected relatively undisturbed and remolded samples to determine the settlement characteristics and shear strength parameters of various soil samples, respectively. The results of these tests are shown graphically on the appended “C” and “D” Plates.

- Atterberg Limits Tests

The Atterberg Limits (liquid limit-plastic limit and plasticity index) were determined on a selected soils sample in accordance with ASTM D4318-10 Test Method, Method A (multi-point test), dry preparation procedures. The test results are as follows and are presented on the appended “E” Plate.

Sample ID	Liquid Limit, LL (%)	Plastic Limit, PL (%)	Plasticity Index, PI	Soil Classification
B-3 @ 0-5’	40	13	27	CL

- Soil Corrosivity Tests

Tests of soluble sulfate and chloride contents were performed in accordance with CTM’s 417 and 422, respectively, to assess the degree of corrosivity of the subgrade soils with regard to concrete and normal grade steel. Resistivity and ph-value tests were performed in accordance with ASTM G187-12a Test Method and CTM 643 to assess the degree of corrosivity of the subgrade soils with regard to ferrous metal piping. The test results are shown below.

Sample ID	Sulfate Content (1) (%)/ Degree of Severity	Chloride Content (2) (ppm) / Degree of Severity	Resistivity (3) (OHM-cm)/ Degree of Corrosivity	Ph-Value(4)
B-3 @ 0-5’	0.060/Not Applicable	21/Not Applicable	660/Very Corrosive	8.15

(1) California Test Method 417. (2) California Test Method 422. (3) ASTM G187-12a Test Method. (4) California Test Method 643.

<u>Laboratory Tests</u> – continued

- <u>Maximum Dry Density/Optimum Moisture Content Test</u>

A maximum density test was conducted in accordance with ASTM D1557-12, Method A, using 5 equal layers, 25 blows each layer, 10-pound hammer, 18 inch drop in a 1/30 cubic foot mold. The results are as follows:

Sample ID	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Material Classification
B-3 @ 0-5’	129.0	9.5	CL

- <u>Expansion Test</u>

An expansion test was performed on a soil sample to determine the swell characteristics. The expansion test was conducted in accordance with ASTM D4829-11 test procedures. The expansion sample was remolded to approximately 90 percent relative compaction at near optimum moisture content subjected to 144 pounds per square foot surcharge load and were saturated.

Sample ID	Molded Dry Density (pcf)	Molded Moisture Content (%)	% Saturation	Expansion Index (EI)	Expansion Classification
B-3 @ 0-5’	115.7	9.0	53.2	76	Medium

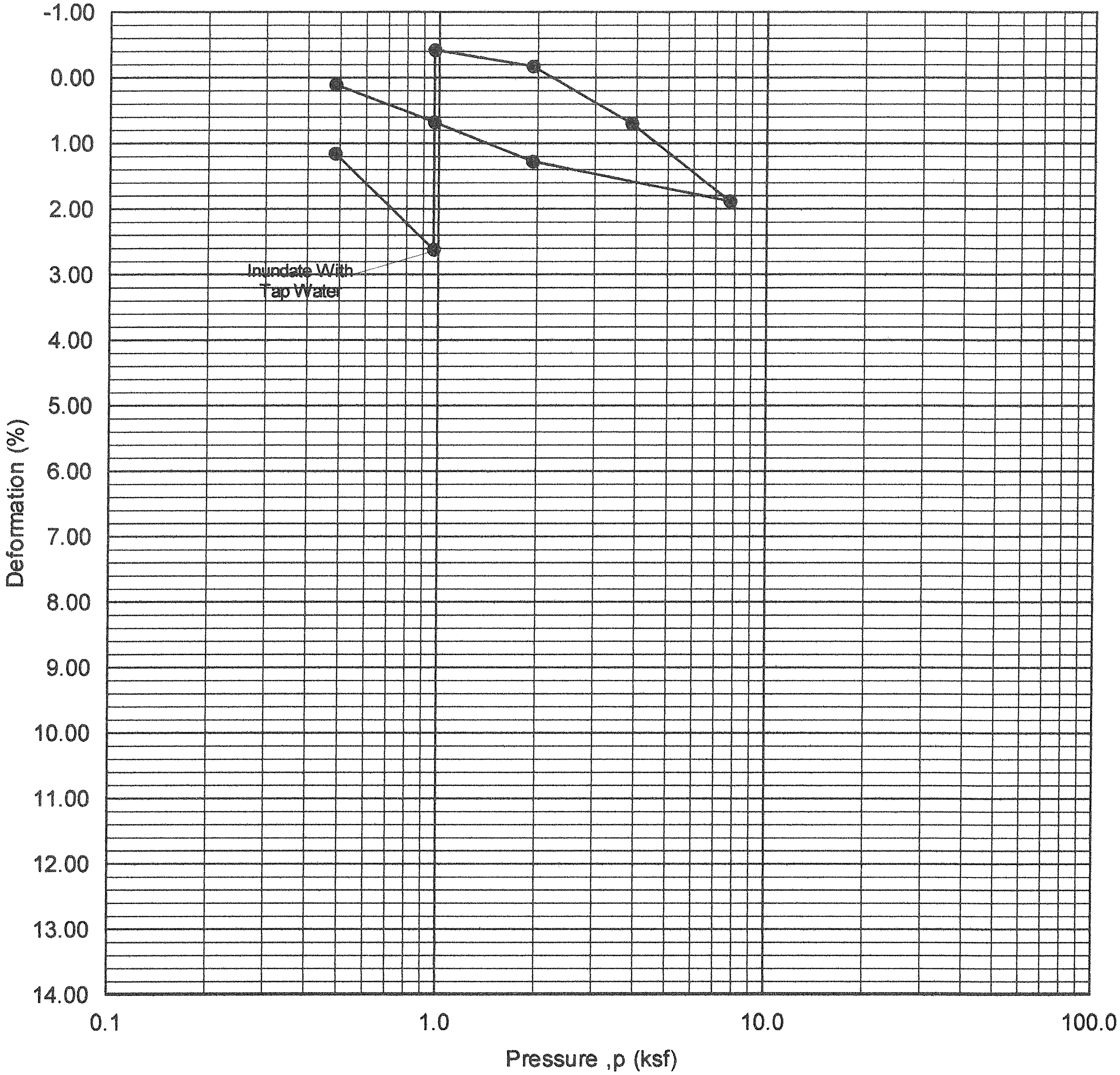
- <u>"R" Value Analysis</u>

The following "R" Value Stabilometer results were obtained in accordance with California 301 test procedures.

Stabilometer Results	Trial #1	Trial #2
Dry Density as molded, pcf	125.2	121.5
Moisture content as molded, %	11.7	13.6
Expansion Pressure, dial reading 10^4	83	25
Exudation Pressure, psi	*	*
Stabilometer "R" Value	*	*
Classification: Brown to Grayish Brown Silty Clay with Fine Sand and some Gravel		
Source: Boring B-2 @ 0-5’		

* Soil extruded out of mold before water, therefore exudation pressure was not determined. Test specification therefore assign the sample and “R” Value of **5**.

Plates C-1 through C-5	Uni-axial Consolidation Test Results
Plates D-1 through D-4	Direct Shear Test Results
Plate E-1	Atterberg Limits Test Results
Plates F-1 and F-2	Field Percolation Data Sheets

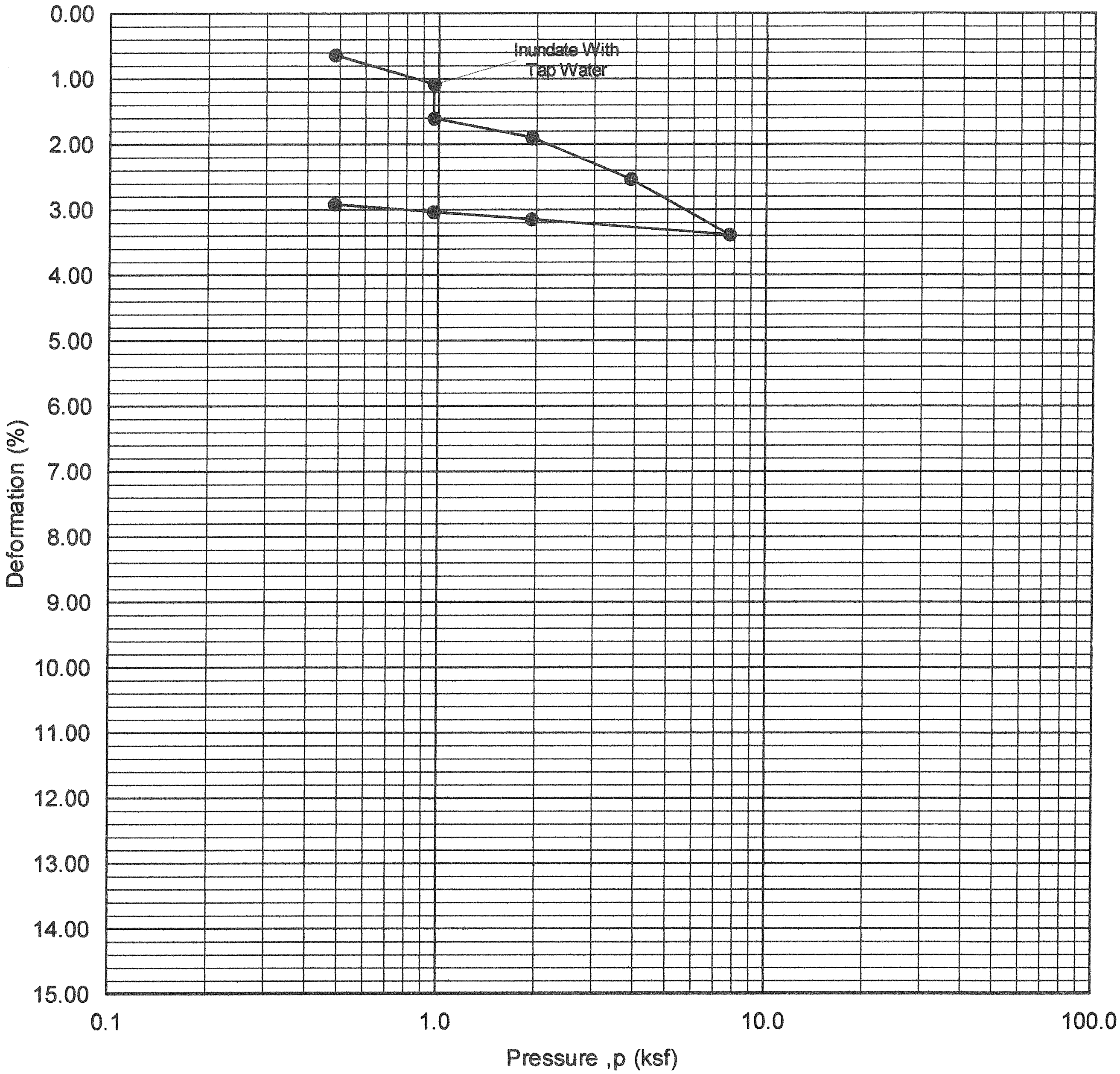


Boring No. : B-1
Depth (ft.) : 5.5
Sample Type: Clayey Silt

Dry Density (pcf) = 118.0
Moisture (%) = 13.2

Project Name:Residential Development-1007 E. Victoria Street,Carson	Project No.: 6816.18
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-1

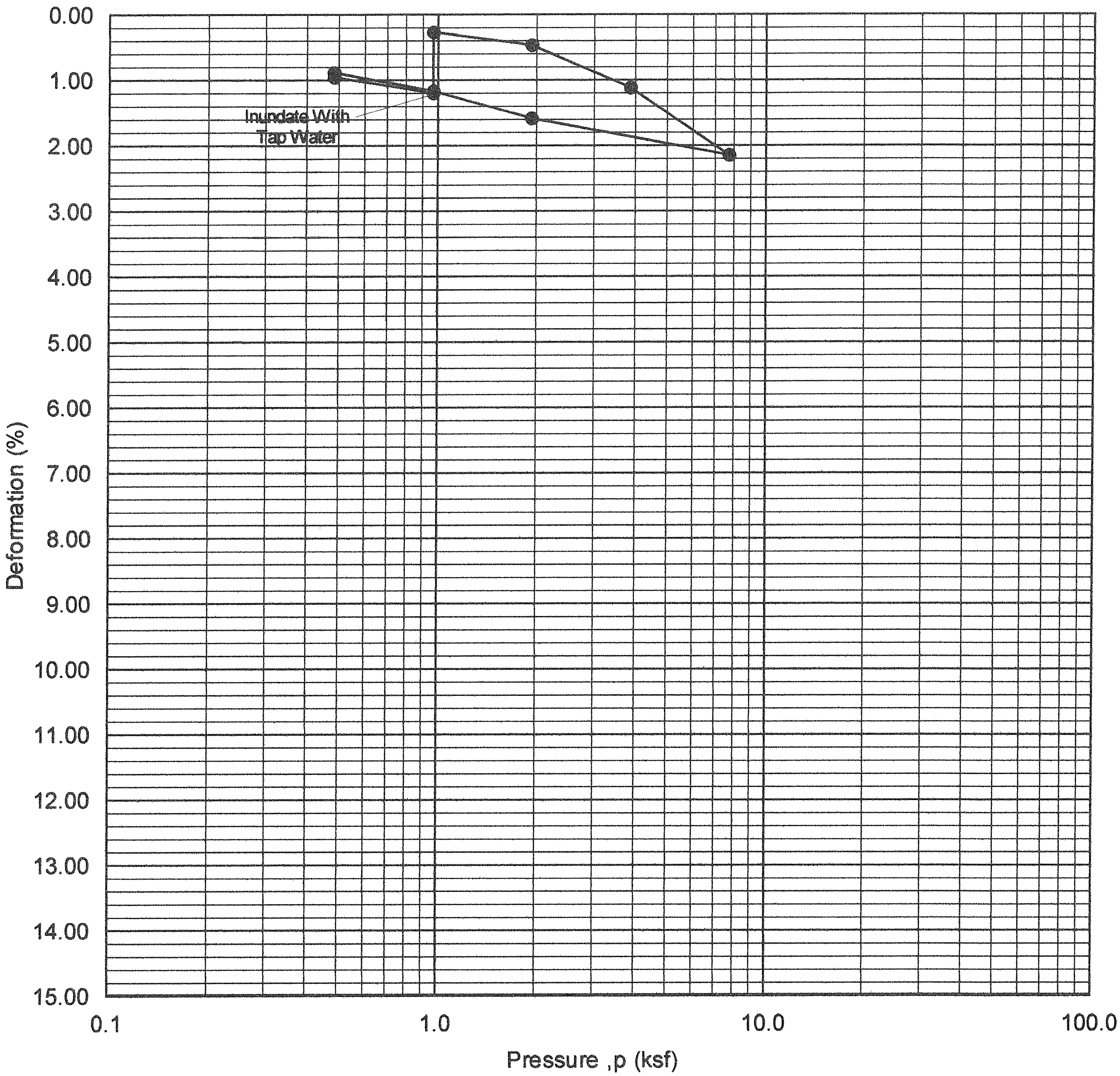


Boring No. : B-1
Depth (ft.) : 15.0
Sample Type: Fine Sand

Dry Density (pcf) = 101.2
Moisture (%) = 3.1

Project Name:Residential Development-1007 E. Victoria Street,Carson	Project No.: 6816.18
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-2

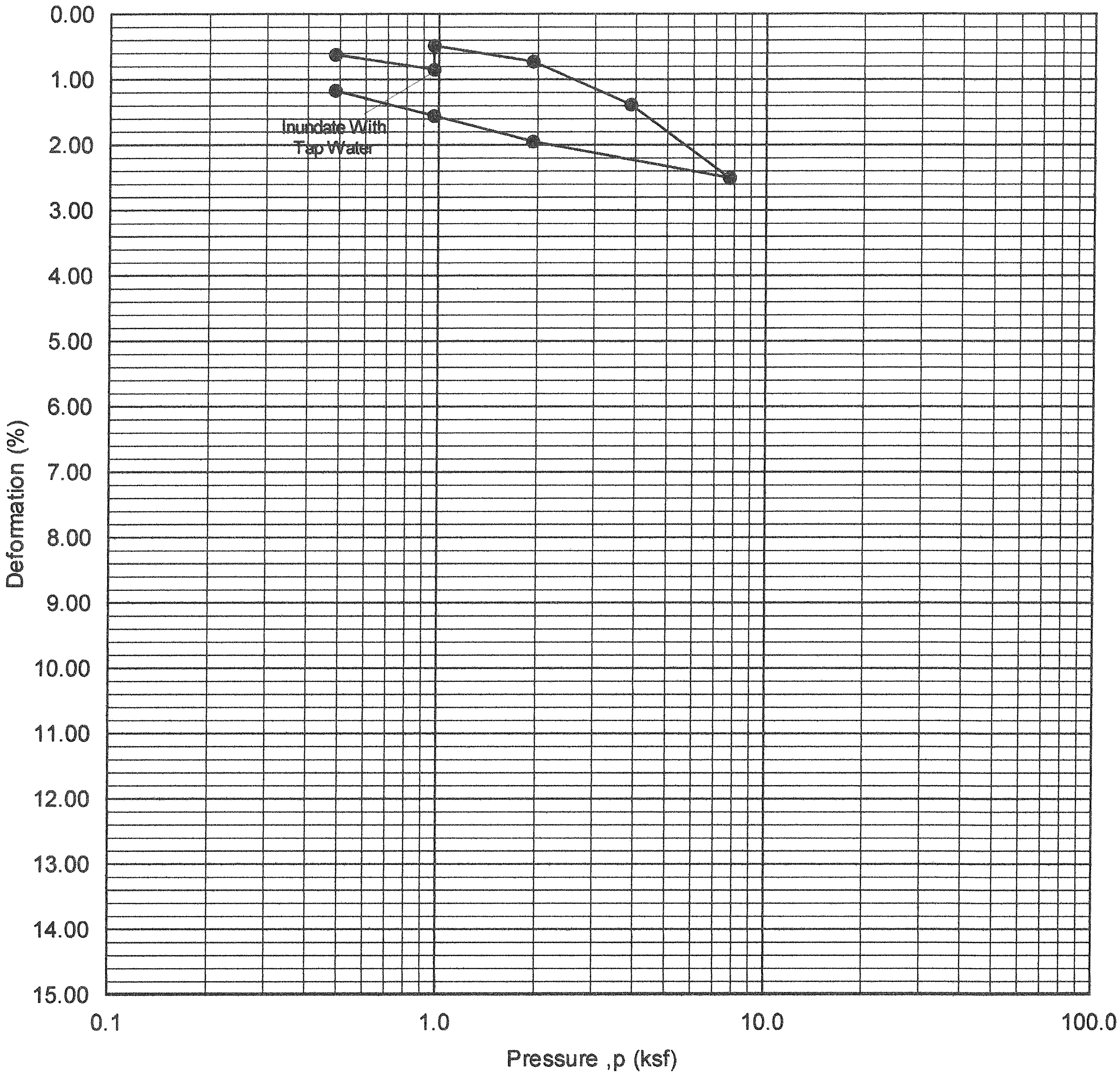


Boring No. : B-2
Depth (ft.) : 5.0
Sample Type: Silty Clay with Fine Sand

Dry Density (pcf) = 122.5
Moisture (%) = 11.7

Project Name:Residential Development-1007 E. Victoria Street,Carson	Project No.: 6816.18
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-3

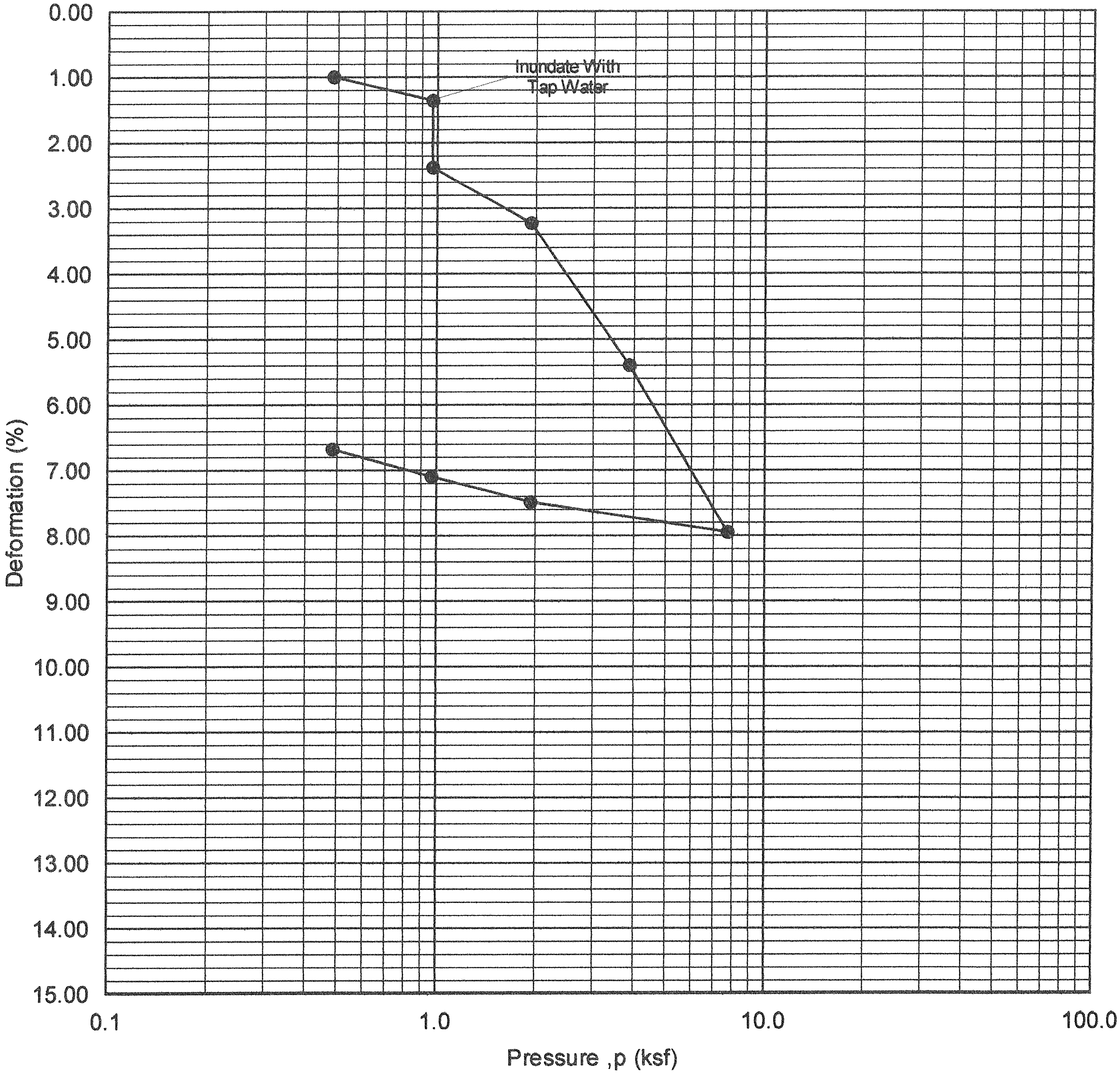


Boring No. : B-3
Depth (ft.) : 10.0
Sample Type: Clayey Silt

Dry Density (pcf) = 112.6
Moisture (%) = 14.9

Project Name:Residential Development-1007 E. Victoria Street,Carson	Project No.: 6816.18
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

PLATE C-4

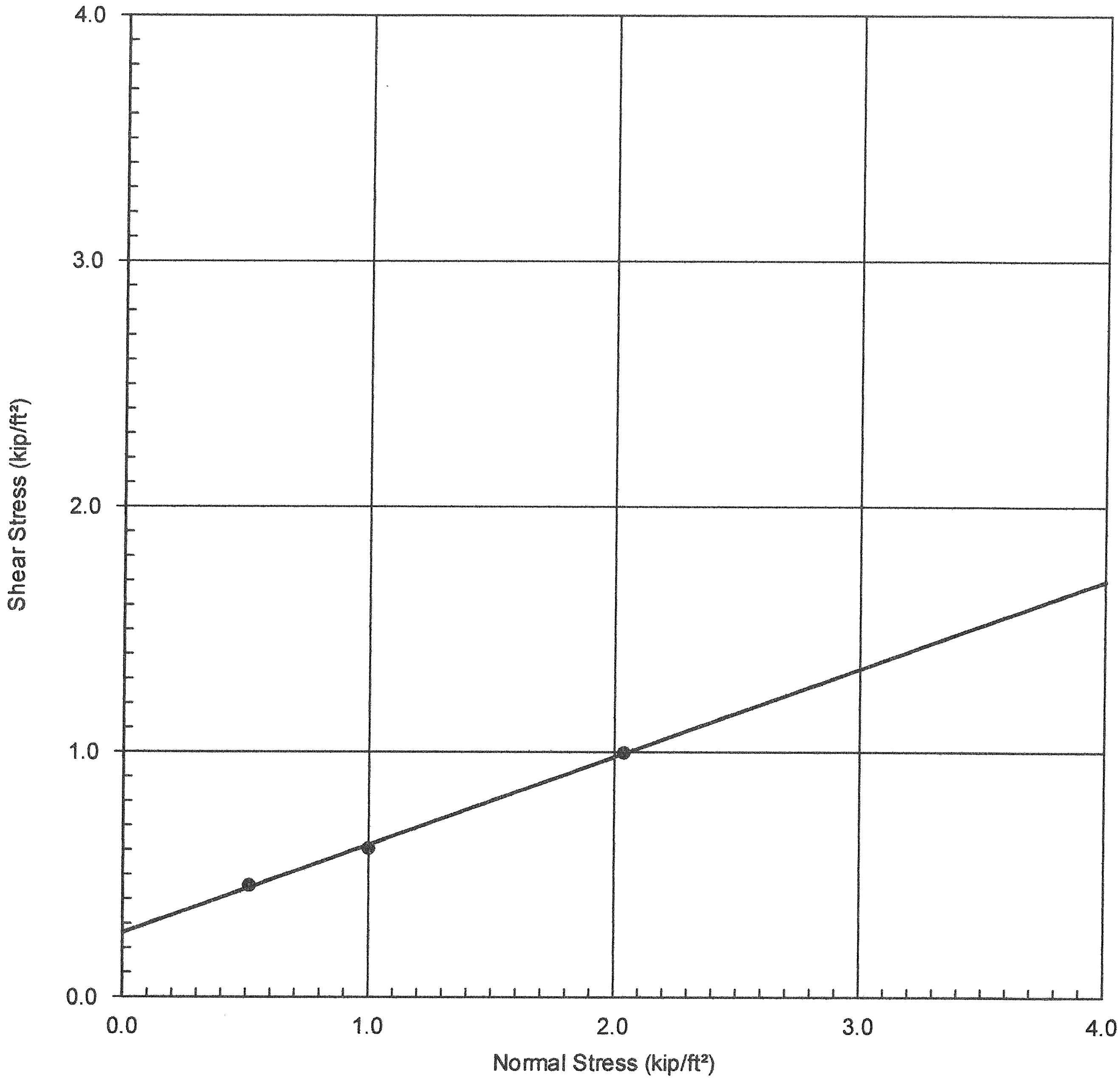


Boring No. : B-4
Depth (ft.) : 2.0
Sample Type: Clayey Silt with Fine Sand

Dry Density (pcf) = 110.3
Moisture (%) = 7.0

Project Name:Residential Development-1007 E. Victoria Street,Carson	Project No.: 6816.18
ASSOCIATED SOILS ENGINEERING, INC.	ONE-DIMENSIONAL CONSOLIDATION PROPERTIES OF SOILS (ASTM D 2435)

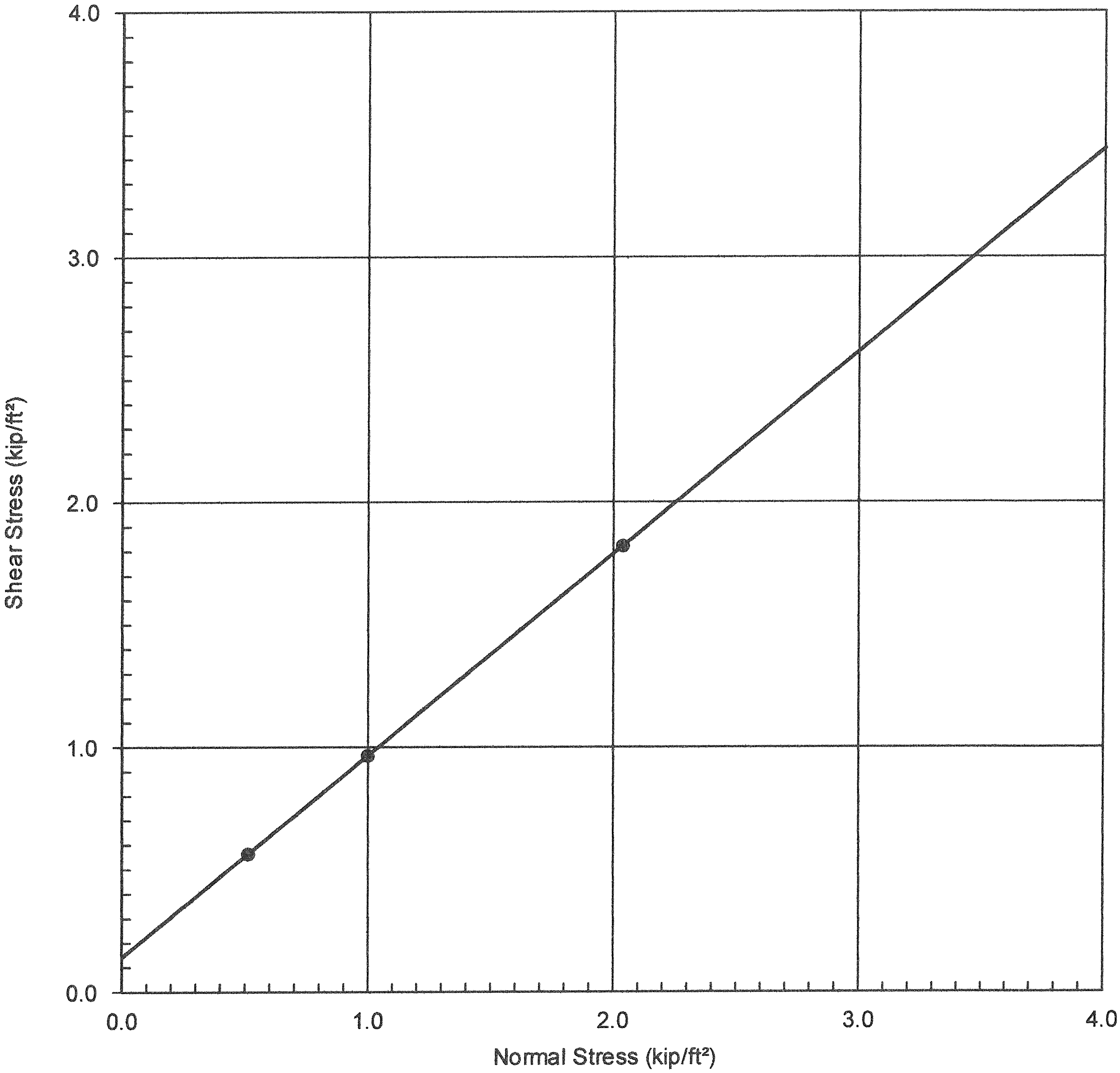
PLATE C-5



Boring No.	: B-1	Cohesion(C)	= 260 psf
Depth (ft.)	: 5.5	Friction (ϕ)	= 19.5°
Sample Type	: Relatively Undisturbed	Dry Density (pcf)	= 118.0
Soil Type	: Clayey Silt	Moisture (%)	= 13.2

Project Name:Residential Development-1007 E. Victoria Street,Carson	Project No.: 6816.18
ASSOCIATED SOILS ENGINEERING, INC.	DIRECT SHEAR TEST RESULTS (ASTM D 3080)

PLATE D-1

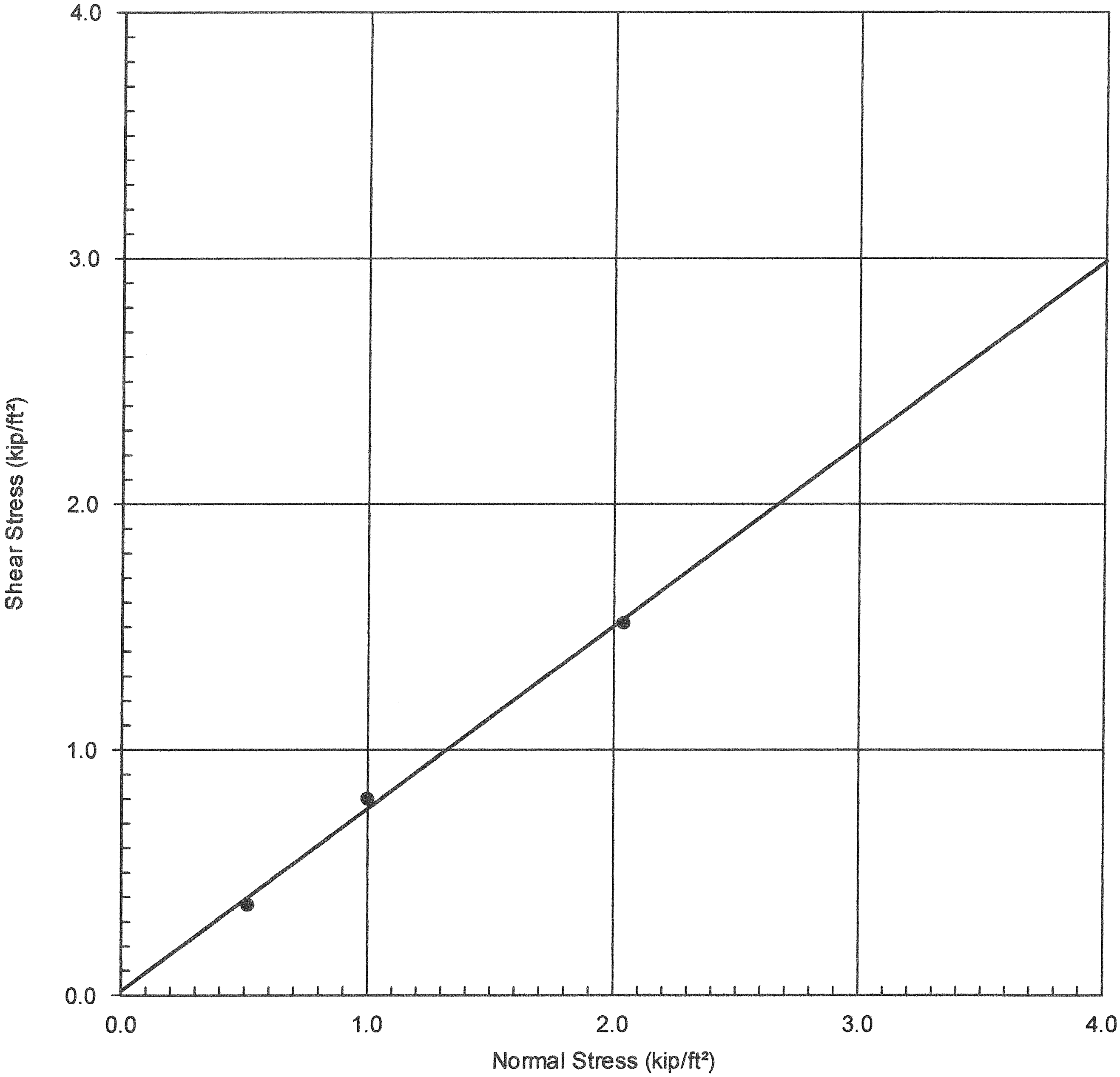


Boring No. : B-2
Depth (ft.) : 5.0
Sample Type : Relatively Undisturbed
Soil Type : Silty Clay with Fine Sand and some Gravel

Cohesion(C) = 140 psf
Friction (ϕ) = 39.5°
Dry Density (pcf) = 122.5
Moisture (%) = 11.7

Project Name:Residential Development-1007 E. Victoria Street,Carson	Project No.: 6816.18
ASSOCIATED SOILS ENGINEERING, INC.	DIRECT SHEAR TEST RESULTS (ASTM D 3080)

PLATE D-2

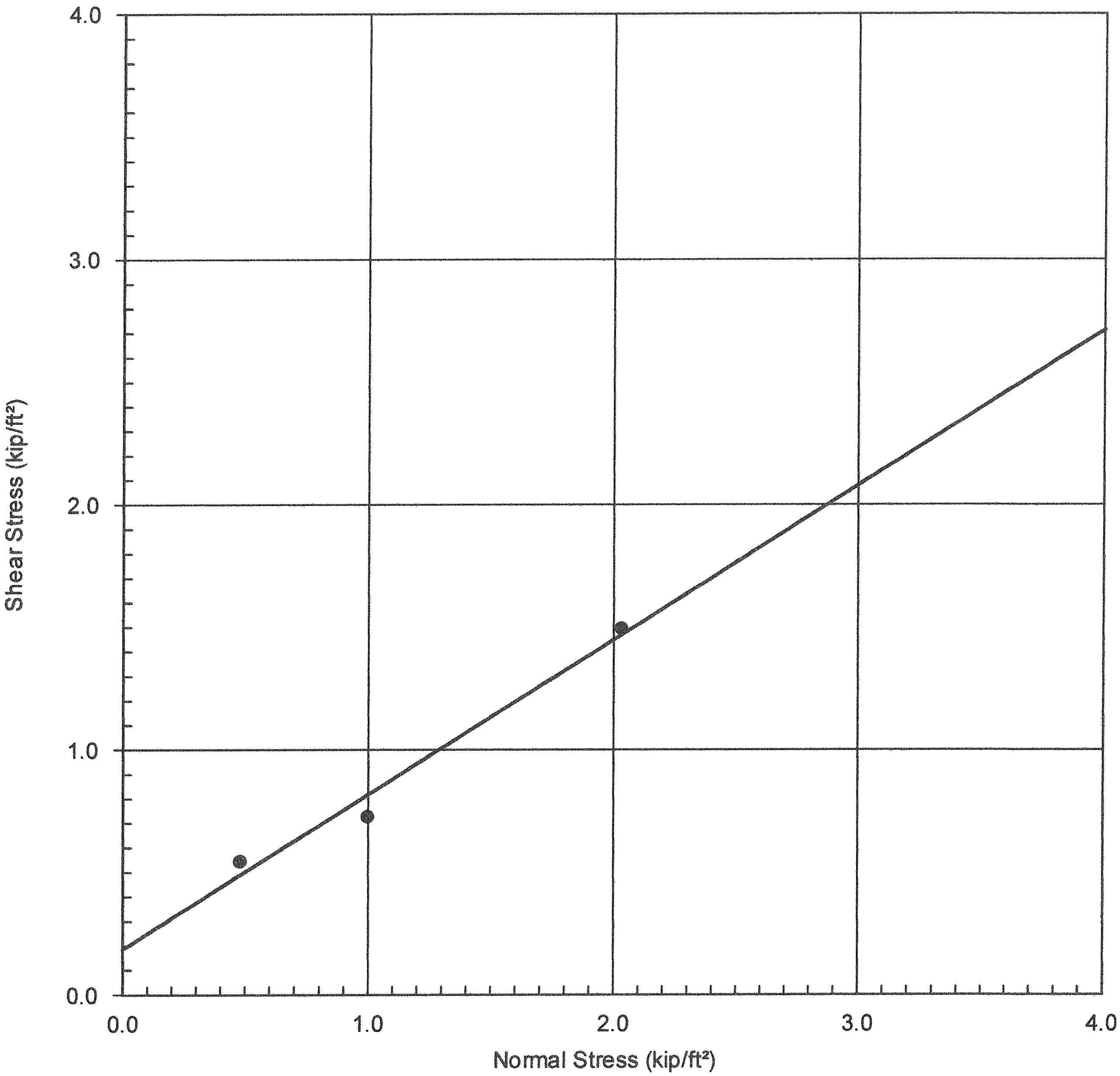


Boring No. : B-4
Depth (ft.) : 2.0
Sample Type : Relatively Undisturbed
Soil Type : Clayey Silt with Fine Sand and some Gravel

Cohesion(C) = 15 psf
Friction (ϕ) = 36.5°
Dry Density (pcf) = 110.3
Moisture (%) = 7.0

Project Name: Residential Development-1007 E. Victoria Street, Carson	Project No.: 6816.18
ASSOCIATED SOILS ENGINEERING, INC.	DIRECT SHEAR TEST RESULTS (ASTM D 3080)

PLATE D-3



Boring No. : B-3
Depth (ft.) : 0-5
Sample Type : Remolded (90% of Maximum Density)
Soil Type : Silty Clay with Fine Sand and some Gravel

Cohesion(C) = 185 psf
Friction (ϕ) = 32°
Dry Density (pcf) = 116.1
Moisture (%) = 9.5

Project Name:Residential Development-1007 E. Victoria Street,Carson	Project No.: 6816.18
ASSOCIATED SOILS ENGINEERING, INC.	DIRECT SHEAR TEST RESULTS (ASTM D 3080)

PLATE D-4

ATTERBERG LIMITS

ASTM D 4318-93

Project Name: Proposed New Residential Development-1007 East Victoria Street,Carson,CA
Project No. : 6816.18
Boring No. : B-3
Depth (feet): 0-5
Visual Sample Description: Silty Clay with Fine Sand (CL)

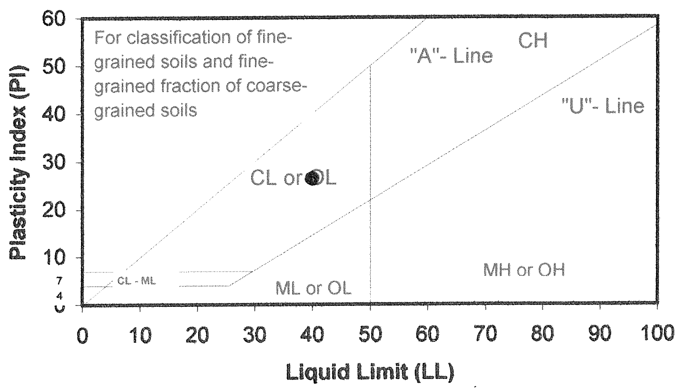
	PLASTIC LIMIT		LIQUID LIMIT			
TEST NO.	1	2	1	2	3	4
Number of Blows [N]			37	28	22	17
Container No.	b1	c1	A1	C1	B1	A2
Wet Wt. of Soil + Cont. (gm)	12.47	12.58	18.22	16.58	18.02	16.45
Dry Wt. of Soil + Cont. (gm)	11.51	11.63	16.28	14.90	16.06	14.76
Wt. of Container (gm)	4.28	4.30	11.01	10.58	11.22	10.75
Moisture Content (%) [Wn]	13.28	12.96	36.91	38.89	40.50	42.14

Liquid Limit	**40**
Plastic Limit	**13**
Plasticity Index	**27**
USCS Classification	**CL**

PI at "A" - Line = 0.73(LL-20) = 14.6

One - Point Liquid Limit Calculation

$LL = Wn(N/25)^{0.121}$



PROCEDURES USED

- [ ] Wet Preparation Multipoint - Wet
- [X] Dry Preparation Multipoint - Dry
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test

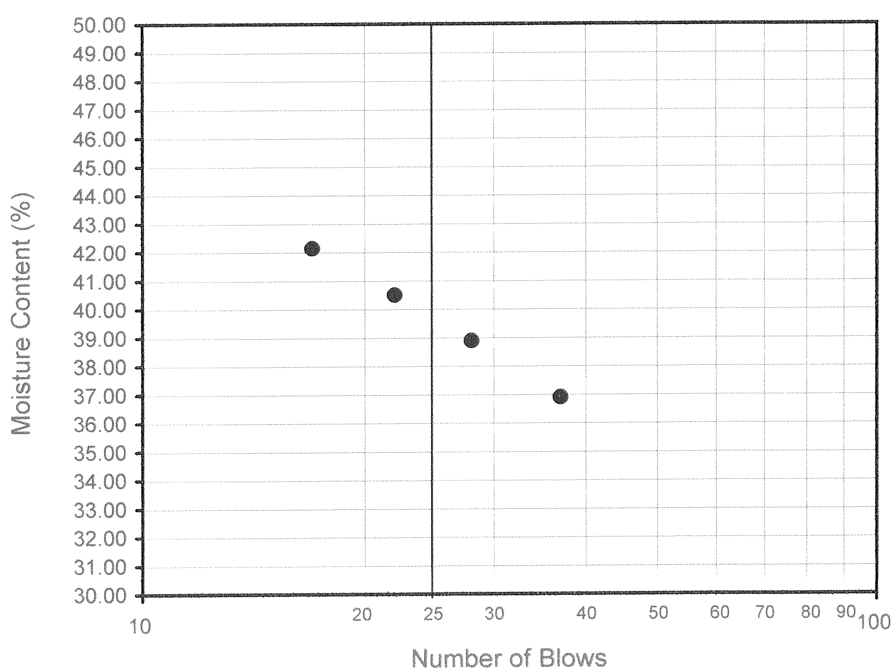


PLATE E-1

PERCOLATION DATA SHEET

Project: On-Site Storm Water Dispersal Job No.: 6816.18

Proposed New Residential Development

1007 East Victoria Street, Carson, CA

Test Hole No.: B-5 Date Excavated: 7/9/2018 Depth of Test Hole: 10'3"

Soil Classification: Clayey Silt Presoak: √

Percolation Tested By: Grant Zike Date: 7/10/2018

USE NORMAL ~~SANDY~~ (CROSS ONE) SOIL CRITERIA

Time	Time Interval (Min.)	Total Elapsed Time (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Percolation Rate (Min./Inches)
7:15 7:45	30	30	-12.75	-17.0	4.25	7.06
7:47 8:17	30	60	-13.0	-16.75	3.75	8.0
8:18 8:48	30	90	-12.75	-16.75	4.0	7.5
8:49 9:19	30	120	-12.5	-16.25	3.75	8.0

PLATE F-1

PERCOLATION DATA SHEET

Project: On-Site Storm Water Dispersal Job No.: 6816.18

Proposed New Residential Development

1007 East Victoria Street, Carson, CA

Test Hole No.: B-6 Date Excavated: 7/9/2018 Depth of Test Hole: 5'3"

Soil Classification: Silty Clay with Sand and Gravel/Silty Clay Presoak: √

Percolation Tested By: Grant Zike Date: 7/10/2018

USE NORMAL ~~SANDY~~ (CROSS ONE) SOIL CRITERIA

Time	Time Interval (Min.)	Total Elapsed Time (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Percolation Rate (Min./Inches)
7:18 7:48	30	30	-10.25	-11.0	0.75	40.0
7:50 8:20	30	60	-11.0	-11.25	0.25	120.0
8:21 8:51	30	90	-11.25	-11.5	0.25	120.0
8:51 9:21	30	120	-11.5	-11.75	0.25	120.0

PLATE F-2

APPENDIX B – SITE FAULTING AND SEISMIC HAZARD DATA

Plates I-1 and I-2	Results of EQFUALT Search

```
***********************
*                     *
*    E Q F A U L T    *
*                     *
*    Version 3.00     *
*                     *
***********************
```

DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 6816.18

DATE: 06-27-2018

JOB NAME: Proposed New residential Development-1007 East Victoria Street,Carson,CA
CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: C:\Program Files\EQFAULT1\Cgsflte.dat

SITE COORDINATES:
SITE LATITUDE: 33.8681
SITE LONGITUDE: 118.2555

SEARCH RADIUS: 62 mi

ATTENUATION RELATION: 20) Sadigh et al. (1997) Horiz. - Soil
UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0
DISTANCE MEASURE: clodis
SCOND: 0
Basement Depth: 5.00 km Campbell SSR: Campbell SHR:
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\Program Files\EQFAULT1\Cgsflte.dat

MINIMUM DEPTH VALUE (km): 0.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

PLATE I-1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT		
		MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
NEWPORT-INGLEWOOD (L.A.Basin)	0.5(0.8)	7.1	0.555	X
PALOS VERDES	6.9(11.1)	7.3	0.332	IX
PUENTE HILLS BLIND THRUST	9.2(14.8)	7.1	0.340	IX
UPPER ELYSIAN PARK BLIND THRUST	14.0(22.5)	6.4	0.171	VIII
WHITTIER	15.8(25.5)	6.8	0.151	VIII
SANTA MONICA	16.3(26.2)	6.6	0.168	VIII
HOLLYWOOD	17.0(27.4)	6.4	0.140	VIII
RAYMOND	17.6(28.4)	6.5	0.146	VIII
VERDUGO	19.1(30.7)	6.9	0.171	VIII
MALIBU COAST	19.4(31.3)	6.7	0.149	VIII
SAN JOAQUIN HILLS	22.1(35.5)	6.6	0.122	VII
SIERRA MADRE	23.8(38.3)	7.2	0.164	VIII
SAN JOSE	24.6(39.6)	6.4	0.093	VII
CLAMSHELL-SAWPIT	26.0(41.8)	6.5	0.094	VII
ANACAPA-DUME	26.5(42.6)	7.5	0.176	VIII
NORTHRIDGE (E. Oak Ridge)	26.5(42.7)	7.0	0.129	VIII
NEWPORT-INGLEWOOD (Offshore)	27.3(44.0)	7.1	0.104	VII
SIERRA MADRE (San Fernando)	28.3(45.6)	6.7	0.098	VII
CHINO-CENTRAL AVE. (Elsinore)	28.8(46.3)	6.7	0.096	VII
SAN GABRIEL	31.1(50.1)	7.2	0.096	VII
SANTA SUSANA	34.1(54.8)	6.7	0.078	VII
CUCAMONGA	35.0(56.4)	6.9	0.087	VII
ELSINORE (GLEN IVY)	35.5(57.1)	6.8	0.062	VI
SIMI-SANTA ROSA	39.1(63.0)	7.0	0.082	VII
HOLSER	39.8(64.0)	6.5	0.055	VI
OAK RIDGE (Onshore)	43.5(70.0)	7.0	0.072	VI
CORONADO BANK	45.5(73.2)	7.6	0.081	VII
SAN ANDREAS - 1857 Rupture M-2a	46.0(74.0)	7.8	0.092	VII
SAN ANDREAS - Whole M-1a	46.0(74.0)	8.0	0.105	VII
SAN ANDREAS - Mojave M-1c-3	46.0(74.0)	7.4	0.070	VI
SAN ANDREAS - Cho-Moj M-1b-1	46.0(74.0)	7.8	0.092	VII
SAN CAYETANO	48.8(78.6)	7.0	0.062	VI
SAN JACINTO-SAN BERNARDINO	50.1(80.7)	6.7	0.037	V
SAN ANDREAS - SB-Coach. M-2b	52.3(84.2)	7.7	0.074	VII
SAN ANDREAS - SB-Coach. M-1b-2	52.3(84.2)	7.7	0.074	VII
SAN ANDREAS - San Bernardino M-1	52.3(84.2)	7.5	0.064	VI
ELSINORE (TEMECULA)	54.3(87.4)	6.8	0.036	V
CLEGHORN	54.7(88.1)	6.5	0.028	V
OAK RIDGE(Blind Thrust Offshore)	57.2(92.1)	7.1	0.054	VI
CHANNEL IS. THRUST (Eastern)	58.5(94.2)	7.5	0.072	VI

DETERMINISTIC SITE PARAMETERS

Page 2

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT		
		MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
SAN ANDREAS - Carrizo M-1c-2	59.2(95.3)	7.4	0.051	VI
SAN JACINTO-SAN JACINTO VALLEY	59.5(95.8)	6.9	0.034	V
VENTURA - PITAS POINT	60.6(97.5)	6.9	0.043	VI
SANTA YNEZ (East)	61.3(98.7)	7.1	0.038	V

-END OF SEARCH- 44 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE NEWPORT-INGLEWOOD (L.A.Basin) FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 0.5 MILES (0.8 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.5545 g

PLATE I-2

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