Appendix D
Geotechnical Report

ALBUS-KEEFE & ASSOCIATES, INC
GEOTECHNICAL CONSULTANTS

January 13, 2020
J.N.: 2810.00

Ms. Doris Nguyen
The Olson Company
3010 Old Ranch Parkway, Suite 100
Seal Beach, California 90740

Subject: **Geotechnical Grading Plan Review Report, Proposed Multi-Family Residential Development, Tentative Tract 82945, Normandie Avenue & West 141st Street, Gardena, California**

Dear Ms. Nguyen,

Albus-Keefe & Associates, Inc. is pleased to present to you our geotechnical-design report for the proposed development at the subject site. This report presents the results of our literature review, subsurface exploration, laboratory testing, and engineering analyses. Conclusions relevant to the feasibility of the proposed site development are also presented herein based on the findings of our work.

We appreciate this opportunity to be of service to you. If you have any questions regarding the contents of this report, please do not hesitate to call.

Sincerely,

ALBUS-KEEFE & ASSOCIATES, INC.

Paul Hyun Jin Kim
Associate Engineer

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of our geotechnical design report is to review the proposed site development shown on the referenced site improvement plan with respect to the geotechnical conditions in order to provide recommendations for site development. The scope of our work included:

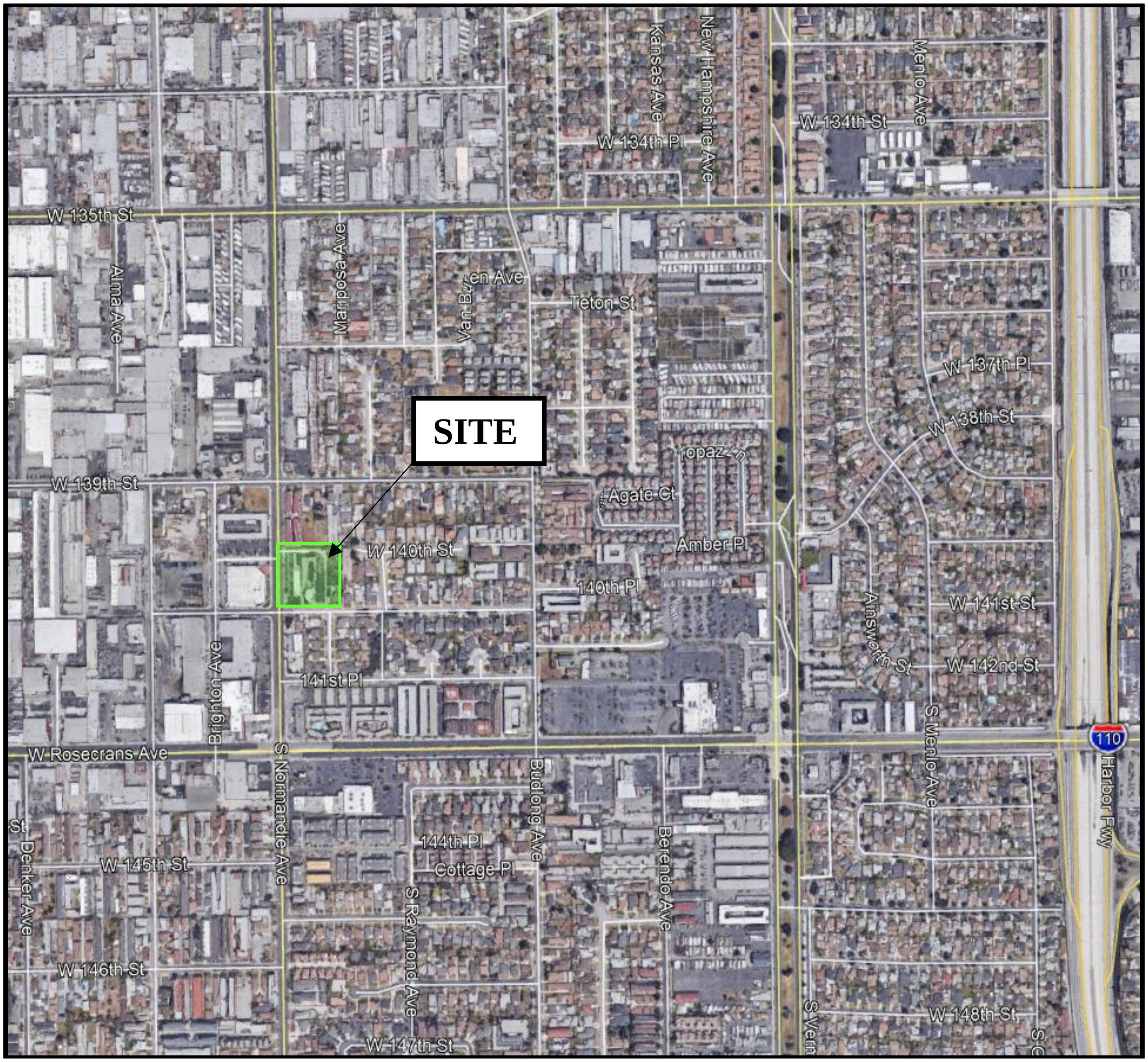
- Review of published geologic reports, maps, and seismic data of the general vicinity
- Review of the referenced improvement plan
- Exploratory drilling and soil sampling
- Laboratory testing of selected soil samples
- Engineering and geologic analyses of data obtained from our review, subsurface exploration and laboratory testing
- Development of recommendations for site construction
- Preparation of this report

1.2 SITE LOCATION AND DESCRIPTION

The site is located at 1335, 1337, 1341, & 1343 West 141st Street in the city of Gardena, California. The site is bordered by residential developments to the north and east, West 141st Street to the south, and South Normandie Avenue to the west. The location of the site and its relationship to the surrounding areas is shown on the Site Location Map, Figure 1.

The site is rectangular in shape and comprises approximately 2.04 acres of land. The site is currently occupied by a plant nursery with several greenhouse structures throughout the site. The southeast corner of the site is developed as a residential parcel. The residential parcel is occupied by two single-story residential buildings.

A wooden fence bounds the property along the north and east property line. The site is relatively level and the elevation ranges from 53 to 54 feet above mean sea level (MSL). Drainage within the site is generally directed to the west and south toward West 141st Street and South Normandie Avenue. Vegetation associated with the single-family residences included small shrubs and scattered medium-sized trees. Native vegetation within the nursery area of the site is sparse and only includes a few medium sized-trees and medium to large-sized shrubs.



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SITE LOCATION MAP
The Olson Company
Proposed Residential Development
1335, 1337, 1341, & 1343 West 141st Street
Gardena, California

NOT TO SCALE

FIGURE 1

1.3 PROPOSED DEVELOPMENT

We understand the site will be developed for multi-family residential use. Based on the plan provided, the proposed site development will consist of 50 residential homes and associated interior driveways, perimeter walls, underground utilities and a storm water infiltration system.

After review of the conceptual grading plan, we anticipate that minor rough grading of the site will be required to achieve future surface configuration. No structural plans were available in preparing of this report. However, we expect the proposed residential dwellings will be a maximum three-story, wood-framed structures with concrete slabs on grade yielding relatively light foundation loads.

2.0 INVESTIGATION

2.1 RESEARCH

We have reviewed the referenced geologic publications, maps, and historical aerial photos of the vicinity. Data from these sources were utilized to the development of some of our findings and conclusions presented in this report. As early as 1941, the site was considered vacant land. The site may have been initially part of a large agricultural development. By 1952, the site appears to have been utilized as a plant nursery. The site consisted of several greenhouses and the existing one-story residential buildings. During 1972 to 1980, two greenhouses within the central and northeast portions of the site were constructed. Since 1980, the subject site was relatively unchanged.

2.2 SUBSURFACE EXPLORATION

Subsurface exploration for this investigation was conducted at the site on May 6, 2019, and consisted of drilling four (4) exploratory borings. The borings were drilled to maximum depths of approximately 51.5 feet below the existing ground surface utilizing a truck-mounted, hollow-stem-auger drill rig. Representatives of *Albus-Keefe & Associates, In*c. logged the exploratory excavations. Visual and tactile identifications were made of the materials encountered, and their descriptions are presented on the Exploration Logs in Appendix A. The approximate locations of the exploratory excavations completed by this firm are shown on the enclosed Geotechnical Map, Plate 1.

Bulk, relatively undisturbed and Standard Penetration Test (SPT) samples were obtained at selected depths within the exploratory boring for subsequent laboratory testing. Relatively undisturbed samples were obtained using a 3-inch O.D., 2.5-inch I.D., California split-spoon soil sampler lined with brass rings. SPT samples were obtained from the boring using a standard, unlined SPT soil sampler. During each sampling interval, the sampler was driven 18 inches with successive drops of a 140-pound automatic hammer falling 30 inches. The number of blows required to advance the sampler was recorded for each six inches of advancement. The total blow count for the lower 12 inches of advancement per soil sample is recorded on the exploration log. Samples were placed in sealed containers or plastic bags and transported to our laboratory for analyses. The borings were backfilled with auger cuttings upon completion of sampling.

In addition, one percolation test boring, P-1, was also excavated to an approximate depth of 20 feet in the vicinity of exploratory boring B-1 for subsequent percolation testing. The percolation test well was later backfilled with auger cuttings upon completion of testing. Results of our percolation testing are discussed in a separate report.

2.3 LABORATORY TESTING

Selected samples of representative earth materials from the borings were tested in our laboratory. Tests consisted of in-situ moisture and dry density, maximum dry density and optimum moisture content, expansion index, soluble sulfate content, consolidation/collapse potential, direct shear, corrosivity (pH, chloride, & minimum resistivity), Atterberg limits, and grain size analysis. Descriptions of laboratory testing and a summary of the test results are presented in Appendix B and on the exploration logs in Appendix A.

3.0 SUBSURFACE CONDITIONS

3.1 SOIL CONDITIONS

Descriptions of the earth materials encountered during our investigation are summarized below and are presented in detail on the Exploration Logs presented in Appendix A.

Soils encountered at the site consist of artificial fill materials overlying alluvial deposits. The artificial fill materials typically consist of brown silty sand and sandy clay. The artificial fill was typically slightly moist and dense to very stiff to hard. The maximum thickness of the fill was encountered from approximately 2 feet below existing grades.

The alluvial deposits were encountered below the artificial fill materials to the maximum depth of exploration, 51.5 feet below the ground surface. The alluvial deposits consisted of interlayered coarse-grained and fine-grained material. The coarse-grained material was typically brown sand with varying amounts of fines. These deposits are slightly moist to very moist and medium dense to very dense. The fine-grained material consisted of brown clay and silt. These deposits are typically slightly moist to moist and hard to very stiff.

3.2 GROUNDWATER

Groundwater was observed at 30 feet during this firm’s subsurface investigation. A review of the CDMG Seismic Hazard Zone Report 027 indicates that historical high groundwater levels for the general site area is as shallow as 30 feet below the existing ground surface.

3.3 FAULTING

Geologic literature and field exploration do not indicate the presence of active faulting within the site. The site does not lie within an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act. Table 3.1 presents a summary of all the known seismically active faults within 10 miles of the site based on the 2008 National Seismic Hazards Maps.

TABLE 3.1
Summary of Faults

Name	Distance (miles)	Slip Rate (mm/yr.)	Preferred Dip (degrees)	Slip Sense	Rupture Top (km)	Fault Length (km)
Newport Inglewood Connected alt 1	0.8	1.3	89	strike slip	0	208
Newport-Inglewood, alt 1	0.8	1	88	strike slip	0	65
Newport Inglewood Connected alt 2	1.17	1.3	90	strike slip	0	208
Puente Hills (LA)	6.21	0.7	27	thrust	2.1	22
Palos Verdes	7.96	3	90	strike slip	0	99
Palos Verdes Connected	7.96	3	90	strike slip	0	285
Puente Hills (Santa Fe Springs)	9.23	0.7	29	thrust	2.8	11

4.0 ANALYSES

4.1 SEISMICITY AND SEISMIC DESIGN PARAMETERS

2019 CBC requires seismic parameters in accordance with ASCE 7-16. Unless noted otherwise, all section numbers cited in the following refer to the sections in ASCE 7-16.

Per Section 20.3 the project site was designated as Site Class D. We used USGS seismic design maps web tool developed by SEAOC and OSHPD to obtain the basic mapped acceleration parameters, including short periods (S_S) and 1-second period (S_1) MCE_R Spectral Response Accelerations. Section 11.4.8 requires site-specific ground hazard analysis for structures on Site Class E with S_S greater than or equal to 1.0 or Site Class D or E with S_1 greater than or equal to 0.2. Based on the mapped values of S_S and S_1 the project site falls within this category, requiring site specific hazard analysis in accordance with Section 21.2.

According to Section 21.2.3 (Supplement 1), the site-specific Risk Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration at any period is the lesser of the probabilistic and the deterministic response accelerations, subject to the exception specified in the same section. The probabilistic response spectrum was developed using USGS Risk Targeted Ground Motion (RTGM) calculator, which implements Method 2 as described on Section 21.2.1.2. The spectral acceleration and annual frequency of exceedance required by the RTGM calculator were extracted from hazard curves produced by USGS Unified Hazard Tool for the project site.

In accordance with Section 21.2.2 (Supplement 1), the deterministic spectral response acceleration at each period was calculated as the 84^{th} percentile, 5% damped, response acceleration, using the NGA-

West2 GMPE Worksheet. For this, the information from at least three causative faults with the greatest contribution per deaggregation analysis were used, and the larger acceleration spectrum among these was selected as the deterministic response spectrum. The deterministic spectrum was adjusted per requirements in Section 21.2.2 (Supplement 1) where applicable. Both probabilistic and deterministic spectra were subjected to the maximum direction scale factors specified in Section 21.2 to produce the maximum acceleration spectra.

Design response spectrum was developed by subjecting the site-specific MCE_R response spectrum to the provisions outlined in Section 21.3. This process included comparison with 80% code-based design spectrum determined in accordance with Section 11.4.6. The short period and long period site coefficient (F_a and F_v, respectively) were determined per Section 21.3 in conjunctions with Table 11.4-1. Site specific design acceleration parameters (S_{MS}, S_{M1}, S_{DS}, and S_{D1}) were calculated according to Section 21.4.

Per Section 11.2 (definitions on Page 79 of ASCE7-16) for evaluation of liquefaction, lateral spreading, seismic settlements, and other soil-related issues, Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration PGA_M shall be used. The site-specific PGA_M is calculated per Section 21.5.3, as the lesser of the probabilistic PGA_M (Section 21.5.1) and deterministic PGA_M (Section 21.5.2), but no less than 80% site modified peak ground acceleration, PGA_M, obtained from SEAOC/OSHPD web-based seismic hazard tool.

4.2 STATIC SETTLEMENT

Analyses were performed to evaluate potential for static settlement. Our analyses were based on the results of consolidation tests performed on selected samples from our borings. Results of our testing indicate the near-surface soils are slightly compressible. If the existing 4 to 5 feet of earth materials are removed and recompacted, we estimate the total settlement will be less than 1 inch.

5.0 CONCLUSIONS

5.1 FEASIBILITY OF PROPOSED DEVELOPMENT

From a geotechnical point of view, the proposed site development is considered feasible provided appropriate geotechnical recommendations are incorporated into the design and construction of the project. Key issues that could have significant fiscal impacts on the geotechnical aspects of the proposed site development are discussed in the following sections of this report.

5.2 GEOLOGIC HAZARDS

5.2.1 Ground Rupture

No known active faults are known to project through the site nor does the site lie within the boundaries of an “Earthquake Fault Zone” as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act. Therefore, the potential for ground rupture due to an earthquake beneath the site is considered low. The nearest zoned fault is the Newport Inglewood fault located approximately 0.8 miles northeast.

5.2.2 Ground Shaking

The site is situated in a seismically active area that has historically been affected by generally moderate to occasionally high levels of ground motion. The site lies in relative close proximity to several seismically active faults; therefore, during the life of the proposed structures, the property will probably experience similar moderate to occasionally high ground shaking from these fault zones, as well as some background shaking from other seismically active areas of the Southern California region. Potential ground accelerations have been estimated for the site and are presented in Section **Error! Reference source not found.** of this report. Design and construction in accordance with the current California Building Code (C.B.C.) requirements is anticipated to adequately address potential ground shaking.

5.2.3 Landsliding

The site is not located within an area identified by the California Geologic Survey (CGS) as having potential for seismic slope instability. Geologic hazards associated with landsliding are not anticipated at the sites.

5.2.4 Liquefaction

Engineering research of soil liquefaction potential (Youd, et al., 2001) indicates that generally three basic factors must exist concurrently in order for liquefaction to occur. These factors include:

- A source of ground shaking, such as an earthquake, capable of generating soil mass distortions.
- A relatively loose silty and/or sandy soil.
- A relative shallow groundwater table (within approximately 50 feet below ground surface) or completely saturated soil conditions that will allow positive pore pressure generation.

The liquefaction susceptibility of the onsite soils was evaluated by analyzing the potential concurrent occurrence of the above-mentioned three basic factors. The liquefaction evaluation for the site was completed under the guidance of Special Publication 117A: Guidelines for Evaluating and Mitigating Seismic Hazards in California (CDMG, 2008).

Groundwater was encountered during this firm's investigation to the depth of 30 feet below existing ground surface. However, the site is generally underlain by interlayered soils that are dense or consist of clay-like soils and therefore not considered susceptible to liquefaction. As such, the potential for liquefaction at the site is very low. In addition, the site is not located within a mapped liquefaction hazard zone by the California Geologic Survey.

5.3 STATIC SETTLEMENT

The existing fill (maximum 2 feet in thickness) is not considered suitable for support of engineered loads. Additionally, the near surface alluvium is subject to slight consolidation upon loading. The deeper portions of the earth materials are generally very stiff to hard and medium dense to dense and are anticipated to result in minor settlement due to the weight of new foundations. Provided the existing upper 4 to 5 feet of earth materials are removed and recompacted, total and differential static settlement can likely be limited to a maximum of 1 inch and ½-inch over 30 feet, respectively. These estimated magnitudes of static settlements are considered within tolerable limits for the proposed residential structures.

5.4 EXCAVATION AND MATERIAL CHARACTERISTICS

In general, the existing near-surface soils are considered unsuitable in their existing condition to support proposed structural fills and site development. This condition can be mitigated by removal and recompaction of unsuitable soils. The anticipated depth of removal to mitigate structural load-induced settlement below the proposed residential buildings and retaining walls is on the order of 4 to 5 feet below existing ground surface. Removals in areas of pavement and hardscapes are anticipated to be on the order of 2 feet below the existing ground surface.

Removal and recompaction of the existing surficial materials are anticipated to result in minor shrinkage. Design of site grading will require consideration of this loss when evaluating earthwork balance issues.

Onsite earth materials are anticipated to be relatively easy to excavate with conventional heavy earthmoving equipment. The site earth materials are generally considered suitable for reuse as fill provided they are cleared on deleterious debris and oversized rocks (greater than 12 inches in greatest dimension). Site materials are generally above the optimum moisture content with a few localized layers significantly above the optimum moisture content. As such, fill soils derived from onsite soils will require drying and mixing in preparation for reuse as compacted fill.

Temporary construction slopes and trench excavations can likely be cut vertically up to a height of 4 feet within the onsite materials provided that no surcharging of the excavations is present. Temporary excavations greater than 4 feet in height will likely require side laybacks to 1:1 (H:V) or flatter to mitigate the potential for sloughing. Site materials may be prone to sloughing and possible caving if allowed to dry.

Asphaltic concrete or Portland cement concrete, if encountered, can also be incorporated into the engineered fill provided they are reduced in size to less than 4 inches.

Onsite disposal systems, clarifiers, and other underground improvements may be present on site. If encountered during future rough grading, these improvements will require proper abandonment or removal.

Off-site improvements exist near and along the property lines. The presence of the existing offsite improvements will limit removals of unsuitable materials adjacent the property lines. Special grading techniques, such as slot cutting, will be required adjacent to the property lines were offsite structures are nearby, particularly along the western portion of the north property line due to the adjacent retaining wall. Construction of perimeter site walls will likely require deepened footings or caissons and grade beams where removals are restricted by property boundaries.

5.5 SHRINKAGE AND SUBSIDENCE

Volumetric changes in earth quantities will occur when excavated onsite soil materials are replaced as properly compacted fill. We estimate the existing upper 4 feet of earth materials will shrink up to approximately 5 to 10 percent. Subsidence of removal bottoms is estimated to be negligible. The estimates of shrinkage and bulkage are intended as an aid for project engineers in determining earthwork quantities. However, these estimates should be used with some caution since they are not absolute values. Contingencies should be made for balancing earthwork quantities based on actual swelling and bulkage that occurs during the grading process.

5.6 SOIL EXPANSION

Based on our laboratory test results and the USCS visual manual classification, the near-surface soils within the site are generally anticipated to possess a **Low** expansion potential. Additional testing for soil expansion may be required subsequent to rough grading and prior to construction of foundations and other concrete work to confirm these conditions.

6.0 RECOMMENDATIONS

6.1 EARTHWORK

6.1.1 General Earthwork and Grading Specifications

All earth earthwork and grading should be performed in accordance with applicable requirements of Cal/OSHA, applicable specifications of the Grading Codes of the City of Gardena, California in addition to the recommendations presented herein.

6.1.2 Pre-Grade Meeting and Geotechnical Observation

Prior to commencement of grading, we recommend a meeting be held between the developer, City Inspector, grading contractor, civil engineer, and geotechnical consultant to discuss the proposed grading and construction logistics. We also recommend that a geotechnical consultant be retained to provide soil engineering and engineering geologic services during site grading and foundation construction. This is to observe compliance with the design specifications and recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated. If conditions are encountered that appear to be different than those indicated in this report, the project geotechnical consultant should be notified immediately. Design and construction revisions may be required.

6.1.3 Site Clearing

All existing site improvements, oversized materials, vegetation and other deleterious materials should be removed from the areas to be developed. Existing underground improvements such as utility lines, septic tanks, seepage pits, etc. are also anticipated at the site. If encountered during site development, these improvements should also be completely removed from the site and seepage pits should be properly abandoned in accordance with the requirements established by the governing agencies as well as recommendations made in the field by the project geotechnical consultant.

In general, seepage pits that are open should be cleared of any fluids and then filled with 2-sack cement slurry up to within 5 feet of proposed grades. Any brick lining that remains in the upper 5 feet should be removed and the remainder of the pit filled with engineered fill in accordance with Section 6.1.7. Seepage pits that are presently backfilled with soil should be removed to a depth of 10 feet below pad grade and be capped with 2-sack cement slurry. The slurry cap should be at least 5 feet thick and should extend at least 12 inches outside the perimeter of the seepage pit. The remaining 5 feet should be filled with engineered fill in accordance with Section 6.1.7.

The project geotechnical consultant should be notified at the appropriate times to provide observation services during clearing operations to verify compliance with the above recommendations. Voids created by clearing and excavation should be left open for observation by the geotechnical consultant. Should any unusual soil conditions or subsurface structures be encountered during site clearing or grading that are not described or anticipated herein, these conditions should be brought to the immediate attention of the project geotechnical consultant for corrective recommendations as needed.

Asphaltic concrete debris generated by site demolition can be reduced to no more than 4 inches in maximum dimension and uniformly incorporated with fill soils during earthwork operations.

6.1.4 Ground Preparation

To provide a uniform bearing material, the upper 4 feet of the existing earth materials should be removed and replaced as engineered compacted fills. These removals will be required in proposed building pads, retaining walls, and any other "structural" areas, and replaced as engineered compacted fill. In areas of the proposed screen walls and pavement, the removals may be limited to within the existing artificial soils, with an estimated thickness of approximately 2 feet. The actual depth of removal should be determined by the geotechnical consultant during grading.

In addition to general removal of unsuitable soils, the existing soils should be over-excavated to a minimum depth of 2 foot below the bottom of footings for residential structures supported by conventional spread footings. Existing soils within driveway and parking areas and retaining walls less than 3 feet, should be removed to at least 12 inches below the proposed pavement subgrade and replaced with engineered compacted fill.

Removals should extend laterally beyond the limits of the proposed buildings and retaining walls over 3 feet in height a distance equal to the depth of removal (i.e. 1:1 projection) but not less than 5 feet. Existing soils below proposed retaining walls less than 3 feet in height, screen walls, and roadways ways should be removed laterally to at least the edge of the structure or pavement. Where removals are limited by existing structures, protected trees or property lines, special considerations may be required in the construction of affected improvements. Under such conditions, specific recommendations should be provided by this firm.

All removal excavations should be evaluated by the geotechnical consultant during grading to confirm the exposed conditions are as anticipated and to provide supplemental recommendations if required.

The grading contractor should take appropriate measures when excavating adjacent any existing improvements to remain in-place to avoid disturbing or compromising support of existing structures.

6.1.5 Scarification

Following removals, the exposed grade should first be scarified to a depth of 6 inches; moisture conditioned to above the optimum moisture content, and then compacted to at least 90 percent of the laboratory determined maximum dry density.

6.1.6 Temporary Excavations

Temporary construction slopes and trench excavations in the surficial units may be cut vertically up to a height of 4 feet provided that no surcharging of the excavations is present. Temporary excavations greater than 4 feet in height but no more than 10 feet should be laid back to a 1:1 (H:V) or flatter or shored to mitigate the potential for instability. Where temporary excavations expose granular soils, the vertical cut may be decreased to as much as zero (0) and lay backs will likely be flatter to a gradient of 2:1 (H:V).

Excavations should not be left open for prolonged periods of time. The project geotechnical consultant should observe all temporary cuts to confirm anticipated conditions and to provide alternate recommendations if conditions dictate. All excavations should conform to the requirements of Cal/OSHA.

The grading contractor should take appropriate measures when excavating adjacent existing improvements to avoid disturbing or compromising support of existing structures.

6.1.7 Fill Placement

In general, materials excavated from the site may be used as fill provided they are free of deleterious materials, do not contain rocks greater than 6 inches in maximum dimension within 3 feet of finished pad grade and do not contain rocks greater than 12 inches in maximum dimension below 3 feet from finish pad grade. Rocks greater than 12 inches in diameter that cannot be reduced in size should be removed from the site. Asphaltic concrete debris generated by site demolition can be reduced to no more than 4 inches in maximum dimension and incorporated with fill soils during earthwork operations. All fills should be sufficiently well graded to prevent nesting of larger particles. Fill should be placed in lifts no greater than 8 inches in loose thickness, moisture-conditioned to above the optimum moisture content, and then compacted in place to at least 90 percent of the maximum dry density determined in accordance with ASTM D 1557. Each lift should be treated in a similar manner. Subsequent lifts should not be placed until the project geotechnical consultants have approved the preceding lift.

6.1.8 Import Materials

If import materials are required to achieve the proposed finish grades, the import soils should have an Expansion Index (EI) less than 40 (ASTM D 4829) and negligible soluble sulfate content. Import sources should be indicated to the geotechnical consultant at least 3 days prior to hauling the materials to the site so that appropriate testing and evaluation of the fill materials can be performed in advance.

6.2 SEISMICITY

Following ASCE7-16, Section 21.5.3, we have estimated site-specific Maximum Considered Earthquake Geometric Mean (MCEG) peak ground acceleration PGAM = 0.828g. Per Section 11.2, this value should be used for evaluation of liquefaction, lateral spreading, seismic settlements, and other soil-related issues. Based on the results of deaggregation analysis performed using USGS Unified Hazard Tool, the mean event associated with a probability of exceedance equal to 2% over 50 years has a moment magnitude of 6.69 and the mean distance to the seismic source is 4.5 miles.

6.3 SEISMIC DESIGN PARAMETERS

For design of the project in accordance with Chapter 16 of the 2019 CBC, the following table presents the seismic design factors:

TABLE 6.1
2019 CBC Seismic Design Parameters

Parameter	Value
Site Class	D
Mapped MCE_R Spectral Response Acceleration, short periods, S_S	1.830
Mapped MCE_R Spectral Response Acceleration, at 1-sec. period, S_1	0.647
Site Coefficient, Fa	1.0
Site Coefficient, Fv	**2.5**
Adjusted MCE_R Spectral Response Acceleration, short periods, S_{MS}	**1.982**
Adjusted MCE_R Spectral Response Acceleration, at 1-sec. period, S_{M1}	**1.633**
Design Spectral Response Acceleration, short periods, S_{DS}	**1.322**
Design Spectral Response Acceleration, at 1-sec. period, S_{D1}	**1.089**
Long-Period Transition Period, T_L (sec.)	8
Seismic Design Category for Risk Categories I-IV	**D**

MCE_R = Risk-Targeted Maximum Considered Earthquake
Boldface values: Site-specific values per ASCE7-16; other values are mapped values.

6.4 CONVENTIONAL FOUNDATION DESIGN

6.4.1 General

The following design parameters are provided to assist the project structural engineer to design foundation systems to support the proposed structures at the site. Recommendations for design of other foundation systems will be provided upon request. These design parameters are based on typical site materials encountered during subsurface exploration and are provided for preliminary design and estimating purposes. Depending on actual materials encountered during site grading and actual foundation loads, the design parameters presented herein may require modification.

6.4.2 Soil Expansion

The recommendations presented herein are based on soils with a **Low** expansion potential (EI <51). Following site grading, additional testing of site soils should be performed by the project geotechnical consultant to confirm the basis of these recommendations. If site soils with higher expansion potentials are encountered or imported to the site, the recommendations contained herein may require modification.

6.4.3 Settlement

Under normal static conditions, the foundation system should be designed to tolerate a total settlement of 1 inches and a differential settlement of ½-inch over 30 feet.

6.4.4 Allowable Bearing Value

Provided site grading is performed as recommended herein, a bearing value of 2,000 pounds per square foot (psf) may be used for continuous beams or isolated spread footings. The bearing value is based on beams having a minimum width of 12 inches and founded at a minimum of 12 inches below the lowest adjacent grade. The bearing value for isolated footings is based on a minimum width of 24 inches and founded a minimum of 12 inches. The above value may be increased by 200 psf and 600 psf for each additional foot in width and depth, respectively, up to a maximum value of 3,200 psf. Recommended allowable bearing values include both dead and live loads and may be increased by one-third for wind and seismic forces.

6.4.5 Lateral Resistance

Provided site grading is performed in accordance with the recommendations provided by the project geotechnical consultant, a passive earth pressure of 240 pounds per square foot per foot of depth up to a maximum value of 2,000 pounds per square foot may be used to determine lateral bearing for beams. This value may be increased by one-third when designing for wind and seismic forces. A coefficient of friction of 0.30 times the dead load forces may also be used between concrete and the supporting soils to determine lateral sliding resistance. No increase in the coefficient of friction should be used when designing for wind and seismic forces. Where lateral removals cannot be performed, the above-noted values should be decreased by 50%.

The above values are based on foundations placed directly against compacted fill. In the case where footing sides are formed, all backfill against the foundations should be compacted to at least 90 percent of the laboratory standard.

6.4.6 Post-Tensioned Slab/Mat on Grade

The proposed structures may be supported by a post-tension slab. Perimeter edge beams for the post-tensioned slabs should have a minimum effective width of 12 inches and be founded at a minimum depth of 18 inches below the lowest adjacent final ground surface. Interior beams may be founded at a minimum depth of 12 inches below the tops of the finish floor slabs. Where a post-tensioned mat is utilized, the exterior edge of the mat should be embedded at least 8 inches below the lowest adjacent grade. The thickness of the floor slab/mat should be determined by the project structural engineer; however, we recommend a minimum slab thickness of 5.0 inches.

Design of the mat may be based on a modulus of subgrade reaction (Kv1) of 54 pounds per cubic inch (pci). The modulus is based on an effective loading area of 1 foot by 1 foot. The modulus may be adjusted for other effective loading areas using the equation provided below.

$$k_b(pci) = 54\left\{\frac{b+1}{2b}\right\}^2$$

where "b" is the effective width of loading (minimum dimension) in feet.

Concrete floor slabs in areas to receive carpet, tile, or other moisture sensitive coverings should be underlain with a minimum of 10-mil moisture vapor retarder conforming to ASTM E 1745, Class A. The membrane should be properly lapped, sealed, and underlain within a layer of sand at least 4 inches thick. Where a mat is used and has a thickness of at least 8 inches, the sand may be limited to 2 inches. One inch of sand may be placed over the membrane to aid in the curing of the concrete. The sand should have a SE no less than 30. This vapor retarder system is anticipated to be suitable for most flooring finishes that can accommodate some vapor emissions. However, this system may emit more than 4 pounds of water per 1000 sq. ft. and therefore, may not be suitable for all flooring finishes. Additional steps should be taken if such vapor emission levels are too high for anticipated flooring finishes.

Prior to placing concrete, subgrade soils below slab-on-grade/mat areas should be thoroughly moistened to provide moisture contents at least 110 percent of the optimum moisture content to a depth of 12 inches.

Based on the guidelines provided in the "Design of Post-Tensioned Slabs-on-Ground" 3rd Edition by Post-Tensioning Institute, the em and ym values are summarized in Table 6.2.

TABLE 6.2
PTI Design Parameters (EI<51)

Parameter	Value
Edge Lift Moisture Variation Distance, e_m	4.3 feet
Edge Lift, y_m	1.419 inches
Center Lift Moisture Variation Distance, e_m	8.2 feet
Center Lift, y_m	0.922 inches

6.4.7 Foundation Observations

Foundation excavations should be observed by the project geotechnical consultant to verify that they have been excavated into competent bearing soils and to the minimum embedment recommended above. These observations should be performed prior to placement of forms or reinforcement. The excavations should be trimmed neat, level and square. Loose, sloughed or moisture-softened materials and debris should be removed prior to placing concrete.

6.5 RETAINING AND SCREENING WALLS

6.5.1 General

The following preliminary design and construction recommendations are provided for general retaining and screen walls. Final wall designs specific to the site development should be provided to project geotechnical consultant for review once completed. The structural engineer and architect should provide appropriate recommendations for sealing at all joints and applying moisture-proofing material on the back of the walls.

6.5.2 Allowable Bearing Value and Lateral Resistance

Provided site grading is performed as recommended herein, the values for bearing and lateral resistance provided in Sections 6.4.4 and 6.4.5 may be utilized in design of retaining and screen walls.

The coefficient of friction should not be applied to portions of the footing in front of keyways used for passive resistance. These values should be reduced by 50% for walls along property lines.

The above values are based on footings placed directly against properly compacted fill. In the case where footing sides are formed, all backfill against the footings should be compacted to at least 90 percent of the laboratory standard.

6.5.3 Earth Pressures

Static and seismic earth pressures for level and 2:1 (H:V) backfill conditions are provided in Table 6.3. Seismic earth pressures provided herein are based on the method provided by Seed & Whitman (1970) using a peak ground acceleration (PGA) of 0.47 g for 10% probability of exceedance in 50 years. As indicated in Section 1803.5.12 of the 2019 CBC, retaining walls supporting 6 feet of backfill or less are not required to be designed for seismic earth pressures. The values provided in the following table do not consider hydrostatic pressure. Retaining walls should also be designed to support adjacent surcharge loads imposed by other nearby footings or traffic loads in addition to the earth pressure.

6.5.4 Drainage and Moisture-Proofing

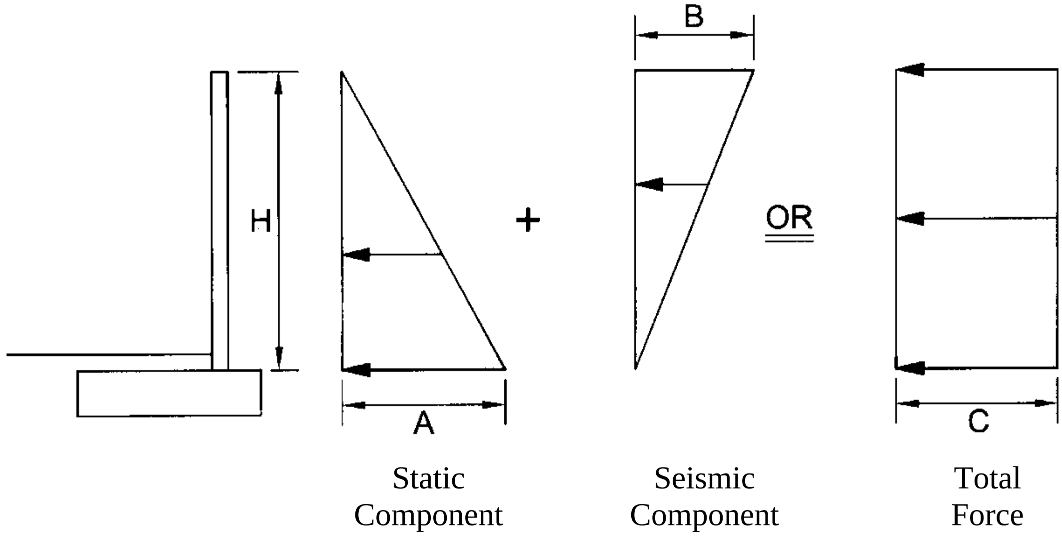
Retaining walls should be constructed with a perforated pipe and gravel subdrain to prevent entrapment of water in the backfill. The perforated pipe should consist of 4-inch-diameter, ABS SDR-35 or PVC Schedule 40 with the perforations laid down. The pipe should be embedded in ¾- to 1½-inch open-graded gravel wrapped in filter fabric. The gravel should be at least one foot wide and extend at least one foot up the wall above the footing and drainage outlet. Drainage gravel and piping should not be placed below outlets and weepholes. Filter fabric should consist of Mirafi 140N, or equal. Outlet pipes should be directed to positive drainage devices.

The use of weepholes may be considered in locations where aesthetic issues from potential nuisance water are not a concern. Weepholes should be 2 inches in diameter and provided at least every 6 feet on center. Where weepholes are used, perforated pipe may be omitted from the gravel subdrain.

Retaining walls supporting backfill should also be coated with a moisture-proofing compound or covered with such material to inhibit infiltration of moisture through the walls. Moisture-proofing material should cover any portion of the back of wall that will be in contact with soil and should lap over and onto the top of footing. A drainage panel should be provided between the soil backfill and water proofing. The panel should extend from the top of the backdrain gravel up to within 12 inches of finish grade. The top of footing should be finished smooth with a trowel to inhibit the infiltration of water through the wall. The project structural engineer should provide specific recommendations for moisture-proofing, water stops, and joint details.

TABLE 6.3

SEISMIC EARTH PRESSURES
Pressure Diagram



Earth Pressure Values
Walls Up to 10 Feet in Height

Value	**Backfill Condition**	
	Level	**2H:1V Slope**
A	46H	75H
B	15H	15H
C	31H	45H

Note:

H is in feet and resulting pressure is in psf. Design may utilize either the sum of the static component and the seismic component force diagrams or the total force diagram above. SEAOSC has suggested using a load factor of 1.7 for the static component and 1.0 for the seismic component. The actual load factors should be determined by the structural engineer.

6.5.5 Footing Reinforcement

All continuous footings should be reinforced with a minimum of two No. 4 bars, one top and one bottom. The structural engineer may require different reinforcement and should dictate if greater than the recommendations provided herein.

6.5.6 Wall Jointing

All free-standing, exterior site walls should be provided with cold joints through the masonry block section at horizontal spacing generally not exceeding 20 feet. The joints should not extend through the footing. Retaining walls that are integral to the building should be provided joints based on recommendations by the structural engineer.

6.5.7 Footing Observations

Footing excavations should be observed by the project geotechnical consultant to verify that they have been excavated into competent bearing soils and to the minimum embedment recommended herein. These observations should be performed prior to placement of forms or reinforcement. The excavations should be trimmed neat, level, and square. Loose, sloughed or moisture-softened materials and debris should be removed prior to placing concrete.

6.5.8 Wall Backfill

Onsite soils may be used for backfill behind retaining walls. The project geotechnical consultant should approve the backfill used for retaining walls. Wall backfill should be thoroughly moistened to provide moisture contents slightly over optimum moisture content; placed in lifts no greater than 12 inches in thickness, and then mechanically compacted with appropriate equipment to at least 90 percent of the laboratory standard. Hand-operated compaction equipment should be used to compact the backfill placed immediately adjacent the wall to avoid damage to the wall.

6.6 EXTERIOR FLATWORK

Exterior flatwork should be a minimum 4 inches thick. Cold joints or saw cuts should be provided at least every 15 feet in each direction. Flatwork more than 7 feet in width across the minimum dimension should be reinforced with 6” by 6”, W4 by W4 welded wire mesh or No 3 bars spaced 12 inches center to center in both directions. Special jointing details should be provided in areas of block-outs, notches, or other irregularities to avoid cracking at points of high stress. Subgrade soils below flatwork should be thoroughly moistened to a moisture content of at least 110 percent of optimum to a depth of 12 inches. Moistening should be accomplished by lightly spraying the area over a period of a few days just prior to pouring concrete. The geotechnical consultant should observe and verify the density and moisture content of subgrade soils prior to pouring concrete to verify the recommended pre-moistening recommendations have been met.

Drainage from flatwork areas should be directed to local area drains or other appropriate collection devices designed to carry runoff water to the street or other approved drainage structures.

6.7 CONCRETE MIX DESIGN AND CORROSION

Laboratory testing of existing near-surface soils for soluble sulfate content indicates soluble sulfate concentration less than 0.10%. We recommend following the procedures provided in ACI 318, Section 4.3, Table 4.3.1 for **negligible** sulfate exposure. Upon completion of rough grading, an evaluation of as-graded conditions and further laboratory testing should be completed for the site to confirm or modify the recommendations provided in this section.

6.8 POST GRADING CONSIDERATIONS

6.8.1 Site Drainage and Irrigation

Positive drainage devices, such as sloping concrete flatwork, graded swales or area drains, should be provided around the new construction to collect and direct all surface water to suitable discharge areas. In general, the site should be graded to conform to the requirements of Section 1804.4 of the 2019 California Building Code. No rain or excess water should be directed toward or allowed to pond against structures such as walls, foundations, flatwork, etc.

Excessive irrigation water can be detrimental to the performance of the proposed site development. Water applied in excess of the needs of vegetation will tend to percolate into the ground. Such percolation can lead to nuisance seepage and shallow perched groundwater. Seepage can form on slope faces, on the faces of retaining walls, in streets, or other low-lying areas. These conditions could lead to adverse effects such as the formation of stagnant water that breeds insects, distress or damage of trees, surface erosion, slope instability, discoloration and salt buildup on wall faces, and premature failure of pavement. Excessive watering can also lead to elevated vapor emissions within buildings that can damage flooring finishes or lead to mold growth inside the home.

Key factors that can help mitigate the potential for adverse effects of overwatering include the judicious use of water for irrigation, use of irrigation systems that are appropriate for the type of vegetation and geometric configuration of the planted area, the use of soil amendments to enhance moisture retention, use of low-water demand vegetation, regular use of appropriate fertilizers, and seasonal adjustments of irrigation systems to match the water requirements of vegetation. Specific recommendations should be provided by a landscape architect or other knowledgeable professional.

6.8.2 Utility Trenches

Trench excavations should be constructed in accordance with the recommendations contained in Section 6.1.6 of this report. Trench excavations must also conform to the requirements of Cal/OSHA.

Trench backfill materials and compaction criteria should conform to the requirements of the local municipalities. As a minimum, utility trench backfill should be compacted to at least 90 percent of the laboratory standard. Trench backfill should be brought to moisture content slightly over optimum, placed in lifts no greater than 12 inches in thickness, and then mechanically compacted with appropriate equipment to at least 90 percent of the laboratory standard. The project geotechnical consultant should perform density testing, along with probing, to test compaction. Jetting should not be completed without prior approval from the project geotechnical consultant.

Within shallow trenches (less than 18 inches deep) where pipes may be damaged by heavy compaction equipment, imported clean sand having a SE of 30 or greater may be utilized. The sand should be placed in the trench, thoroughly watered, and then compacted with a vibratory compactor. For utility trenches located below a 1:1 (H:V) plane projecting downward from the outside edge of the adjacent footing base or crossing footing trenches, concrete or slurry should be used as trench backfill.

6.9 PRELIMINARY PAVEMENT DESIGN

6.9.1 Preliminary Structural Sections

Based on the soil conditions present at the site and estimated traffic indices, preliminary pavement sections are provided in Table 6.4 below. A preliminary "R-value" of 5 was used for the near-surface soil in this preliminary pavement design. The sections provided below are for planning purposes only and should be re-evaluated subsequent to site grading. Final pavement sections should be based on actual R-value testing of in-place soils and analysis of anticipated traffic.

TABLE 6.4
PRELIMINARY PAVEMENT STRUCTURAL SECTIONS
FOR RESIDENTIAL DEVELOPMENT

Location	Traffic Index	AC (inches)	Paver Thickness (mm)	Portland Cement Concrete (inches)	AB (inches)
Main Street	5.5	3.0 4.0	--	--	13.0 10.0
		--	80	--	14.0
		---	--	8.0	---
Parking Stalls	--	3.0		---	6.0

6.9.2 Subgrade Preparation

Prior to placement of pavement elements, subgrade soils should be moisture-conditioned to at least 110 percent of the optimum moisture content then compacted to at least 90 percent of the laboratory determined maximum dry density. Areas observed to pump or yield under vehicle traffic should be removed and replaced with firm and unyielding compacted soil or aggregate base materials.

6.9.3 Aggregate Base

Aggregate base should be moisture conditioned to slightly over the optimum moisture content, placed in lifts no greater than 6 inches in thickness, then compacted to at least 95 percent of the laboratory standard (ASTM D 1557). Aggregate base materials should be Class 2 Aggregate Base conforming to Section 26-1 of the latest edition of the Caltrans Standard Specifications, Crushed Aggregate Base conforming to Section 200-2.2 of the latest edition of the Standard Specifications for Public Works Construction (Greenbook) or Crushed Miscellaneous Base conforming to Section 200-2.4 of the Greenbook.

6.9.4 Asphaltic Concrete

Paving asphalt should be PG 64-10. Asphaltic concrete materials should conform to Section 203-6 of the Greenbook and construction should conform to Section 302 of the Greenbook. Where traffic will traverse over cold joints in asphaltic concrete such as against concrete ribbon gutters and concrete paver sections, the asphaltic concrete section should be thickened by 1 additional inch from the values indicated in the above Table 6.3 within 2 feet of cold joints.

6.9.5 Concrete Pavers

Concrete pavers should conform to the requirements of ASTM C 936. Construction of the pavers, including bedding sand, should follow manufacturer's specifications. Typical thickness of bedding sand is about 1 inch. The gradation of bedding sand should meet the requirement in Table 6.5.

TABLE 6.5
Gradation for Sand Bedding

Sieve Size	Percent Passing
⅜"	100
No. 4	95 - 100
No. 8	80 - 100
No. 16	50 - 85
No. 30	25 - 60
No. 50	5 - 30
No. 100	0 - 10
No. 200	0 - 1

Construction of edge restraints should also follow manufacturer's specifications. As a minimum, restraints should be provided along the perimeter of concrete pavers and where there is a change in the paving materials. The proposed concrete bands should extend to the bottom of the base course underlying the concrete pavers. Portland cement concrete used to construct concrete bands should conform to Section 201 of the Greenbook and should have a minimum compressive strength of 2,500 pounds per square inch (psi) at 28 days. Reinforcement and jointing of concrete pavement sections should be designed according to the minimum recommendations provided by the Portland Cement Association (PCA). For rigid pavement, transverse and longitudinal contraction joints should be provided at spacing no greater than 15 feet. Score joints may be constructed by saw cutting to a depth of ¼ of the slab thickness. Expansion/cold joints may be used in lieu of score joints. However, cold joints should be provided with dowels or keyways are recommended by PCA.

6.9.6 Portland Cement Concrete (PCC)

Portland cement concrete used to construct concrete paving should conform to Section 201 of the Greenbook and should have a minimum compressive strength of 3,500 pounds per square inch (psi) at 28 days. Reinforcement and jointing of concrete pavement sections should be designed according to the minimum recommendations provided by the Portland Cement Association (PCA). For rigid pavement, transverse and longitudinal contraction joints should be provided at spacing no greater than 15 feet. Score joints may be constructed by saw cutting to a depth of ¼ of the slab thickness. Expansion/cold joints may be used in lieu of score joints. Such joints should be properly sealed. Where traffic will traverse over cold joints or edges of concrete paving, the edges should be thickened by 20% of the design thickness toward the edge over a horizontal distance of 5 feet.

Trash pickup areas should be provided with a concrete slab where the bins will be picked up and extend at least 3 feet past the front wheel landing areas. The slab should be at least 8 inches thick and be reinforced with No. 4 bars spaced at 24 inches on centers, both ways. The slabs should be provided transverse and longitudinal joints spacing as specified above. Dowels or a keyway should be provided at all cold joints.

6.10 PLAN REVIEW AND CONSTRUCTION SERVICES

We recommend *Albus-Keefe & Associates, Inc*. be engaged to review any future development plans, including revisions to the grading plans, foundation plans and proposed structural loads, prior to construction. This is to verify that the assumptions of this report are valid and that the preliminary conclusions and recommendations contained in this report have been properly interpreted and are incorporated into the project plans and specifications. If we are not provided the opportunity to review these documents, we take no responsibility for misinterpretation of our preliminary conclusions and recommendations.

We recommend that a geotechnical consultant be retained to provide soil engineering services during construction of the project. These services are to observe compliance with the design, specifications or recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

If the project plans change significantly from the assumed development described herein, the project geotechnical consultant should review our preliminary design recommendations and their applicability to the revised construction. If conditions are encountered during construction that appear to be different than those indicated in this report or subsequent design reports, the project geotechnical consultant should be notified immediately. Design and construction revisions may be required.

7.0 LIMITATIONS

This report is based on the proposed development and geotechnical data as described herein. The materials described herein and in other literature are believed representative of the total project area, and the conclusions contained in this report are presented on that basis. However, soil materials can vary in characteristics between points of exploration, both laterally and vertically, and those variations could affect the conclusions and recommendations contained herein. As such, observation and testing by a geotechnical consultant prior to and during the grading and construction phases of the project are essential to confirming the basis of this report.

This report summarizes several geotechnical topics that should be beneficial for project planning and budgetary evaluations. The information presented herein is intended only for a preliminary feasibility evaluation and is not intended to satisfy the requirements of a site specific and detailed geotechnical investigation required for further planning and permitting.

This report has been prepared consistent with that level of care being provided by other professionals providing similar services at the same locale and time period. The contents of this report are professional opinions and as such, are not to be considered a guaranty or warranty.

This report should be reviewed and updated after a period of one year or if the site ownership or project concept changes from that described herein.

This report has been prepared for the exclusive use of **The Olson Company** to assist the project consultants in determining the feasibility of the proposed development. This report has not been prepared for use by parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

Respectfully submitted,

ALBUS-KEEFE & ASSOCIATES, INC

Paul Hyun Jin Kim
Associate Engineer
GE 3106

REFERENCES

Publications

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Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J., Dobry, R., Finn, W.D.L., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., and Stokoe, K.H., "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils", Journal of Geotechnical and Geoenvironmental Engineering, October, 2001.

Plans

Tentative Tract No. 82945, For Condominium Purposes, A Vesting Tentative Tract Map Conceptual Grading, dated November 15, 2019, Scale: 1" = 20', Sheet 2 of 2.

0 20 40 80

APPROX SCALE : 1" = 40'

EXPLANATION

(Locations Approximate)

- Exploratory Boring
- Exploratory Percolation Test Boring

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

GEOTECHNICAL MAP

Job No.: 2810.00 | Date: 05/22/19 | Plate: 1

APPENDIX A

EXPLORATION LOGS

EXPLORATION LOG

Project:		Location:
Address:		Elevation:
Job Number:	Client:	Date:
Drill Method:	Driving Weight:	Logged By:

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		EXPLANATION							
		Solid lines separate geologic units and/or material types.							
5		Dashed lines indicate unknown depth of geologic unit change or material type change.							
		Solid black rectangle in Core column represents California Split Spoon sampler (2.5in ID, 3in OD).							
		Double triangle in core column represents SPT sampler.							
10		**Vertical Lines** in core column represents Shelby sampler.							
		Solid black rectangle in Bulk column respresents large bag sample.							
15		**Other Laboratory Tests:** Max = Maximum Dry Density/Optimum Moisture Content EI = Expansion Index SO4 = Soluble Sulfate Content DSR = Direct Shear, Remolded DS = Direct Shear, Undisturbed SA = Sieve Analysis (1" through #200 sieve) Hydro = Particle Size Analysis (SA with Hydrometer)							
20		200 = Percent Passing #200 Sieve Consol = Consolidation SE = Sand Equivalent Rval = R-Value ATT = Atterberg Limits							

EXPLORATION LOG

Project: Olson - Gardena		Location: B-1
Address: 1343 W 141st St, Gardena, CA 90247		Elevation: 52.9
Job Number: 2810.00	Client: The Olson Company	Date: 5/6/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: SD

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ARTIFICIAL FILL (Af) Silty Sand (SM): Dark reddish brown, damp, dense, fine to medium grained sand							DS
		ALLUVIUM (Qal) Sandy Clay (CL): Mottled reddish brown and dark brown, damp, hard, fine to medium grained sand, pockets of fine to medium grained sand, trace gravel, trace pores		62			14.4	116.3	
5				63			14	117.9	
		Silty Sand (SM): Light brown, moist, medium dense, fine to medium grained sand		36			11.5	115.9	Consol
		Sandy Clay (CL): Brown, moist, very stiff, some pores, mica present, trace silt							
10		Clayey Sand/ Sandy Clay (SC/CL): Brown, moist, dense/hard, fine to medium grained sand, trace pinhole pores, mica present		54			13	119.9	
		Silty Sand (SM): Brown, damp, dense, fine to coarse grained sand, mica present							
15				30					SA Hydro
		Clayey Sand (CL): Reddish brown, moist, dense/hard, fine to medium grained sand, mica present, some silt, increased sand toward sampler tip							
20				26					SA Hydro
		Silty Sand (SM): Grayish brown, moist, dense, fine grained sand							
25				31					
		Sand (SP): Yellowish brown, moist to very moist, dense, fine to medium grained sand, iron oxide							

Albus-Keefe & Associates, Inc.

Plate A-2

EXPLORATION LOG

Project: Olson - Gardena		Location: B-1
Address: 1343 W 141st St, Gardena, CA 90247		Elevation: 52.9
Job Number: 2810.00	Client: The Olson Company	Date: 5/6/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: SD

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
				25					
		Lean Clay (CL): Reddish brown, moist, hard, mica present, iron oxide							ATT
35		Clay (CL): Reddish brown, moist, hard, fine grained sand, mica present, iron oxide, some silt		21					
40		Clay with Sand (CL): Grayish brown, moist, hard, fine to medium grained sand, mica present		30					
45		Sand with Silt (SP-SM): Grayish brown, moist, very dense, fine to medium grained sand, mica present, some silt		69					
50		Lean Clay (CL): Dark brown, moist, hard, fine to medium grained sand, mica present		27					ATT
		End of boring at depth of 51.5. Groundwater observed at depth of 30 ft. Backfilled with soil cutting.							

Albus-Keefe & Associates, Inc.

EXPLORATION LOG

Project: Olson - Gardena		Location: B-2
Address: 1343 W 141st St, Gardena, CA 90247		Elevation: 53.5
Job Number: 2810.00	Client: The Olson Company	Date: 5/6/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: SD

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Core	Bulk	Laboratory Tests: Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
		ARTIFICIAL FILL (Af) Sandy Clay (CL): Reddish dark brown, damp, hard, fine to medium grained sand							
		ALLUVIUM (Qal) Sandy Clay (CL): Reddish brown, damp, fine to medium grained sand, mica present		50			15.1	105.7	
5		@ 4 ft, increased sand		56			13.9	117.6	
				47			19.3	111.1	
10		@ 10 ft, hard, increased sand, decreased clay, some silt		42			19.5	110.8	
		Silty Sand (SM): Reddish brown, moist, dense, with clay							
15		@ 15.5 ft, yellowish brownincreased silt		31					
		Silt (ML): Light brown, moist, very stiff, mica present, iron oxide							
20		Sandy Clay (CL): Light brown, moist, very stiff, iron oxide		20					
		Sand (SP): Light brown, moist, medium dense, fine to medium grained sand							
		End of boring at depth of 21.5 ft. No groundwater encountered. Backfilled with soil cuttings.							

EXPLORATION LOG

Project: Olson - Gardena		Location: B-3	
Address: 1343 W 141st St, Gardena, CA 90247		Elevation: 53.1	
Job Number: 2810.00	Client: The Olson Company	Date: 5/6/2019	
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: SD	

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ARTIFICIAL FILL (Af) Sandy Clay (CL): Dark brown, damp, very stiff, coarse grained sand							
		ALLUVIUM (Qal) Sandy Clay (CL): Brown, damp, very stiff, fine to coarse grained sand, mica present, some fine gravel, trace debris (brick)		23			12.8	120.9	
5				55			16.8	113	Consol
		@ 4 ft, hard, mica presnt, some silt		50			13	118.1	
		@ 6 ft, increased sand, trace pores							
10		@ 10 ft, increased silt		43			15.8	116.4	
15		@ 15 ft, fine to medium grained sand		34					
		Silty Sand (SM): Light brown, damp, medium dense, mica present, iron oxide, some clay, increased sand toward sampler tip							
20				17					
		End of boring at depth of 21.5. No groundwater encountered. Backfilled with soil cutting.							

Albus-Keefe & Associates, Inc.

Plate A-5

EXPLORATION LOG

Project: Olson - Gardena		Location: B-4
Address: 1343 W 141st St, Gardena, CA 90247		Elevation: 53.6
Job Number: 2810.00	Client: The Olson Company	Date: 5/6/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: SD

Depth (feet)	Lith- ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ARTIFICIAL FILL (Af) Sandy Clay (CL): Dark brown, damp, very stiff, fine to medium grained sand							SO4 ATT pH Resist Ch
		ALLUVIUM (Qal) Sandy Clay (CL): Brown, damp, very stiff, fine to medium grained sand, mica present, trace pores, rootlets, increased sand toward sampler tip, with silt		24			17.9	109.6	
5				38			11.7	120	
		@ 4 ft, increased clay		30			13.7	116.5	
10		@ 10 ft, hard, pocket of fine to medium grained sand		53			14.8	117.4	
15		@ 15 ft, very stiff, fine to medium grained sand		20					
		Sand (SP): Mottled light brown and yellowish brown, moist, medium dense, fine to medium grained sand							
20		Clayey Sand/ Sandy Clay (SC/CL): Light brown, moist, medium dense/ very stiff, some pores, iron oxide,magnesium oxide, mica present		18					
		End of boring at depth of 21.5 ft. No groundwater encountered. Backfilled with soil cuttings.							

APPENDIX B

LABORATORY TEST PROGRAM

LABORATORY TESTING PROGRAM

Soil Classification

Soils encountered within the exploratory borings were initially classified in the field in general accordance with the visual-manual procedures of the Unified Soil Classification System (ASTM D 2487). The samples were re-examined in the laboratory and classifications reviewed and then revised where appropriate. The assigned group symbols are presented on the Exploration Logs provided in Appendix A.

In-Situ Moisture Content and Dry Density

Moisture content and dry density of in-place soil materials were determined in representative strata. Test data are presented on the Exploration Logs provided in Appendix A.

Laboratory Maximum Dry Density

Maximum dry density and optimum moisture content of onsite soils were determined for selected samples in general accordance with Method A of ASTM D 1557. Pertinent test values are given on Table B-1.

Direct Shear

The Coulomb shear strength parameters, angle of internal friction and cohesion, were determined for a bulk sample obtained from one our borings. The tests were performed in general conformance with Test Method ASTM D 3080. The sample was remolded to 90 percent of maximum dry density and at the optimum moisture content. Three specimens were prepared for each test, artificially saturated, and then sheared under varied loads at an appropriate constant rate of strain. Results are graphically presented on Plate B-5.

Soluble Sulfate Content

Chemical analysis was performed on selected samples to determine soluble sulfate content. The tests were performed in accordance with California Test Method No. 417. The test results are included on Table B-1.

Expansion Potential

An Expansion Index test was performed on a selected sample in accordance with ASTM D 4829. The test result and expansion potential are presented on Table B-1.

Atterberg Limits

Atterberg Limits (Liquid Limit, Plastic Limit, and Plasticity Index) were performed in accordance with Test Method ASTM D4318. Pertinent test values are presented within Table B-1.

Consolidation

Consolidation Tests were performed in general conformance with Test Method ASTM D 2435. Axial loads were applied in several increments to a laterally restrained 1-inch-thick sample. Loads were applied in geometric progression by doubling the previous load, and the resulting deformations were recorded at selected time intervals. The test samples were inundated at a selected surcharge loading in order to evaluate the effects of a sudden increase in moisture content. Results of these tests are graphically presented on Plates B-3 to B-4.

Corrosion

Select samples were tested for minimum resistivity and pH in accordance with California Test Method 643. Results of these tests are provided in Table B-1.

Particle-Size Analyses

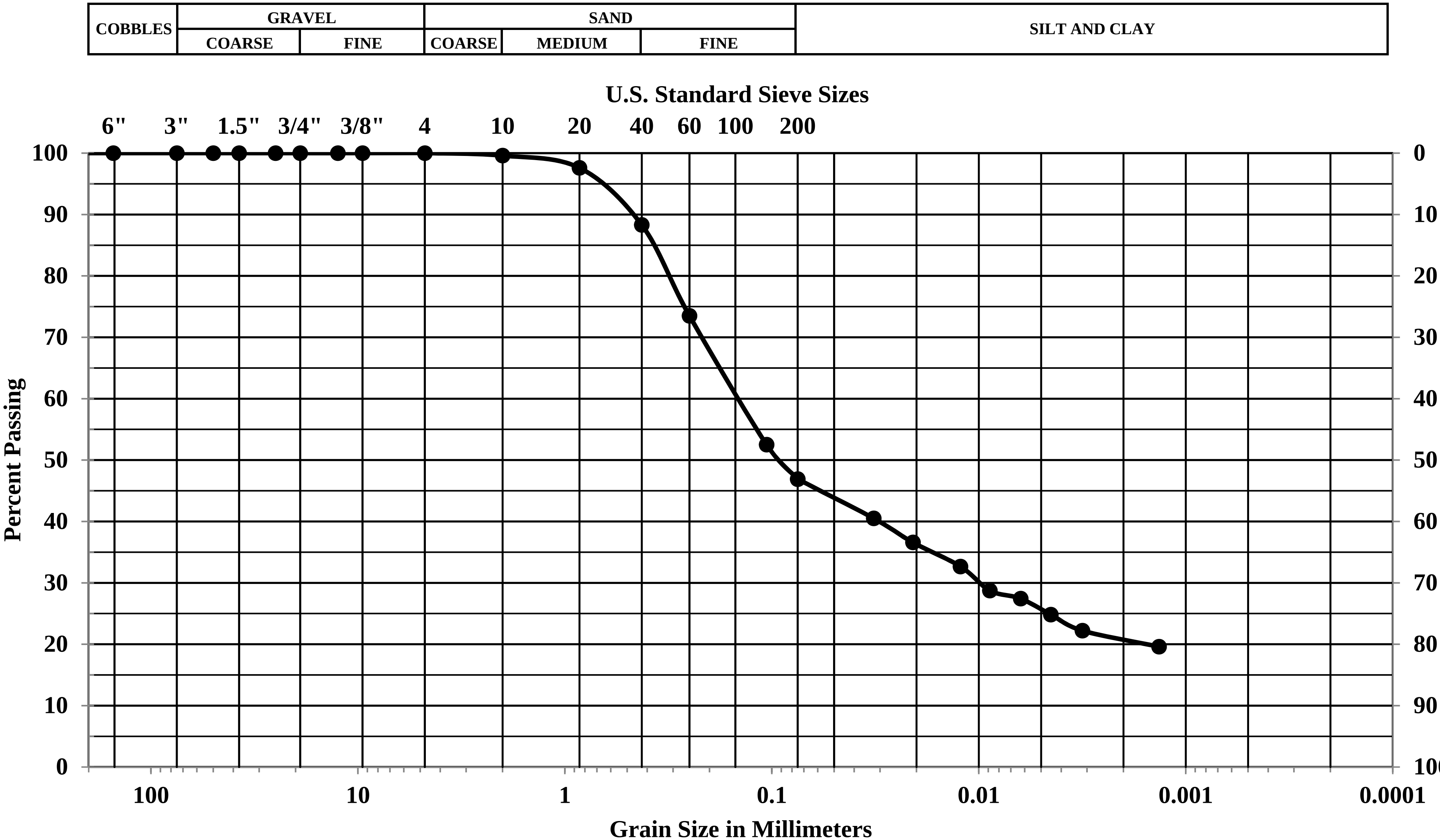
Particle-size analyses were performed on selected samples in accordance with ASTM D 422. The results are presented graphically on the attached Plates B-1 and B-2.

TABLE B-1
SUMMARY OF LABORATORY TEST RESULTS

Boring No.	Sample Depth (ft.)	Soil Description	Test Results	
B-1	0-5	Sandy Clay (CL)	Maxim Dry Density: Optimum Moisture Content:	128.5 pcf 10.0 %
B-1	31-31.5	Lean Clay (CL)	Liquid Limit: Plasticity Index:	39 17
B-1	50	Lean Clay (CL)	Liquid Limit: Plasticity Index:	32 16
B-4	0-5	Sandy Lean Clay (CL)	Liquid Limit: Plasticity Index: pH: Resistivity: Chloride: Expansion Index: Expansion Potential: Soluble Sulfate Content: Sulfate Exposure:	29 17 7.6 1,800 ohm-cm 9.0 ppm 39 Low 0.001% Negligible

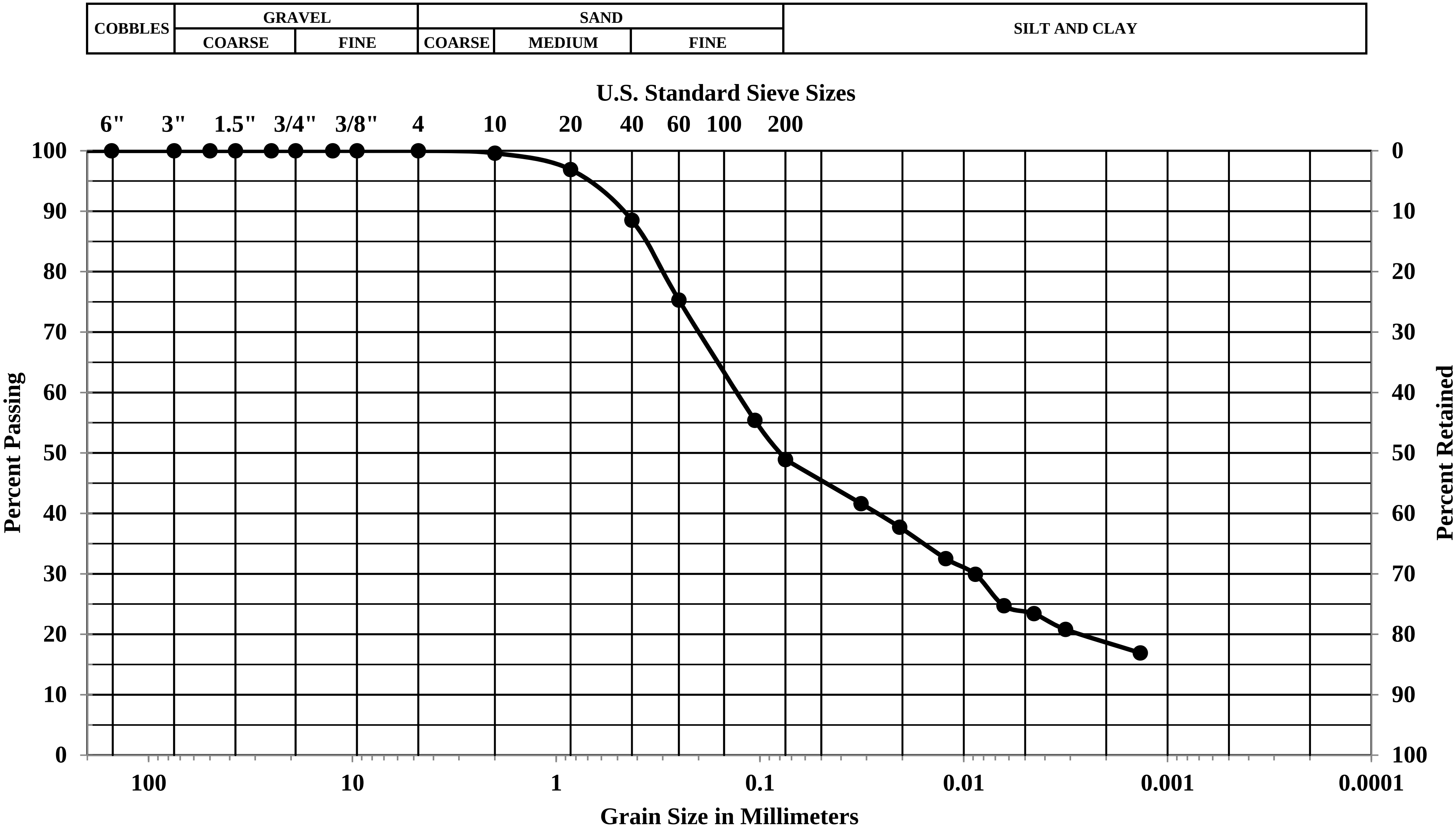
Note: Additional laboratory test results are provided on the boring logs in Appendix A.

GRAIN SIZE DISTRIBUTION



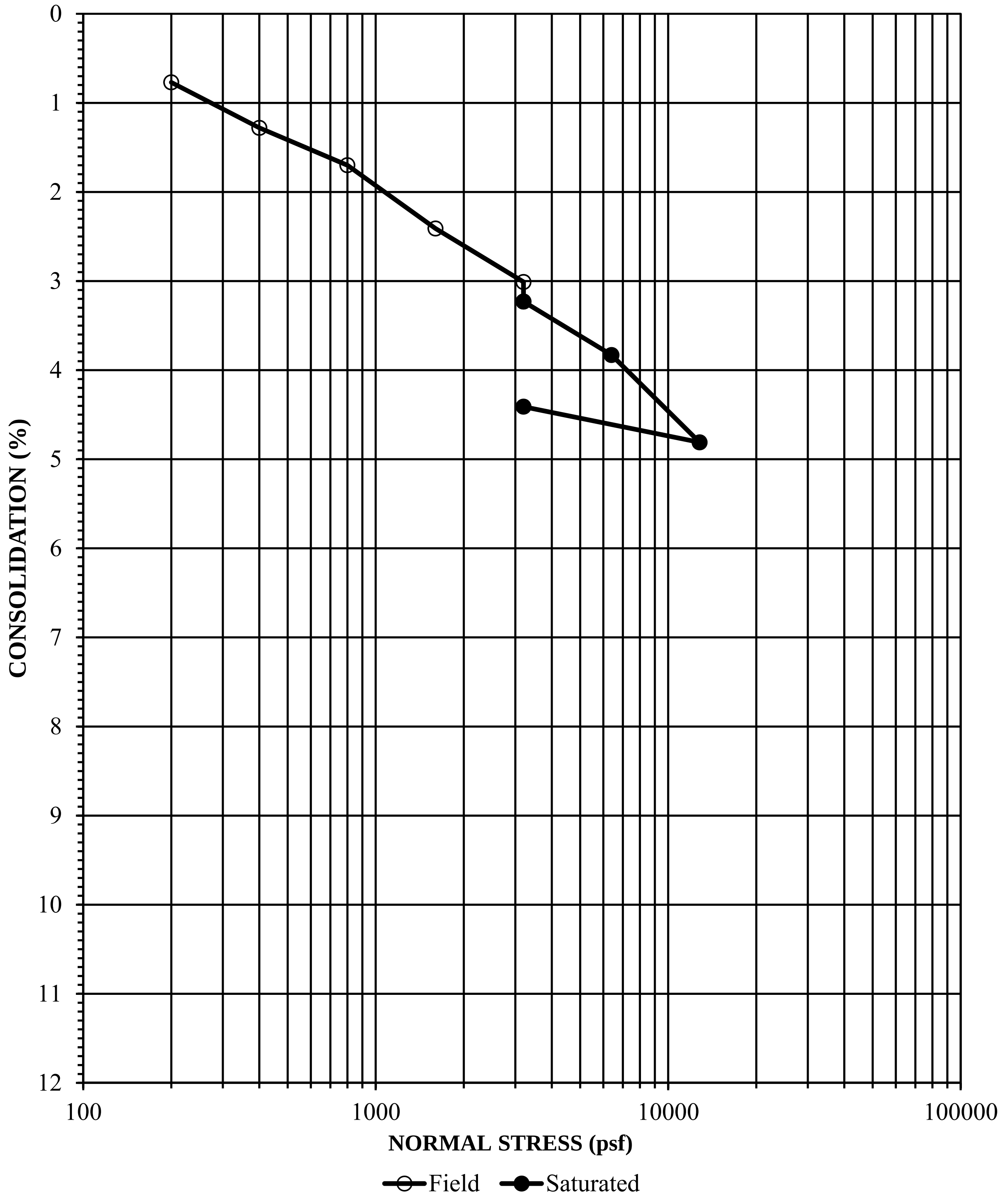
Job Number	Location	Depth	Description
2810.00	B-1	15	Clayey Sand (SC)

GRAIN SIZE DISTRIBUTION



Job Number	Location	Depth	Description
2810.00	B-1	20	Clayey Sand (SC)

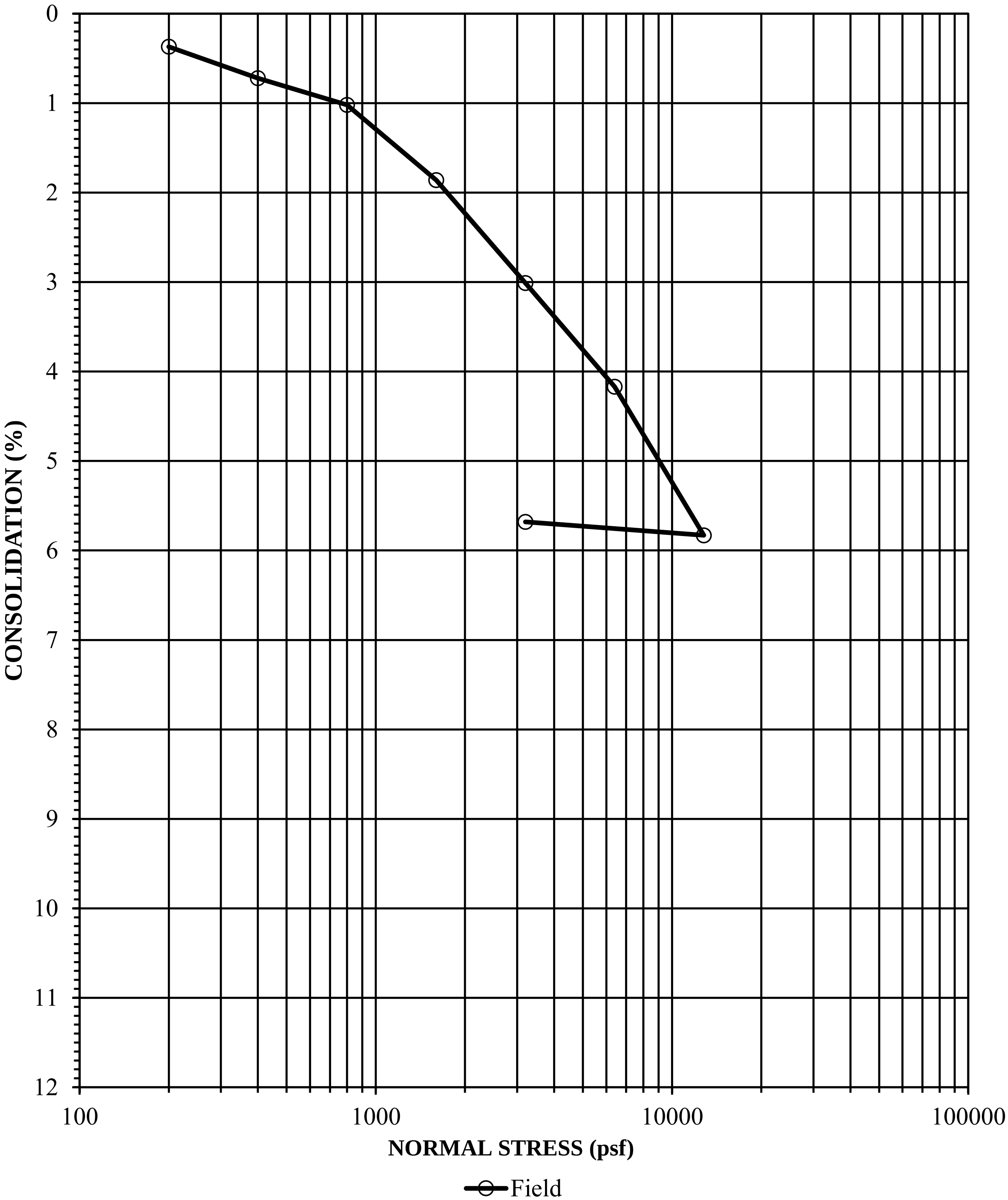
CONSOLIDATION



Job Number	Location	Depth	Description
2810.00	B-1	6	Silty Sand (SM)

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
115.9	11.5	16.3

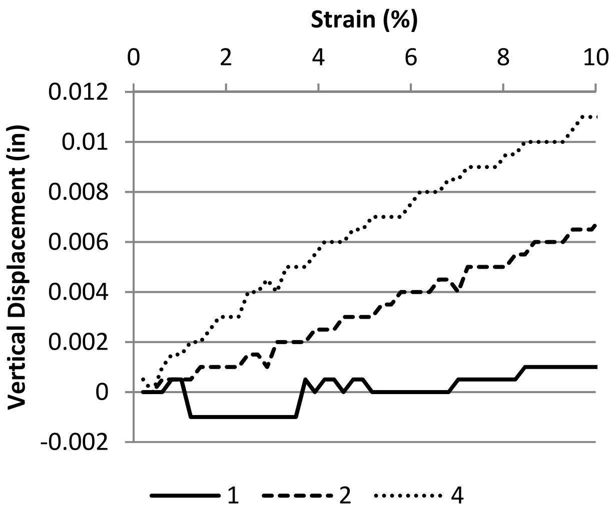
CONSOLIDATION

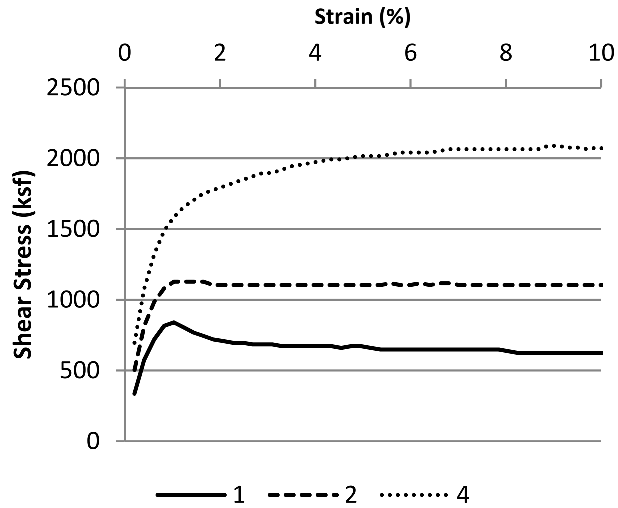


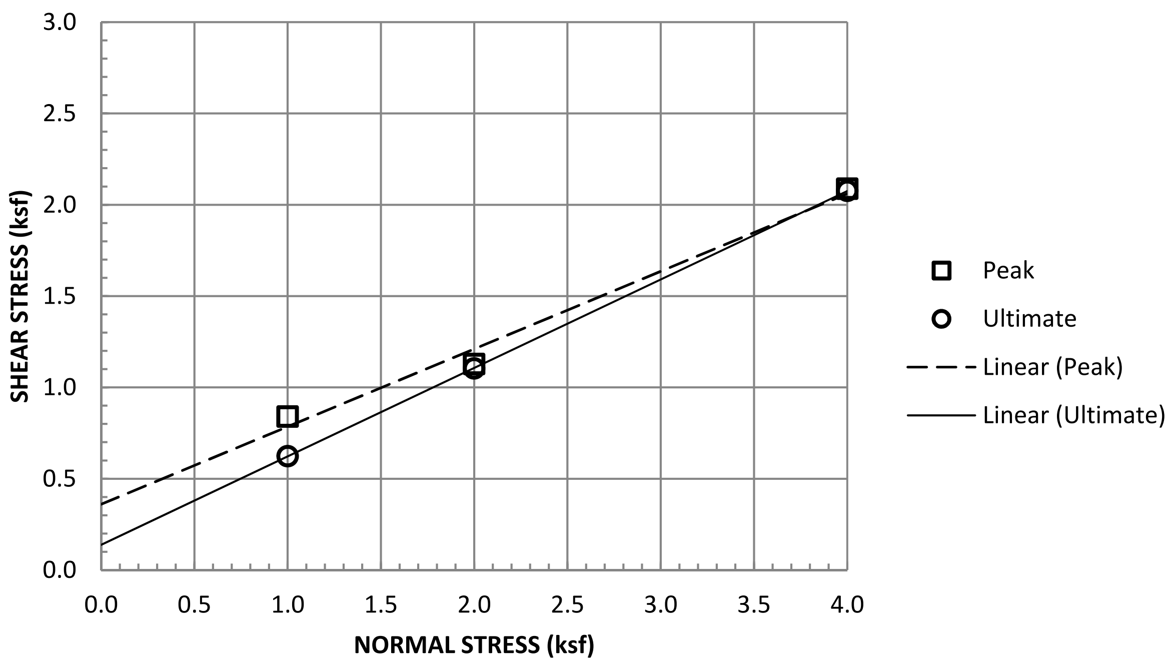
Job Number	Location	Depth	Description
2810.00	B-3	4	Sandy Clay (CL)

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
111.8	16.8	13.9

DIRECT SHEAR







Sample Type:	Remolded 90% of 128.5 @ 10%, Saturated		
Normal Stress (ksf)	1	2	4
Peak Shear Stress (ksf)	0.84	1.128	2.088
Peak Displacement (in)	0.002	0.007	0.011
Ultimate Shear Stress (ksf)	0.624	1.104	2.076
Ultimate Displacement (in)	0.25	0.25	0.25
Initial Dry Density (pcf)	115.7	115.7	115.7
Initial Moisture Content (%)	10	10	10
Final Moisture Content (%)	14.3	14.3	14.5
Strain Rate (in/min)	0.005		

Job Number	Location	Depth	Description
2810.00	B-1	0-5	Sandy Clay (CL)