



November 20, 2025

J.N.: 3374.00

Mr. Brian Cunningham
The Olson Company
3010 Old Ranch Parkway, Suite 100
Seal Beach, California 90740

Subject: Preliminary Geotechnical Investigation, Proposed Multi-Family Residential Development, 17332 Irvine Boulevard, Tustin, California

Dear Mr. Cunningham,

Pursuant to your request, *Albus & Associates, Inc.* is pleased to present to you our preliminary geotechnical investigation report for the proposed development at the subject site. This report presents the results of our aerial photo and literature review, subsurface exploration, laboratory testing, and engineering analyses. Conclusions and recommendations pertaining to 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 should have any questions regarding the contents of this report, please do not hesitate to call.

Sincerely,

ALBUS & ASSOCIATES, INC.

A handwritten signature in blue ink, appearing to read "Daniel Albus", is written over a horizontal line.

Daniel Albus
Project Engineer

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Plate 1 - Geotechnical Map

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Table B-1 – Summary of Laboratory Test Results

Plates B-1 through B-3– Grain Size Distribution Plots

Plate B-4 – Direct Shear Plot

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purposes of our preliminary geotechnical investigation were to evaluate geotechnical conditions within the project area and to provide conclusions and recommendations relevant to the design and construction of the proposed improvements at the subject site. The scope of this investigation included the following:

- Review of the historical aerial photographs
- Review of published geologic and seismic data for the site and surrounding area
- Exploratory drilling and soil sampling
- Laboratory testing of selected soil samples
- Engineering analyses of data obtained from our review, exploration, and laboratory testing
- Evaluation of site seismicity, liquefaction potential, and settlement potential
- Preparation of this report

1.2 SITE LOCATION AND DESCRIPTION

The site is located at 17332 Irvine Boulevard in the city of Tustin, California. The property is bordered by Irvine Boulevard to the north, Yorba Street to the west, Mountain View Drive to the east, and residential properties to the south. The location of the site and its relationship to the surrounding areas are shown in Figure 1, Site Location Map.

The site consists of one rectangularly-shaped property containing approximately 1 acre of land in total. The site is relatively flat, with elevations ranging from 146 to 148 feet above mean sea level (based on Google Earth). Site drainage appears to be directed as sheet flow generally to the east.

The site is occupied by a large commercial building occupying the northern half of the site. The remainder of the site is covered in asphalt driveways and parking bays associated with the building. Light vegetation and moderate size trees are also present onsite within scattered planter spaces. A masonry block wall is present along the south property line. Other improvements are present in association with the existing building.

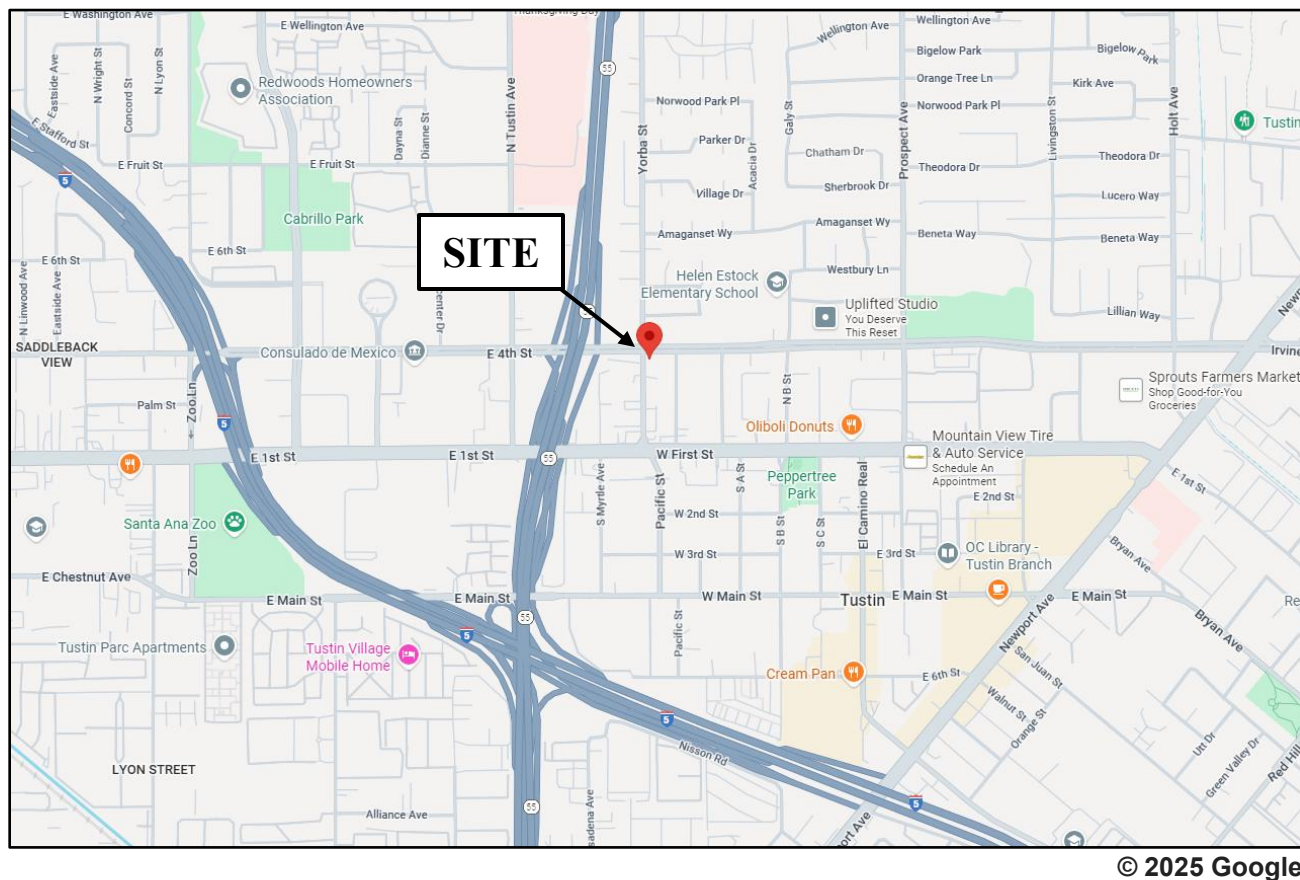


FIGURE 1 - SITE LOCATION MAP

**Proposed Multi-Family Residential Development
17332 Irvine Boulevard, Tustin, California**

NOT TO SCALE

1.3 PROPOSED DEVELOPMENT

Based on the conceptual site plan provided by ktgy, we understand that the proposed development will likely consist of 15 three-story residential townhomes at-grade. Associated interior driveways, parking, underground utilities, and decorative hardscape and landscape areas are also anticipated. No structural plans were available for our review at this time.

No grading or structural plans were available for review at the time of this report. However, we anticipate grading will entail some minor cuts and fills of the site achieve future surface configuration, and we expect future foundation loads will be moderate.

2.0 INVESTIGATION

2.1 RESEARCH

Review of the Geologic Map of Orange County by Morton (1981) suggests the site is underlain by Younger Alluvium (Qya). Older Alluvium (Qoa) has been mapped relatively close to the site to the north and northeast. The Qoa may be present at depth below the stie.

Historical imagery shows that the site was covered by mature citrus trees as early as 1946 and Irvine Boulevard was not present. By 1963, Irvine Boulevard is present and the site is still occupied by fruit trees. By 1965, the site has been cleared of the trees are rough graded. By 1972, the site has been developed into it's current configuration.

2.2 SUBSURFACE EXPLORATION

Subsurface exploration for this investigation was conducted on November 6, 2025, that consisted of advancing three (3) soil borings to approximate depths of 11.5 to 51.5 feet below the existing ground surface (bgs). The borings were drilled using a truck-mounted, continuous flight, hollow-stem-auger drill rig. A representative of Albus & Associates, Inc., was present to log the exploratory borings. Visual and tactile identifications were made of the materials encountered, and their descriptions are presented in 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 borings for subsequent laboratory testing. A California split-spoon soil sampler with a 3-inch O.D., 2.5-inch I.D., lined with brass rings, was used to gather soil samples. 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 analysis. The borings were backfilled with auger cuttings upon completion of sampling and capped with cold-patch asphalt.

Two additional borings (P-1 and P-2) were drilled adjacent to boring B-1 for percolation testing. A brief summary of our findings are presented below in Section 5.7.

2.3 LABORATORY TESTING

Selected samples obtained from our subsurface exploration were tested in our soil laboratory. Testing consisted of in-situ moisture content and dry density, maximum dry density and optimum moisture content, soluble sulfate, expansion index, direct shear, grain-size distribution analysis, pH, chloride content, and minimum resistivity. Descriptions of laboratory test criteria and test results are presented in Appendix B.

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.

From our exploration of the site, we found artificial fills (Af) to be generally 4 feet thick within all the borings. The fill encountered onsite is likely a result of past site development. These materials consisted of gray to red brown sands and silty sands that were loose, dry to moist and fine-grained. Localized, deeper fills may be present onsite in the form of utility trenches or from past grading activities.

Younger Alluvium (Qal) was encountered underlying the artificial fill and extended to a depth of 16 feet. The alluvium typically consisted of fine to coarse-grained sands that became gravelly below a depth of 10 feet. The younger alluvial soils were light reddish brown, typically loose to medium dense, and moist.

Below the Younger Alluvium, soils believed to be Older Alluvium (Qoal) were encountered between the depths of 16 feet and to the maximum depth explored of 41.5 feet. These materials consisted of silty fine sand to 30 feet then became a sandy silt with a trace clay. The older alluvium was reddish brown, medium dense/stiff, and moist. These materials were believed to be older alluvium primarily based on the deeper red coloring (increased oxidation) and textural features along with the general mapping of older alluvial soils exposed in other nearby areas.

3.2 GROUNDWATER

Groundwater was not encountered during this firm's subsurface exploration to a depth of 41.5 feet. Estimating from data in CDMG Special Report 012, the historic high groundwater for the subject site is mapped at a depth of about 50 feet.

3.3 FAULTING

Based on our review of the referenced publications and seismic data, no active faults are known to project through or immediately adjacent to the subject sites and the sites do not lie within an "Earthquake Fault Zone" as defined by the State of California in Earthquake Fault Zoning Act. Table 3.1 presents a summary of known seismically active faults within 10 miles of the sites based on the 2008 USGS National Seismic Hazard Maps.

TABLE 3.1
Summary of Faults

Name	Dist. (miles)	Slip Rate (mm/yr.)	Preferred Dip (degrees)	Slip Sense	Rupture Top (km)	Fault Length (km)
<u>San Joaquin Hills</u>	4.09	0.5	23	Thrust	2	27

4.0 ANALYSES

4.1 SEISMICITY

2022 CBC requires seismic parameters in accordance with ASCE 7-16 and supplements 1, 2, and 3. 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 the OSHPD seismic hazard tool 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 and supplement 3 require site-specific ground hazard analysis for structures on Site Class D 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. However, “A ground motion hazard analysis is not required for structures where the value of the parameter S_{M1} determined by Eq. (11.4-2) is increased by 50% for all applications of S_{M1} in the Standard. The resulting value of the parameter S_{D1} determined by Eq. (11.4-4) shall be used for all applications of S_{D1} in this Standard.” Assuming this exception is met for this project, a ground motion hazard analysis is not required and mapped seismic values can be used. Should this exception not be met, a ground motion hazard analysis is required to determine the Design response spectra for the proposed structures at this site. Both mapped and site-specific seismic design parameters are provided in this report, as presented in Section 6.2. Details of a ground motion hazard analysis are explained below.

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 the computer program OpenSHA (Field et al., 2013), which implements Method 1 as described in Section 21.2.1.1. Fault Models 3.1 and 3.2 from the Third Uniform California Earthquake Rupture Forecast (UCERF3) were used as the earthquake rupture forecast models for the PSHA. In addition to known fault sources, background seismicity was also included in the PSHA. The ground motion Prediction Equations (GMPEs) selected for use in this analysis are those developed for the Pacific Earthquake Engineering Research Center (PEER) Next Generation Attenuation (NGA) West 2 project. Four GMPEs - Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and Youngs (2014) were used to perform the analysis.

In accordance with Section 21.2.2 (Supplement 1), the deterministic spectral response acceleration at each period was calculated as the 84th percentile, 5% damped response acceleration, using NGA-West2 GMPE Worksheet. For this, 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 coefficients (F_a and F_v , respectively) were determined per Section 21.3 in conjunction 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 OSHPD seismic hazard tool. According to Section 21.5.3 of ASCE 7, either mapped or site-specific PGA_M may be used for soil related analysis. For this investigation, a mapped PGA_M of 0.85 was incorporated into the analysis.

4.2 STATIC SETTLEMENT

Analyses were performed for the potential settlement of the underlying soils encountered during our investigation. The site is underlain predominantly by coarse-grained materials. As such, analyses of settlement were based on the elastic method using estimated moduli correlated from N_{60} blow counts. Two analyses were performed to evaluate settlement of the structures. The first model was based on a conventional shallow strip footing 1.5 feet wide and a wall load of 3 k/ft. The second model was based on a conventional square footing with 3 feet width and a column load of 30 kips. Both models yielded a total settlement of less than 0.5 inches.

5.0 CONCLUSIONS

5.1 FEASIBILITY OF PROPOSED DEVELOPMENT

From a geotechnical point of view, the proposed site development is considered feasible, provided the recommendations presented in this report are incorporated into the design and construction of the project. Furthermore, it is also our opinion that the proposed development will not adversely impact the stability of adjoining properties. 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 subject sites nor do the sites lie within the boundaries of an “Earthquake Fault Zone” as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act. The closest known active fault is the San Joaquin Hills fault, located approximately 4.09 miles away. Therefore, the potential for ground rupture due to an earthquake beneath the sites is considered relatively low.

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 relatively close proximity to several seismically active faults; therefore, during the life of the proposed improvements, 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. Design and construction in accordance with the current California Building Code (CBC) requirements is anticipated to address the issues related to potential ground shaking.

5.2.3 Landsliding

Geologic hazards associated with landsliding are not anticipated at the site since the site is relatively level. Additionally, the site is not located in a state designated landslide zone.

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 site is **not** located within a State-designated potentially liquefiable zone, as identified by the California Geological Survey (CGS). Additionally, groundwater was not encountered to the maximum depth explored of 41.5 feet and expected to be present below a depth of 50 feet. As a result, the potential for liquefaction to occur beneath the site is considered very low.

5.3 STATIC SETTLEMENT

Analyses were performed to evaluate the potential for static settlement of the underlying alluvium. Provided site grading is performed in accordance with the recommendations provided herein and based on the anticipated relatively light foundation loads, total and differential static settlement is not anticipated to exceed 1 inch and ½-inch over 30 feet, respectively, for the proposed residential structure. The estimated magnitudes of static settlements are considered within tolerable limits for the proposed structure.

5.4 EXCAVATION AND MATERIAL CHARACTERISTICS

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 of deleterious debris and oversized rocks (greater than 4 inches in greatest dimension). If encountered, portions of concrete debris and asphalt can likely be reduced in size (4" minus) and incorporated within fill soils during earthwork operations.

Temporary construction slopes will be required to complete removal of unsuitable soils and for construction of underground utilities. Site materials are typically coarse-grained and will be prone to caving. Such excavations will require laybacks where they are surcharged or where they exceed 3 feet in height. Specific recommendations for temporary slopes are provided in Section 6.1.7.

Most of the near-surface soils are below optimum moisture content. As such, the addition of water to these materials will be required during placement and compaction as engineered fills.

5.5 SHRINKAGE AND SUBSIDENCE

Volumetric changes in earth quantities will occur when excavated onsite soil materials are replaced as properly compacted fill. Based on our experience with similar types of earth materials, we estimate that the existing soils will shrink approximately 8 to 13 percent when removed and replaced as compacted fill. Subsidence due to reprocessing of removal bottoms is anticipated to be on the order of 0.1 feet.

The estimates of shrinkage and subsidence 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 to 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 are generally anticipated to possess a **Very Low** expansion potential. As such, special requirements as set forth in CBC Chapter 18 regarding expansive soils are not required. Additional testing for soil expansion will be required prior to construction of foundations and other concrete work to confirm these conditions.

5.7 PERCOLATION CHARACTERISTICS

Site materials are predominantly coarse-grained and provide good characteristics for infiltration of storm water. The site can utilize either a shallow chamber system or a dry well system. A chamber set at a depth of 10 feet is expected to provide a design infiltration rate of about 2.5 in/hr. The estimated design capture volume (DCV) is about 2,600 cf. Therefore, a typical system could consist of about 65 lineal feet of StormTech MC-4500 chamber set at a depth of 10 feet. As an alternative, the system could consist of a single dry well with a supplemental storage system having a capacity of about 500 cf. A more detailed design can be provided once specific hydrology information is provided by the civil engineer. Detailed results of the percolation testing can be provided in a separate report.

6.0 RECOMMENDATIONS

6.1 EARTHWORK

6.1.1 General Earthwork and Grading Specifications

All earthwork and grading should be performed in accordance with applicable requirements of Cal/OSHA, applicable specifications of the Grading Codes of the City of Tustin, 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 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 for 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

Areas to be graded should be cleared of vegetation, existing asphalt, underground improvements to be abandoned and deleterious materials. Existing underground utility lines within the project area that will be protected in place and that fall within a 1 to 1 (H:V) plane projected down from the edges of footings may be subject to surcharge loads. Under such conditions, this office should be made aware of these conditions for evaluation of potential surcharging. Supplemental recommendations may be required to protect such improvements in place.

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.

Temporary construction equipment (office trailers, power poles, etc.) should be positioned to allow adequate room for clearing and recommended ground preparation to be performed for proposed structures, pavements, and hardscapes.

6.1.4 Ground Preparation

In general, the existing artificial fill is considered unsuitable for support of the proposed development. These unsuitable soils are estimated to be 3-4 feet thick. These materials should be removed and replaced as engineered compacted fills below proposed residential structures, retaining walls, and pavement.

Removals should extend a minimum distance of 5 feet beyond the residential foundations. The removals should extend at least 2 feet beyond the edges of pavement and wall footings where feasible. 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 at least 1 foot below the bottom of footings for residential structures supported by conventional spread footings.

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.

Following removals and overexcavation, the exposed grade should first be scarified to a depth of 6 inches, brought to at least the optimum moisture content, and then compacted to at least 90 percent of the laboratory standard (ASTM D 1557).

6.1.5 Fill Placement

Materials excavated from the site may be reused as fill provided they are free of deleterious materials and particles greater than 4 inches in maximum dimension (oversized materials). Asphaltic and concrete debris generated during site demolition or encountered within the existing fill can be incorporated within new fill soils during earthwork operations provided they are reduced to no more than 4 inches in maximum dimension. Such materials should be mixed thoroughly with fill soils to prevent nesting. All fill should be placed in lifts no greater than 8 inches in loose thickness, moisture conditioned to at least the optimum moisture content, then compacted in place to at least 90 percent of the laboratory standard. Each lift should be treated in a similar manner. Subsequent lifts should not be placed until the project geotechnical consultant has approved the preceding lift.

6.1.6 Import Materials

If import materials are required to achieve the proposed finish grades, the proposed import soils should have an Expansion Index (EI, ASTM D 4829) less than 21 and possess negligible soluble sulfate concentrations. Import sources should be indicated to the geotechnical consultant prior to hauling the materials to the site so that appropriate testing and evaluation of the fill materials can be performed in advance.

6.1.7 Temporary Excavations

Temporary construction slopes and trench excavations may be cut vertically up to a height of 3 feet provided that no surcharging of the excavations is present. Temporary excavations greater than 3 feet in height but no more than 10 feet should be laid back to a 1.5:1 (H:V) or flatter or shored to mitigate the potential for instability. Specific recommendations should be provided by this office if excavations will be surcharged.

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.

6.2 SEISMIC DESIGN PARAMETERS

6.2.1 Mapped Seismic Design Parameters

For the design of the project in accordance with Chapter 16 of the 2022 CBC, the mapped seismic parameters may be taken as presented in the tables below.

TABLE 6.1
2022 CBC Mapped Seismic Design Parameters

Parameter	Value
Site Class	D
Mapped MCE_R Spectral Response Acceleration, short periods, S_S	1.289
Mapped MCE_R Spectral Response Acceleration, at 1-second period, S_1	0.46
Site Coefficient, F_a	1
Site Coefficient, F_v *	1.84
Adjusted MCE_R Spectral Response Acceleration, short periods, S_{MS}	1.289
Adjusted MCE_R Spectral Response Acceleration, at 1-second period, S_{M1} *	1.270
Design Spectral Response Acceleration, short periods, S_{DS}	0.859
Design Spectral Response Acceleration, at 1-second period, S_{D1} *	0.846
Long-Period Transition Period, T_L (sec.)	8
Seismic Design Category for Risk Categories I-IV	II
MCE_R = Risk-Targeted Maximum Considered Earthquake	
*According to Section 11.4.8 in ASCE 7-16 and supplement 3, “a ground motion hazard analysis shall be performed in accordance with Section 21.2 for the following structures on Site Class D site with S_1 greater than or equal to 0.2.” However, “A ground motion hazard analysis is not required for structures where the value of the parameter S_{M1} determined by Eq. (11.4-2) is increased by 50% for all applications of S_{M1} in the Standard. The resulting value of the parameter S_{D1} determined by Eq. (11.4-4) shall be used for all applications of S_{D1} in this Standard.” Should this exception not be met, the site-specific seismic design parameters provided in the next section should be used.	

6.2.2 Site-Specific Seismic Design Parameters

In addition to the Code Spectra parameters presented in Table 6.1, we have performed a site-specific ground motion hazard analysis in accordance with Chapter 21 of ASCE 7-16 to obtain site-specific seismic design acceleration parameters, the risk-targeted maximum considered earthquake response spectrum, and the design earthquake response spectrum. The site-specific seismic design parameters are presented below.

TABLE 6.2
2022 CBC Site-Specific Seismic Design Parameters

Parameter	Value
Site Class	D
Site Coefficient, F_a	1.0
Site Coefficient, F_v	2.5
Adjusted MCE Spectral Response Acceleration, short periods, S_{MS}	1.031
Adjusted MCE Spectral Response Acceleration, at 1-second period, S_{M1}	0.920
Design Spectral Response Acceleration, short periods, S_{DS}	0.687
Design Spectral Response Acceleration, at 1-second period, S_{D1}	0.613
MCE = Maximum Considered Earthquake	

6.3 CONVENTIONAL FOUNDATION DESIGN

6.3.1 General

The following design parameters are provided to assist the project structural engineer to design foundations for structures at the site. These design parameters are based on typical site materials encountered during subsurface exploration and are provided for preliminary design and estimating purposes. The project geotechnical consultant should provide final design parameters following observation and testing of site materials during grading. Depending on actual materials encountered during site grading, the design parameters presented herein may require modification.

6.3.2 Soil Expansion

The recommendations presented herein are based on soils with a **Very Low** expansion potential. 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.

6.3.3 Static and Seismic Settlement

Based on anticipated foundation loads and provided that the recommendations for ground preparation in this report are followed, total and differential static settlement are anticipated to be less than 1 inch and ½ inch over 30 feet, respectively. These values are considered within tolerable limits of proposed structures and site improvements. Design of the structures should consider these maximum anticipated settlements.

6.3.4 Allowable Bearing Value

Foundations may utilize a bearing value of 2,000 pounds per square foot (psf) for continuous and pad footings a minimum width of 12 inches and founded at a minimum depth of 12 inches below the lowest adjacent grade. This value may be increased by 250 psf and 1,000 psf for each additional foot in width and depth, respectively, up to a maximum value of 3,000 psf. Recommended allowable bearing values include both dead and live loads and may be increased by one-third for wind and seismic forces.

6.3.5 Lateral Resistance

A passive earth pressure of 200 pounds per square foot per foot of depth (psf/ft) up to a maximum value of 1,000 pounds per square foot (psf) may be used to determine lateral bearing for footings. This value may be increased by one-third when designing for wind and seismic forces. A coefficient of friction of 0.37 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.

The above values are based on footings placed directly against compacted fill or competent native soils. 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.

6.3.6 Footing and Slab on Grade

Exterior and interior building footings should be founded at a minimum depth of 12 inches and 12 inches, respectively, below the lowest adjacent grade. 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.

Interior isolated pad footings should be a minimum of 24 inches square and founded at minimum depths of 12 inches below the lowest adjacent final grade. Exterior isolated pad footings intended for support of patio covers or similar construction should be a minimum of 24 inches square and founded at a minimum depth of 12 inches below the lowest adjacent final grade.

Interior concrete slabs constructed on grade should be a nominal 4 inches thick and should be reinforced with No. 3 bars spaced 30 inches on center, each way. Care should be taken to ensure the placement of reinforcement at mid-slab height. The structural engineer may recommend a greater slab thickness and reinforcement based on proposed use and loading conditions and such recommendations should govern if greater than the recommendations presented herein.

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 with at least 2 inches of sand having a SE no less than 30. One inch of this sand may be placed over the membrane to aid in the curing of the concrete. 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.

Special consideration should be given to slabs in areas to receive ceramic tile or other rigid, crack-sensitive floor coverings. Design and construction of such areas should mitigate hairline cracking as recommended by the structural engineer.

Garage floor slabs should have a minimum thickness of 4 inches and should be reinforced in a similar manner as living floor slabs. Garage floor slabs should also be poured separately from adjacent wall footings with a positive separation maintained with 3/8-inch minimum felt expansion joint materials and provided with saw cuts or cold joints at a maximum spacing of 15 feet in each direction.

Consideration should be given to providing a vapor retarder below the garage slab to mitigate the potential for effervescence on the slab surface.

Block-outs should be provided around interior columns to permit relative movement and mitigate distress to the floor slabs due to differential settlement that will occur between column footings and adjacent floor subgrade soils as loads are applied.

Prior to placing concrete, subgrade soils below slab-on-grade areas should be thoroughly moistened to provide a moisture content of at least 100 percent of the optimum moisture content to a depth of 12 inches.

6.3.7 Foundation Observations

Foundation excavation 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.

6.4 RETAINING AND SCREENING WALLS

6.4.1 General

The following preliminary design and construction recommendations are provided for general retaining and screen walls supported by engineered compacted fill or competent native soils. Final wall designs specific to the site development should be provided 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.

6.4.2 Allowable Bearing Value and Lateral Resistance

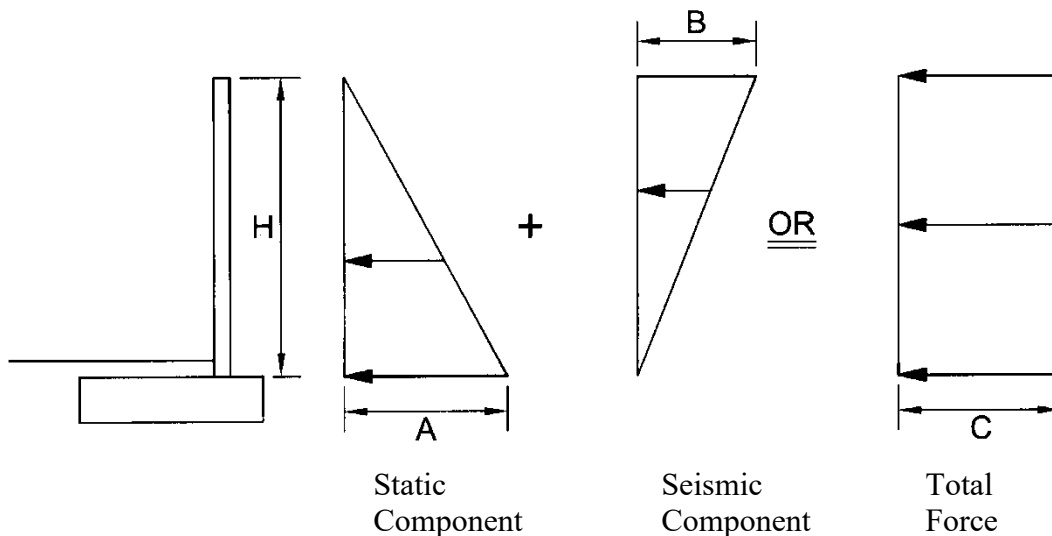
Design of retaining and screen walls may utilize the bearing and lateral resistance values provided in Section 6.3.4 and 6.3.5. The passive earth pressure for walls along property lines, where lateral removals are likely restricted, should be reduced by 25%.

6.4.3 Active 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 0.386g for 10% probability of exceedance in 50 years. As indicated in Section 1807.2.2 of the 2022 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.

TABLE 6.3

SEISMIC EARTH PRESSURES
Pressure Diagram



Pressure Values
Walls Up To 10 Feet High

Value	Backfill Condition	
	Level	2H:1V Slope
A	34H	55H
B	12H	12H
C	23H	34H
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.		

6.4.4 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 $\frac{3}{4}$ - to $1\frac{1}{2}$ -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.

6.4.5 Foundation 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 EXTERIOR FLATWORK

Concrete sidewalks, patios, and similar flatwork should be a nominal 4 inches thick and provided with saw cuts or expansion joints at spacing no greater than 15 feet in each direction. Special jointing details should be provided in areas of block-outs, notches, or other irregularities to avoid cracking at points of high stress.

Drainage from flatwork areas should be directed to local area drains and/or other appropriate collection devices designed to carry runoff water to the street or other approved drainage structures. The concrete flatwork should also be sloped at a minimum gradient of 0.5 percent away from building foundations and retaining walls.

Subgrade soils below flatwork should be thoroughly moistened to at least 100 percent of the optimum moisture content to a depth of 12 inches. Flooding or ponding of the subgrade is not recommended. Moistening should be accomplished by lightly spraying the area 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 ensure that the required compaction and pre-moistening recommendations have been met.

6.6 CONCRETE MIX DESIGN

Laboratory testing of near-surface soils for soluble sulfate content indicates soluble sulfate concentration of 0.000%. We recommend following the procedures provided in ACI 318, Section 19.3, Table 19.3.1.1 for **S0** 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.

6.7 PRELIMINARY PAVEMENT DESIGN

6.7.1 Preliminary Pavement Structural Sections

Based on the soil conditions present at the site and an estimated traffic index, preliminary pavement sections are provided in the table below. An assumed “R-value” of 35 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

Location	Traffic Index	AC (inches)	Concrete Pavers (mm)	PCC (inches)	AB (inches)
Entry Way and Drives	5.0	3.0	--	--	5.0
		4.0			4.0
		--	--	6.0	--
		--	80.0	--	6.0
Parking Stalls	--	3.0	--	--	4.0

6.7.2 Subgrade Preparation

Prior to placement of paving elements, subgrade soils should be scarified 6 inches, moisture-conditioned to above the optimum moisture content then compacted to at least 90 percent of the maximum dry density determined in accordance with ASTM D1557. Areas observed to pump or yield under vehicle traffic should be removed and replaced with firm and unyielding engineered compacted soil or aggregate base materials.

6.7.3 Aggregate Base

Aggregate base materials should be Crushed Aggregate Base or Crushed Miscellaneous Base conforming to Section 200-2 of the Standard Specification for Public Works Construction (Greenbook) or Class 2 Aggregate Base conforming to the Caltrans' Standard Specifications. The materials should be moisture conditioned to slightly over the optimum moisture content then compacted to at least 95 percent of ASTM D 1557.

6.7.4 Asphaltic Concrete

Paving asphalt should be PG 64-10 conforming to the requirements of Section 203-1 of the Greenbook. Asphalt concrete materials should conform to Section 203-6 and construction should conform to Section 302 of the Greenbook.

6.7.5 Concrete Paver

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
$\frac{3}{8}$ "	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

6.7.6 Portland Cement Concrete

Portland cement concrete used to construct concrete paving should conform to Section 201 of the Greenbook and should have a minimum compressive strength of 3,250 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 $\frac{1}{4}$ 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 without keyways or dowels 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.

6.8 POST GRADING CONSIDERATIONS

6.8.1 Site Drainage and Irrigation

The ground immediately adjacent to foundations should be provided with positive drainage away from the structures in accordance with 2022 CBC, Section 1804.4. However, the slope of the ground surface may be reduced to 2% based on climatic and soil conditions. No rain or excess water should be 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 6.1.7 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. Materials placed within the pipe zone (6 inches below and 12 inches above the pipe) should consist of particles no greater than $\frac{3}{4}$ inches and have a SE of at least 30. The materials within the pipe zone should be moisture-conditioned and compacted by hand-operated compaction equipment. Above the pipe zone (>1 foot above pipe), the backfill may consist of general fill materials. Trench backfill should be moisture-conditioned to slightly over the 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. For trenches with sloped walls, backfill material should be placed in lifts no greater than 8 inches in loose thickness, and then compacted by rolling with a sheepsfoot roller or similar equipment. The project geotechnical consultant should perform density testing along with probing to verify that adequate compaction has been achieved.

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 PLAN REVIEW AND CONSTRUCTION SERVICES

We recommend *Albus & Associates, Inc.* be engaged to review any future development plans, including civil plans (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 encountered on the project site and utilized in our laboratory testing for this investigation are believed representative of the total project area, and the conclusions and recommendations contained in this report are presented on that basis. However, soil and bedrock 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 during the grading and construction phases of the project are essential to confirming the basis of this report.

This report has been prepared consistently 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** and their project consultants in the planning and design 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.

This report is subject to review by the controlling governmental agency.

Respectfully submitted,

ALBUS & ASSOCIATES, INC.



David E. Albus
Principal Engineer
GE 2455



Daniel Albus
Senior Project Engineer

8.0 REFERENCES

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0 20 40 80
APPROX SCALE : 1" = 40'

EXPLANATION

(Locations Approximate)

-  - Exploratory Boring
-  - Percolation Testing

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Base Provided By Land Development Consultants



GEOTECHNICAL MAP

Job No.: 3374.00 | Date: 11/20/2025 | Plate: 1

APPENDIX A
EXPLORATION LOGS

Field Identification Sheet



Light gray



Gray



Dark gray



Gray Brown



Light brown



Brown



Dark Brown



Olive brown



Olive



Yellow



Yellowish brown



Yellowish red



Red



Reddish Brown



Tan

Description Order:

Description, Color, Moisture, Density, Grain Size, Additional Description

Description	%	Example
	0-5	Sand
trace	5-15	Sand trace Silt
with	15-30	Sand with Silt
	30+	Silty Sand

More Examples

Sand with Silt trace Clay

Sand trace Silt and Clay

Sand with Silt and Clay

Gravelly Sand with Silt trace Clay

Silty Clay with Sand trace Gravel

Moisture

Dry	absence of water
Damp	below optimum
Moist	near optimum
Very Moist	above optimum
Wet	free water visible

Density (Navfac)

Coarse grained soils	SPT	CA
Very Loose	0-3	0-5
Loose	3-8	5-13
Medium Dense	8-14	13-22
Dense	14-25	22-40
Very Dense	25>	40>

Fine grained soils

Very Soft	2<	0-3
Soft	2-4	3-6
Medium Stiff	4-8	6-13
Stiff	8-15	13-24
Very Stiff	15-30	24-48
Hard	30>	48>

Grain Size

Description	Sieve Size	Approx. Size
Boulders	>12"	Larger than basketball
Cobbles	3-12"	Fist to basketball
Gravel	coarse 3/4-3"	Thumb to Fist
	fine #4-3/4"	Pea to Thumb
Sand	coarse #10-4	Rock Salt to Pea
	medium #40-10	Sugar to Rock Salt
	fine #200-40	Flour to Sugar
Fines	Pass #200	Smaller than Flour

Additional Description (ie. roots, pinhole pores, debris, etc.)

Trace 5%

Moderate 15%

Abundant 30%



EXPLORATION LOG												
JOB NO.		CLIENT/PROJECT					DAY		DATE			
LOCATION					LATITUDE		LONGITUDE		ELEVATION			
LOGGED BY			DRILLER		DRILL METHOD			DRIVING WEIGHT				
DEPTH	LITHO	DESCRIPTION				H2O	COR	BAG	BLOW COUNT	MC (%)	DD (pcf)	LAB
		<u>EXPLANATION</u>										
1												
2		Solid lines separate geologic units and/or material types										
3		Dashed lines indicate unknown depth of geologic unit change or material type change.										
4												
5		Solid rectangle in Core column represents California										
6		Split Spoon sampler (2.5-in. ID, 3-in. OD).										
7		Double triangle in core column represents SPT sampler.										
8												
9		Vertical lines in core column represents Shelby sampler.										
10												
11		Solid rectangle in Bulk column represents large bag										
12		sample.										
13												
14		<u>Other Laboratory Tests:</u>										
15		Max = Maximum Dry Density/Optimum Moisture Content										
16		EI = Expansion Index										
17		SO ₄ = Soluble Sulfate Content										
18		DSR = Direct Shear, Remolded										
19		DS = Direct Shear, Undisturbed										
20		SA = Sieve Analysis (1" through #200 sieve)										
21		Hydro = Particle Size Analysis (SA with Hydrometer)										
22		200 = Percent Passing #200 Sieve										
23		Consol = Consolidation										
24		SE = Sand Equivalent										
25		Rval = R-Value										
26		ATT = Atterberg Limits										

EXPLORATION LOG B-1

JOB NO. 3374.00		CLIENT/PROJECT The Olson Company				DAY Thursday		DATE 2025-11-06	
LOCATION 17332 Irvine Boulevard, Tustin				LATITUDE 33.74782		LONGITUDE -117.82939		ELEVATION 146.5	
LOGGED BY ddalbus			DRILLER 2R Drilling		DRILL METHOD Hollow-Stem Auger		DRIVING WEIGHT 140 lbs / 30 in		
DEPTH	LITHO	DESCRIPTION	H2O	COR	BAG	BLOW COUNT	MC (%)	DD (pcf)	LAB
1		asphalt							
2		<u>Artificial Fills (Af)</u> Sand (SP): light gray, dry to damp, loose, fine to medium-grained				10	2.8	107.7	
3									
4		<u>Younger Alluvium (Qya)</u> Sand (SP): light reddish brown, moist, loose, fine to coarse-grained				12	2.2	107.6	
5									
6		@ 6 ft, medium dense				17	3.2	98.7	
7									
8									
9									
10		Gravelly Sand (SP): light reddish brown, moist, medium dense , fine to coarse-grained, river rock				36	1.7	119.1	sa
11									
12									
13									
14									
15		@ 15 ft, loose				5			
16		<u>Older Alluvium (Qoa)</u> Silty Sand (SM): reddish brown, moist, medium dense, fine-grained							
17									
18									
19									
20						8			sa
21									hydro
22									
23									
24									



EXPLORATION LOG B-1

JOB NO. 3374.00		CLIENT/PROJECT The Olson Company				DAY Thursday		DATE 2025-11-06												
LOCATION 17332 Irvine Boulevard, Tustin				LATITUDE 33.74782		LONGITUDE -117.82939		ELEVATION 146.5												
LOGGED BY ddalbus			DRILLER 2R Drilling		DRILL METHOD Hollow-Stem Auger		DRIVING WEIGHT 140 lbs / 30 in													
DEPTH	LITHO	DESCRIPTION				H2O	COR	BAG	BLOW COUNT	MC (%)	DD (pcf)	LAB								
26		Older Alluvium (Qoa) Silty Sand (SM): reddish brown, moist, medium dense, fine-grained							10											
27																				
28																				
29																				
30																				
31						Sandy Silt trace Clay (ML): reddish brown, moist, stiff, fine grained										8			sa hydro	
32																				
33																				
34																				
35																				
36																				
37																				
38																				
39																				
40	@ 35 ft, very stiff											9								
41																				
42																				
43																				
44																				
45																				
46																				
47																				
48																				
49																				
40		No Recovery, rock fragments							80/ 11"		no recover									
41																				
42		Total Depth 41.5 feet No Groundwater																		
43																				
44																				
45																				
46																				
47																				
48																				
49																				

EXPLORATION LOG B-2

JOB NO. 3374.00		CLIENT/PROJECT The Olson Company				DAY Thursday		DATE 2025-11-06				
LOCATION 17332 Irvine Boulevard, Tustin				LATITUDE 33.74762		LONGITUDE -117.82972		ELEVATION 146.4				
LOGGED BY ddalbus			DRILLER 2R Drilling		DRILL METHOD Hollow-Stem Auger		DRIVING WEIGHT 140 lbs / 30 in					
DEPTH	LITHO	DESCRIPTION				H2O	COR	BAG	BLOW COUNT	MC (%)	DD (pcf)	LAB
		asphalt										
1		<u>Artificial Fills (Af)</u>										max
2		Sand (SP): light gray, dry, medium dense , fine to medium-grained							17	2.4	109.5	ei so4
3												200
4		<u>Younger Alluvium (Qya)</u>							14	2.1	107.3	ph
5		Sand trace Gravel (SP): light reddish brown, moist, loose, fine to medium-grained, trace gravel										resist
6		@ 6 ft, medium dense, coarse-grained							26	3.2	102.3	ch
7												
8												
9												
10		@ 10 ft, river rock							36	4.1	116.4	
11												
12		Total Depth 11.5 feet										
13		No Groundwater										
14												
15												
16												
17												
18												
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23												
24												

EXPLORATION LOG B-3

JOB NO. 3374.00		CLIENT/PROJECT The Olson Company				DAY Thursday		DATE 2025-11-06				
LOCATION 17332 Irvine Boulevard, Tustin				LATITUDE 33.74771		LONGITUDE -117.82995		ELEVATION 147				
LOGGED BY ddalbus		DRILLER 2R Drilling		DRILL METHOD Hollow-Stem Auger			DRIVING WEIGHT 140 lbs / 30 in					
DEPTH	LITHO	DESCRIPTION				H2O	COR	BAG	BLOW COUNT	MC (%)	DD (pcf)	LAB
		asphalt										
1		<u>Artificial Fills (Af)</u> Silty Sand (SM): reddish brown, moist, medium dense, fine-grained										
2												
3												
4		<u>Younger Alluvium (Qya)</u> Sand trace Gravel (SP): light reddish brown, moist, loose, fine to coarse-grained							23	6.9	116	
5												
6												
7		@ 6 ft, coarse-grained							13	4.4	102.6	
8												
9												
10		@ 10 ft, river rock							13	3.4	98.6	
11												
12												
13		Total Depth 11.5 feet No Groundwater										
14												
15												
16												
17												
18												
19												
20												
21												
22												
23												
24												

APPENDIX B

LABORATORY TEST RESULTS

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 and Density

Moisture content and unit dry density of in-place soil materials were determined in representative strata. Test data are summarized in the Boring Logs, Appendix A.

Maximum Dry Density and Optimum Moisture Content

Maximum dry density and optimum moisture content were performed on representative samples of the site materials obtained from our field explorations. The test was performed in accordance with ASTM D 1557. Pertinent test values are given in Table B.

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 in Table B.

Soluble Sulfate Content

Chemical analysis is being performed on selected samples to determine soluble sulfate content. The test was performed in accordance with California Test Method No. 417.

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 through B-3.

Hydrometer

Hydrometer analyses were performed on representative samples of site materials in accordance with ASTM D 7928. The results are presented graphically on the attached Plates B-2 and B-3.

Direct Shear

Direct shear testing was performed for a selected soil sample remolded to 90 percent of the maximum dry density. This test was performed in general accordance with ASTM D3080. Three specimens were prepared for the test. The test specimens were artificially saturated, and then sheared under varied normal loads at a constant rate. The results are graphically presented on Plate B-4.

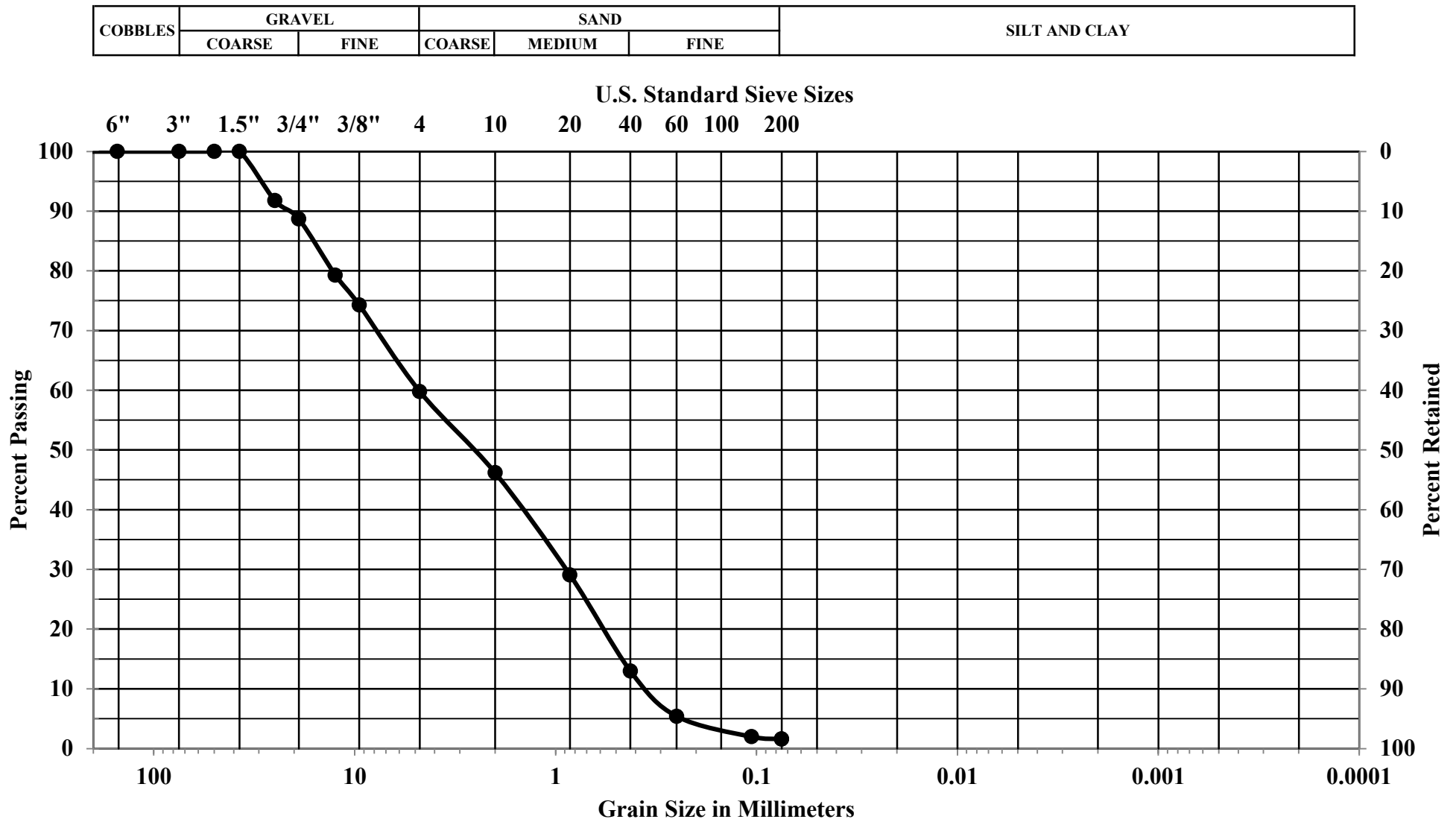
Corrosion Analysis

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

TABLE B
SUMMARY OF LABORATORY TEST RESULTS

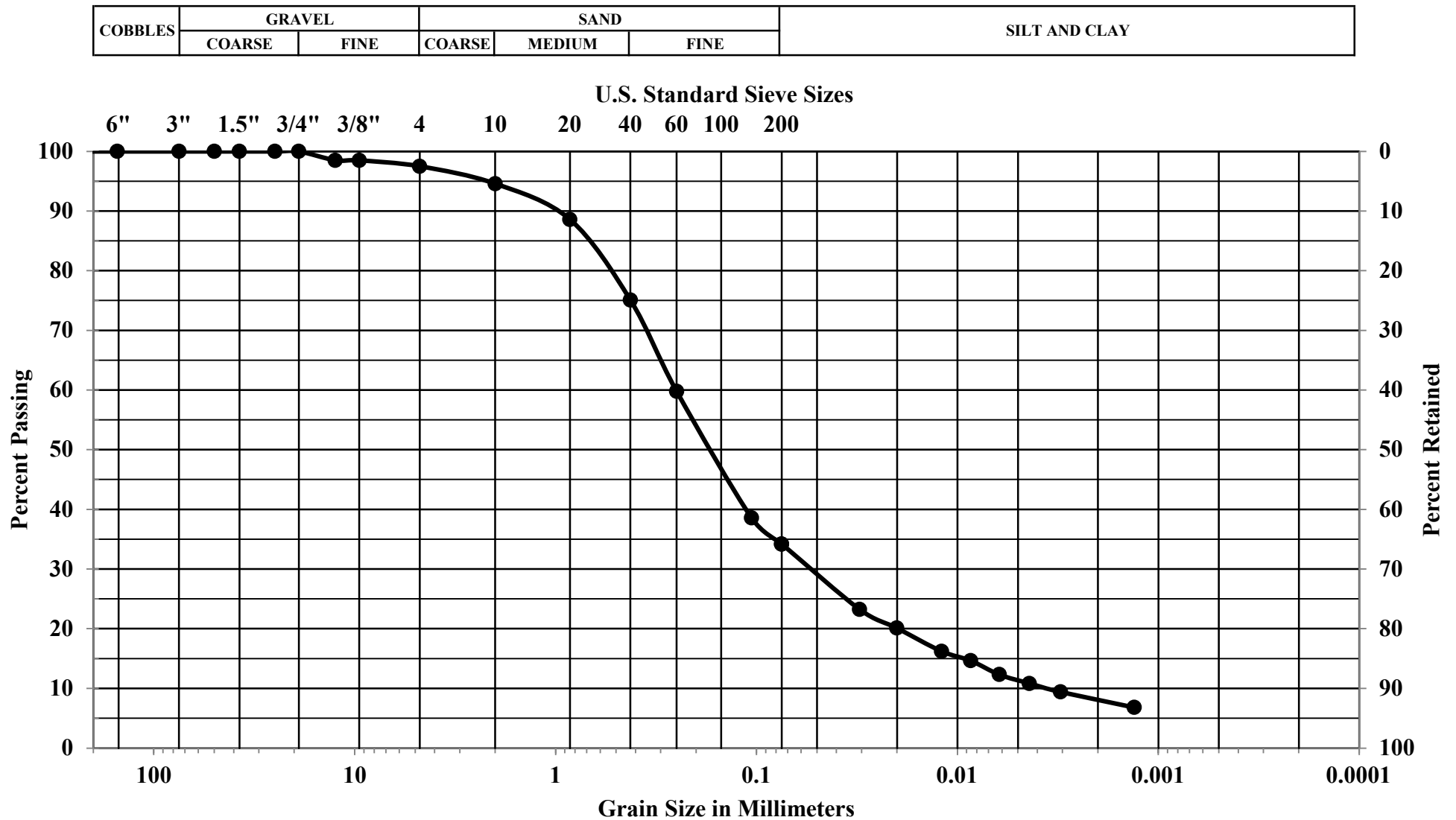
Boring No.	Sample Depth (ft)	Soil Description	Test Results	
B-2	0-5	Sand (SP)	Maximum Dry Density (pcf):	129.3
			Optimum Moisture Content (%):	9
			Expansion Index:	0
			Expansion Potential:	Very Low
			Soluble Sulfate Content (%):	0.000
			Sulfate Exposure:	S0
			pH:	7.07
			Minimum Resistivity (ohm-cm):	6,000
			Chloride (ppm):	35

GRAIN SIZE DISTRIBUTION



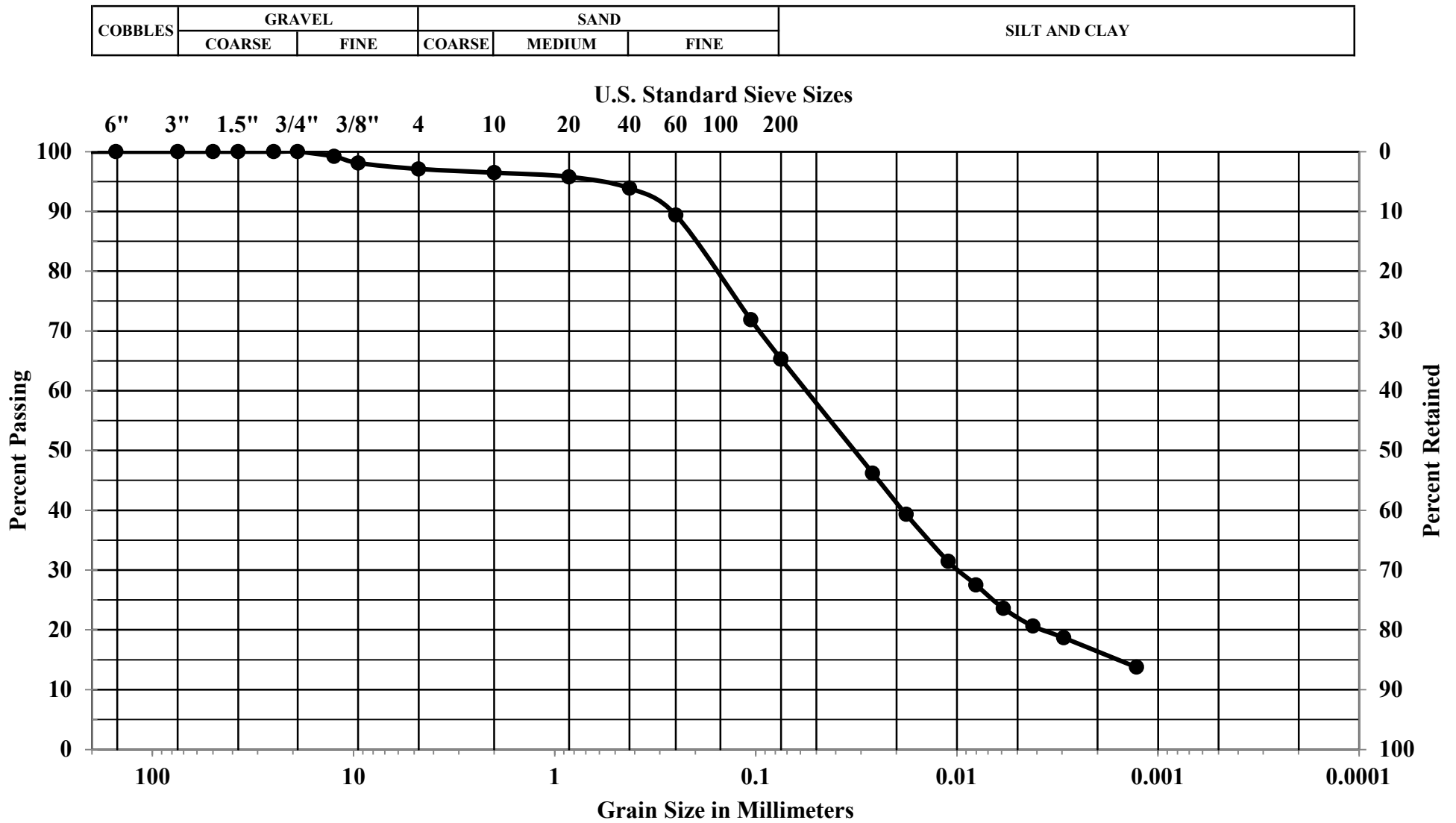
Job Number	Location	Depth	Description
3374.00	B-1	10	Gravelly Sand (SW)

GRAIN SIZE DISTRIBUTION

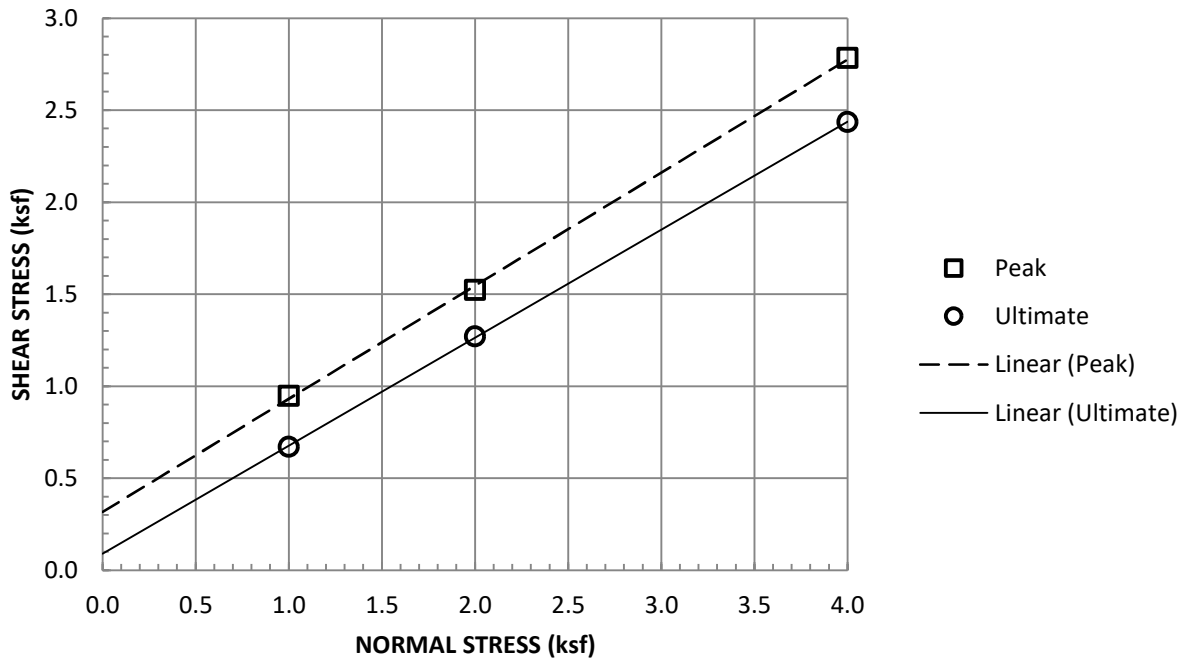
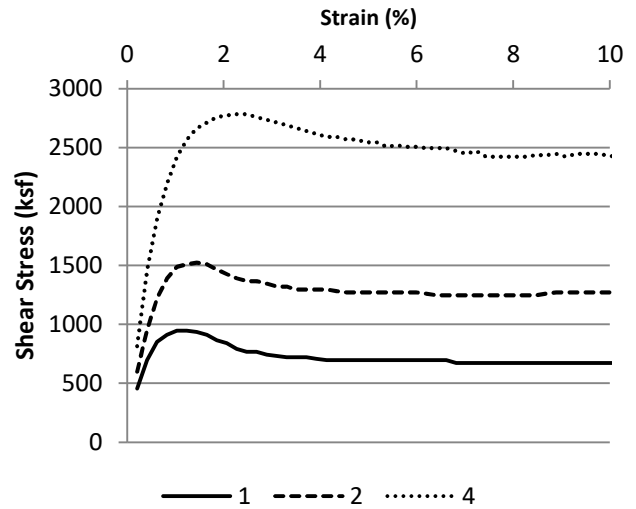
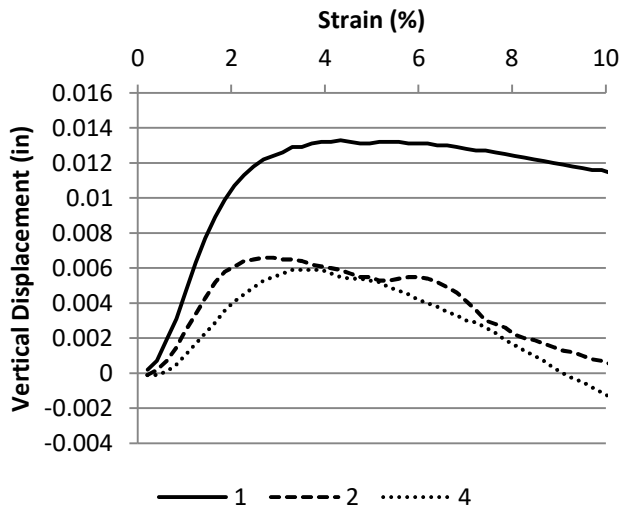


Job Number	Location	Depth	Description
3374.00	B-1	20	Silty Sand (SM)

GRAIN SIZE DISTRIBUTION



DIRECT SHEAR



Sample Type:	Remolded to 90% of 129.3 pcf, Saturated		
Normal Stress (ksf)	1	2	4
Peak Shear Stress (ksf)	0.948	1.524	2.784
Peak Displacement (in)	0.013	0.007	0.006
Ultimate Shear Stress (ksf)	0.672	1.272	2.436
Ultimate Displacement (in)	0.25	0.25	0.25
Initial Dry Density (pcf)	116.3	116.2	116.3
Initial Moisture Content (%)	9	9	9
Final Moisture Content (%)	13.7	13.9	13.6
Strain Rate (in/min)	0.01		

Job Number	Location	Depth	Description
3374.00	B-2	0-5	Sand (SP)