Appendix D. Preliminary Geotechnical Investigation

April 28, 2021
J.N.: 2967.00

Dennis Buccola and Gilad Ganish
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2881 East La Cresta Ave.
Anaheim, CA 92806

Subject: Preliminary Geotechnical Investigation, Proposed 5-story Residential Building Development, 777 West Orangethorpe Avenue, Placentia, California

Dear Mr. Ganish,

Pursuant to your request, ***Albus & Associates, Inc***. is pleased to present to you our preliminary geotechnical investigation report for the proposed residential building 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 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 should have any questions regarding the contents of this report, please do not hesitate to call.

Sincerely,

ALBUS & ASSOCIATES, INC.

David E. Albus
Principal Engineer

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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 project site is located at the address of 777 West Orangethorpe Avenue within the city of Placentia, California. The site is bordered by a Jack in the box to the southwest corner, West Orangethorpe Avenue to the south, West Placentia Avenue to the west, a 3-story commercial building with the maXum therapy and the interface rehab, inc. to the north, and the Coastal Spa and Patio Store building to the east. The location of the site and its relationship to the surrounding areas are shown in Figure 1, Site Location Map.

The site is irregularly shaped and encompasses approximately 2.7 acres of land. Currently, the site consists of a commercial building with parking lots and associated roads.

Topographically, the project site is relatively flat, with elevations ranging from 191 to 201 feet above mean sea level (based on Google Earth). Drainage is generally directed to the southwest onto Orangethorpe Avenue and Placentia Avenue. The existing building was used for Premier Automotive of Placentia and but is currently closed. The remainder of the site is covered in the pavement for associated parking lots and roads.

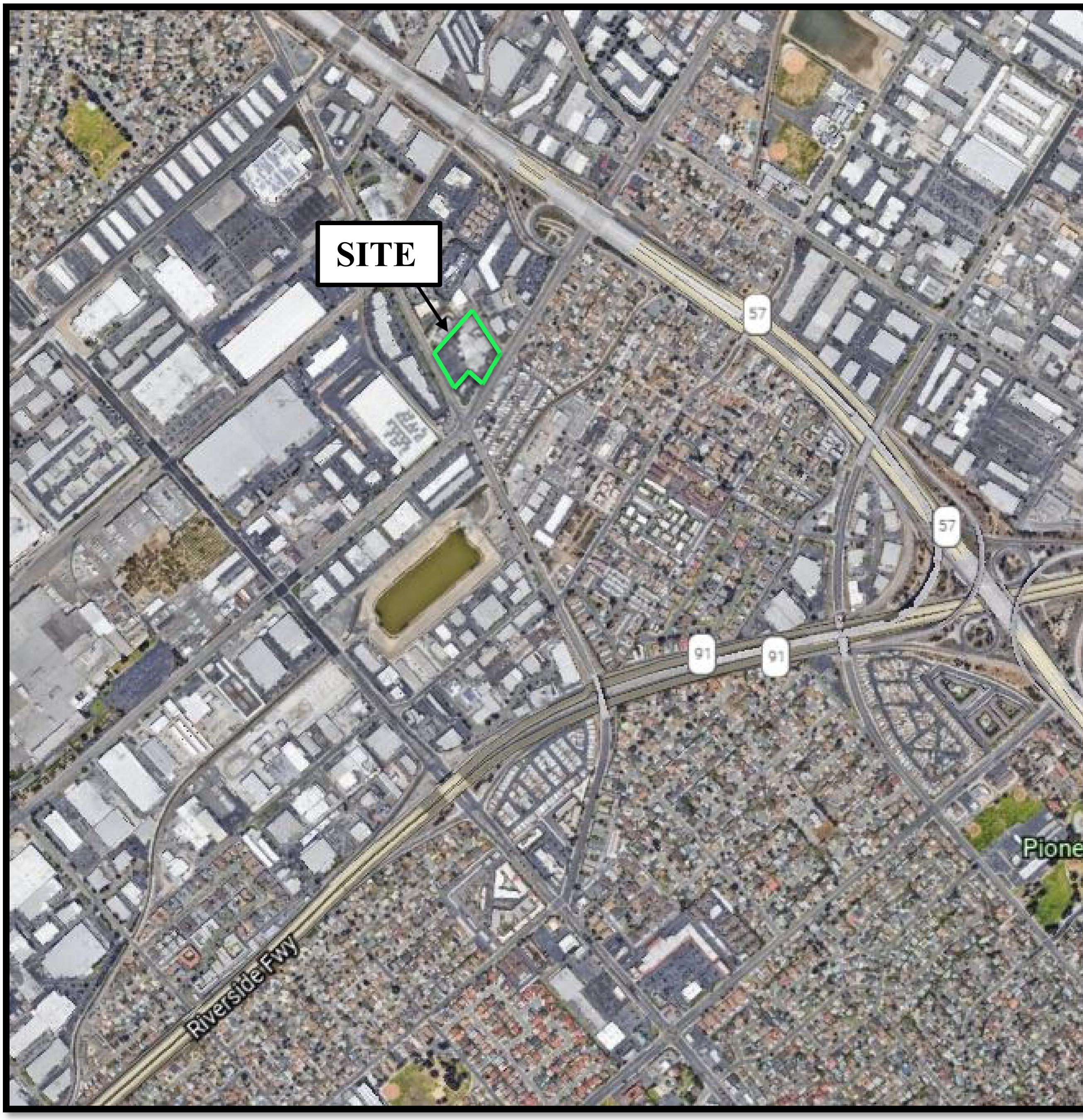

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FIGURE 1
SITE LOCATION MAP

777 West Orangethorpe Avenue
Placentia, California

NOT TO SCALE

1.3 PROPOSED DEVELOPMENT

Based on our understanding, the site will be redeveloped in the future for residential use. The development will likely consist of a 5-story mixed-use structure (residential over retail) wrapped around a 5-level parking structure. Associated parking lots, interior driveways, decorative hardscape, and underground utilities are also anticipated.

No grading or structural plans were available in preparing this report. However, we anticipate that minor rough grading of the site will be required to achieve future surface configuration and we expect future foundation loads will be relatively moderate. All structures are anticipated to be at grade.

2.0 INVESTIGATION

2.1 RESEARCH

We have reviewed the referenced geologic publications and maps (see references). Data from these sources were utilized to develop some of the findings and conclusions presented herein.

We have also reviewed available historical aerial photographs. The aerial photos indicate that in 1949, an old road passed from northwest to southeast through the middle of the project site. The road appears to have many trees along the southwest edge of the road and the area northeast of the old road is a citrus grove. By 1952, Orangethorpe Avenue and Placentia Avenue were constructed and the project site appears to be entirely used for citrus groves. By 1970, the site and the surrounding areas were no longer used for citrus and the site is cleared of the trees. No site improvements are evident at this time. By 1973, the site appears to have been developed into its current configuration and has remained relatively unchanged since that time.

2.2 SUBSURFACE EXPLORATION

Subsurface exploration for this investigation was conducted on April 14, 2021 and consisted of the drilling of six (6) soil borings to depths ranging from approximately 26.5 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. logged 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. 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.

Two additional borings (P-1 and P-2) were drilled for percolation testing. Details and results of percolation tests are reported under a separate cover.

2.3 LABORATORY TESTING

Selected samples obtained from our subsurface exploration were tested in our soil laboratory. Tests consisted of in-situ moisture content and dry density, maximum dry density and optimum moisture content, expansion index, soluble sulfate content, direct shear, consolidation/collapse potential, grain-size distribution analysis, passing 200, and corrosivity testing (pH, chloride, and resistivity). A description of laboratory test criteria and test results are presented in Appendix B.

3.0 SUBSURFACE CONDITIONS

3.1 SOIL CONDITIONS

Soil materials encountered at the subject site consisted of young alluvial fan deposits overlain by approximately 3 to 4 feet of undocumented artificial fill. Across most of the area of the subject site, the artificial fill is covered by asphalt concrete pavement and 3 to 6 inches of aggregate base. The artificial fill is generally comprised of brown, clayey and silty sand. These materials are typically moist to damp and loose to medium dense. Asphalt and rock fragments were observed within the fill soils.

Young alluvial fan deposits were encountered below the artificial fill materials to the maximum depth of exploration (51.5 feet below the ground surface). The young alluvial fan deposits are predominantly comprised of gray, light brown, and olive brown silty sand and sand. These deposits are typically damp to moist and loose to very dense.

3.2 GROUNDWATER

Groundwater was not encountered during this subsurface exploration to a depth of 51.5 feet. The CDMG Special Report 037 suggests that historic high groundwater for the subject site deeper than 50 feet below ground surface.

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)
Puente Hills (Coyote Hills)	1.78	0.7	26	thrust	2.8	17
Elsinore;W+GI	5.01	n/a	81	strike slip	0	83
Elsinore;W+GI+T+J+CM	5.01	n/a	84	strike slip	0	241
Elsinore;W	5.01	2.5	75	strike slip	0	46
Elsinore;W+GI+T	5.01	n/a	84	strike slip	0	124
Elsinore;W+GI+T+J	5.01	n/a	84	strike slip	0	199
Puente Hills (Santa Fe Springs)	9.22	0.7	29	thrust	2.8	11

4.0 ANALYSES

4.1 SEISMICITY

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 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 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.

However, "A ground motion hazard analysis is not required for structures where: Structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient Cs is determined by Eq. (12.8-2) for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$." 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, 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 coefficients (Fa and Fv, 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. From our analyses, we obtain a PGA_M of 0.756g.

4.2 STATIC SETTLEMENT

Analyses were performed to evaluate static settlements of continuous and spread footings for expected loading locations for the proposed development. Our analyses were based on the correlations between deformation properties (elastic modulus) and the onsite soil properties as represented by standard penetration test blow counts corrected for hammer efficiency (N_{60}). Analyses assume footings are 2 feet in depth and underlain by 2 feet of engineered fill except for shear wall footings. For shear wall footings, analyses assume footings underlain by 4 feet of engineered fill to mitigate excessive settlement.

Settlements of the proposed structure will depend on the magnitude of the structural loads. Specific structural loads are not known at this time. Based on previous experience, the foundation loads summarized in Table 4.1 were used for evaluation to represent potential development concepts.

TABLE 4.1
Assumed Foundation Loads

Structure	Loading Type	Max. Dead plus Live Load
5-level parking garage structure	Interior Column Load	520 kips
	Shear Wall Load	42 kips/ft for 36 ft long
5-story wood frame building	Interior Column Load	75 kips
	Wall Load	7 kips/ft

A summary of the estimated static settlements and configurations is shown in Table 4.2 below.

TABLE 4.2
Estimated Static Settlement

Loading Type	Bearing Pressure (psf)	Footing Depth (ft.)	Footing Width (ft.)	Footing Length (ft.)	Settlement Range (inches)
5-level parking garage structure					
Interior Column Load	4,000	2	14	14	0.6 – 0.7
Shear Wall Load	5,250	2	8	36	0.7 – 0.9
5-story wood frame building					
Interior Column Load	3,000	2	5	5	0.2 – 0.3
Wall Load	3,500	2	2	99	0.3 – 0.4

Once finalized structural loads and details of footing designs are provided to us, static settlements may require re-evaluation.

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 Puente Hills (Coyote Hills) fault, located approximately 1.78 miles away. Therefore, the potential for ground rupture due to an earthquake beneath the sites is considered 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 are 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.

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.

Considering that the site is not underlain by relatively loose silty and/or sandy soils and that groundwater was not encountered to the maximum depth of 51.5 feet during our site exploration and historic high groundwater is deeper than 50 feet, the potential for liquefaction to occur beneath the site is considered very low. The site is also not located within a State-designated zone of potentially liquefiable soils. As such, no mitigation is deemed necessary for adverse effects from liquefaction.

5.3 STATIC SETTLEMENT

Provided site grading is performed in accordance with the recommendations provided herein and based on the anticipated foundation loads, total and differential static settlement is not anticipated to exceed 1 inch and ½-inch over 30 feet, respectively, for the proposed structure. The estimated magnitude of static settlement is 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 on 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. The upper soils tend to be at or below optimum moisture content. As such, recompaction of soils will generally require the addition of water during grading.

Temporary construction slopes will be required to complete removal of unsuitable soils and for construction of underground utilities. Such excavations will require laybacks where they are surcharged or where they exceed 4 feet in height.

5.5 SHRINKAGE AND SUBSIDENCE

Volumetric changes in earth quantities will occur when excavated onsite soil materials are replaced as properly compacted fill. Removal and recompaction of existing fill materials (generally the upper 8 feet) is anticipated to result in shrinkage of about 15 to 20 percent. 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 for balancing earthwork quantities based on actual shrinkage and subsidence 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. Additional testing for soil expansion will be required 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 earthwork and grading should be performed in accordance with applicable requirements of Cal/OSHA, applicable specifications of the Grading Codes of the City of Placentia, 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. Asphaltic concrete and Portland Cement concrete can be incorporated into the fill as recommended in Section 6.1.5.

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, any existing artificial fill and upper loose young alluvial fan soils are considered unsuitable for support of the proposed structures at the site. These materials should be removed from within the limits of the proposed building, retaining walls, and any other "structural" areas and replaced as engineered compacted fill. The removal depth of artificial fill and upper young alluvial fan soils is estimated to vary from 4 to 6 feet below current grades. Deeper removals may be required for the footings supporting the parking garage structure depending on the actual foundation loads. The removals should extend laterally a distance of at least 5 feet beyond the limits of the residential structures or a 1:1 (H:V) projection down and away from the bottom of the footings, whichever is greater. Once final foundation loads are known, the requirements for removal depths should be re-evaluated by the geotechnical consultant.

Within the limits of pavement and screen walls, the existing soils should be removed to 1 foot below subgrade and 1 foot below bottom of footings. Such removals should extend at least 2 feet beyond the edges of pavement and footings where feasible.

The actual depth of removal should be determined by the geotechnical consultant during grading. 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.

If these removals are restricted due to the presence of existing features such as property lines, additional considerations may be required in the design and construction of site improvements affected by these limitations. 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 110 percent of 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 in site materials that are not surcharged may be cut vertically up to a height of 4 feet. Temporary excavations greater than 4 feet but no greater than 10 feet in height that are not surcharged should be laid back at a maximum gradient of 1.5:1 (H:V) or properly shored.

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.

Where temporary excavations cannot accommodate a 1.5:1 layback or where surcharging occurs, shoring, slot cutting, underpinning, or other methods should be used. Specific recommendations for other options if considered should be provided by the geotechnical consultant based on review of the final design plans.

6.2 SEISMIC DESIGN PARAMETERS

6.2.1 Mapped Seismic Design Parameters

For design of the project in accordance with Chapter 16 of the 2019 CBC, the mapped seismic parameters may be taken as presented in the tables below.

TABLE 6.1
2019 CBC Mapped Seismic Design Parameters

Parameter	Value
Site Class	D
Mapped MCE_R Spectral Response Acceleration, short periods, S_S	1.610
Mapped MCE_R Spectral Response Acceleration, at 1-sec. period, S_1	0.568
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.7*
Adjusted MCE_R Spectral Response Acceleration, short periods, S_{MS}	1.610
Adjusted MCE_R Spectral Response Acceleration, at 1-sec. period, S_{M1}	0.966
Design Spectral Response Acceleration, short periods, S_{DS}	1.074
Design Spectral Response Acceleration, at 1-sec. period, S_{D1}	0.756
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, "a ground motion hazard analysis shall be performed in accordance with Section 21.2 for the following structures on Site Class D and E sites with S_1 greater than or equal to 0.2." However, "A ground motion hazard analysis is not required for structures where: Structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Eq. (12.8-2) for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$." The F_v value of 1.7 above from Table 11.4-2 assumes that this exception is met and that a ground motion hazard analysis is not required. 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
2019 CBC Site Specific Seismic Design Parameters

Parameter	Value
Site Coefficient, F_a	1.0
Site Coefficient, F_v	2.5
Adjusted MCE Spectral Response Acceleration, short periods, S_{MS}	1.855
Adjusted MCE Spectral Response Acceleration, at 1-sec. period, S_{M1}	1.136
Design Spectral Response Acceleration, short periods, S_{DS}	1.237
Design Spectral Response Acceleration, at 1-sec. period, S_{D1}	0.757

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 2,900 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 190 psf and 540 psf for each additional foot in width and depth, respectively, up to a maximum value of 4,500 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 300 pounds per square foot per foot of depth (psf/ft) up to a maximum value of 1,500 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 0.29 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 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 minimum 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 1745-11, 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.

Parking garage floor slabs should have a minimum thickness of 5 inches and should be reinforced with No. 3 bars spaced 18 inches on center, each way . Parking 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. The slab should be provided with cold joints or saw cuts at a maximum spacing of 15 feet in each direction. Cold joints should be provided with a keyway or dowels. Consideration should be given to providing a vapor barrier 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 that is equal to or greater than 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 50%.

6.4.3 Active Earth Pressures

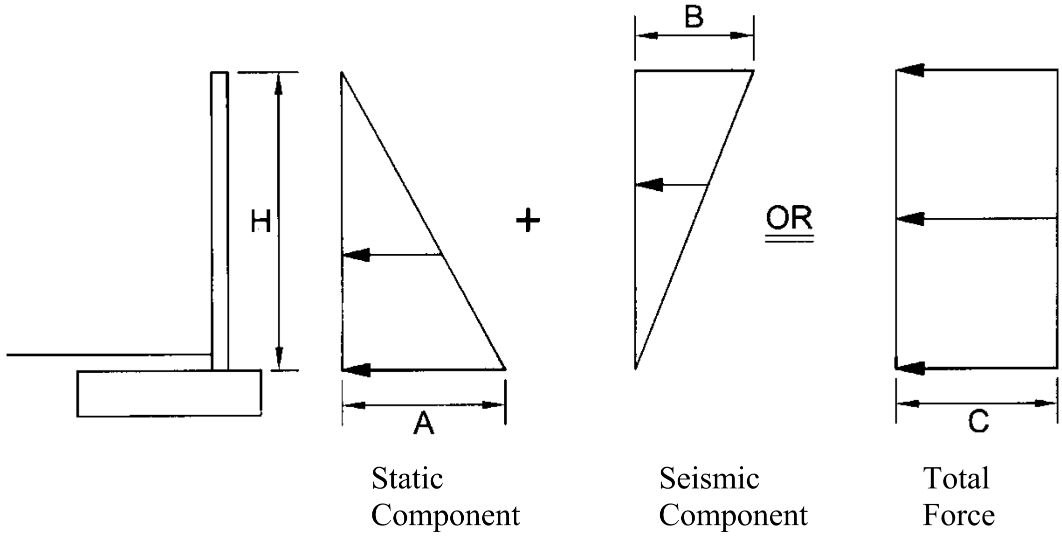
Static and seismic earth pressures for level and 2:1 (H:V) backfill conditions are provided in Table 6.3. Seismic earth pressures provided herein are based on the method provided by Seed & Whitman (1970) using a peak ground acceleration (PGA) of 0.44 g for 10% probability of exceedance in 50 years. As 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.

6.4.4 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 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.

TABLE 6.3

SEISMIC EARTH PRESSURES
Pressure Diagram



Pressure Values
Walls Up To 10 Feet High

Value	Backfill Condition	
	Level	2H:1V Slope
A	35H	60H
B	14H	14H
C	25H	37H

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.

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 few inches. The top of footing should be finished smooth with a trowel where the water-proofing is applied to inhibit the infiltration of water through the wall. 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 project structural engineer should provide specific recommendations for moisture-proofing, water stops, and joint details.

6.4.5 Wall Backfill

The values for active pressure are based on using backfill consisting of onsite materials having an Expansion Index of 20 or less. If import soil is used for retaining wall backfill, the earth pressures should be re-evaluated by the geotechnical consultant based on the properties of the import material. Import source(s) should be indicated to the geotechnical consultant prior to hauling the materials to the site so that appropriate testing and evaluation of the fill material can be performed in advance. Backfill should be placed within a 1:1 plane projected up from the base of the wall stem.

6.4.6 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

Exterior flatwork should be a nominal 4 inches thick. Cold joints or saw cuts should be provided at least every 15 feet in each direction. Special jointing detail 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 at least 110 percent of the optimum moisture content 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 ensure that the required compaction and pre-moistening recommendations have been met.

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.

6.6 CONCRETE MIX DESIGN

Laboratory testing of near-surface soils for soluble sulfate content indicates soluble sulfate concentration of 0.003%. We recommend following the procedures provided in ACI 318-14, Section 19.3, Table 19.3.2.1 for S0 sulfate exposure. Upon completion of rough grading, 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 CORROSION

Results of preliminary testing of soils for pH, chloride, and minimum resistivity indicate the site is potentially **Highly Corrosive** to metals that are in contact or close proximity to onsite soils. As such, specific recommendations should be obtained from a corrosion specialist if construction will include metals that will be near or in direct contact with site soils.

6.8 PRELIMINARY PAVEMENT DESIGN

6.8.1 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 30 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	5.5	3.0 4.0	--	--	8.0 5.0
		--	--	6.0	--
		--	80.0	--	8.0
Parking Stalls	--	3.0	--	--	5.0

6.8.2 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.

6.8.3 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.

6.8.4 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.

6.8.5 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
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

6.8.6 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 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 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.

6.9 POST GRADING CONSIDERATIONS

6.9.1 Site Drainage and Irrigation

The ground immediately adjacent to foundations should be provided with positive drainage away from the structures in accordance with 2019 CBC, Section 1804.4. However, the slope of the ground surface adjacent to the structure may be reduced to 2 percent for soils and climatic reasons. 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.

6.9.2 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 ¾ 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.

6.10 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 appears 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 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 **Orangethorpe Investment Partners LLC** 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.

Eung Jin Jeon, Ph.D.
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G.E. 3096

David E. Albus
Principal Engineer
G.E. 2455

8.0 REFERENCES

Publications

Abrahamson, N.A., Silva, W.J., and Kamai, R. 2014. "Update of the AS08 Ground-Motion Prediction Equations Based on the NGA-West2 Data Set." Earthquake Spectra 30 (3): 1179-1197.

American Society of Civil Engineers, 2017, Minimum Design Loads and Associated Criteria for Buildings and Other Structures, ASCE 7-16.

Boore, D.M., Stewart, J.P., Seyhan, E., and Atkinson, G.M. 2014. "NGA-West2 Equations for Predicting PGA, PGV, and 5% damped PSA for Shallow Crustal Earthquakes." Earthquake Spectra 30 (3): 1057-1085.

California Geologic Survey. 1998. Earthquake Zones of Required Investigation Anaheim Quadrangle. Released April 15, 1998.

California Geologic Survey. 2008. Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A.

California Department of Conservation, Division of Mines and Geology. 1997. "Seismic Hazard Zone Report for the Anaheim and Newport Beach 7.5-Minute Quadrangle, Orange Counties, California." Seismic Hazard Report 037.

Campbell, K.W., and Bozorgnia, Y. 2014. "NGA-West2 Ground Motion Model for the Average Horizontal Components of PGA, PGV, and 5%-damped Linear Acceleration Response Spectra." Earthquake Spectra 30 (3): 1087-1115.

Chiou, B.S.-J., and Youngs, R.R. 2014. "Update of the Chiou and Youngs NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra, Earthquake Spectra, 30 (3): 1117-1153.

Field, E.H., Biasi, G.P., Bird, P., Dawson, T.E., Felzer, K.R., Jackson, D.D., Johnson, K.M., Jordan, T.H., Madden, C., Michael, A.J., Milner, K.R., Page, M.T., Parsons, T., Powers, P.M., Shaw, B.E., Thatcher, W.R., Weldon, R.J., II, and Zeng, Y., 2013, Uniform California earthquake rupture forecast, version 3 (UCERF3)—The time-independent model: U.S. Geological Survey Open-File Report 2013–1165, 97 p., California Geological Survey Special Report 228, and Southern California Earthquake Center Publication 1792, http://pubs.usgs.gov/of/2013/1165/.

Seed, H.B., and Whitman, R.V. "Design of Earth Retaining Structures for Dynamic Loads," ASCE Specialty Conference, Lateral Stresses in the Ground and Design of Earth Retaining Structures, Cornell Univ., Ithaca, New York, 103-147, 1970

REFERENCES (cont.)

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. 2001. "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils." ASCE Journal of Geotechnical and Geoenvironmental Engineering 127 (10): 817-833.

Aerial Photographs

Photo Source	Flown Date	Flight Number
Continental Aerial Photo, Inc.	1999-02-23	C133-33-223
Continental Aerial Photo, Inc.	1997-10-16	C118-34-104
Continental Aerial Photo, Inc.	1993-06-09	C93-15-109
Continental Aerial Photo, Inc.	1992-01-24	C85-5-14
Continental Aerial Photo, Inc.	1990-06-12	C84-14-31
Continental Aerial Photo, Inc.	1986-12-30	F-115
Continental Aerial Photo, Inc.	1983-03-30	218-6-15
Continental Aerial Photo, Inc.	1981-01-31	211-7-9
Continental Aerial Photo, Inc.	1980-02-25	80033-37
Continental Aerial Photo, Inc.	1978-12-10	203-6-17
Continental Aerial Photo, Inc.	1975-01-13	157-6-13
Continental Aerial Photo, Inc.	1973-10-29	132-6-4
Continental Aerial Photo, Inc.	1970-01-30	60-5-146
Continental Aerial Photo, Inc.	1967-03-01	1-29
Continental Aerial Photo, Inc.	1967-03-01	1-30
Continental Aerial Photo, Inc.	1959-03-25	261-3-15-117
Continental Aerial Photo, Inc.	1952-12-26	AXK-5K-80
Continental Aerial Photo, Inc.	1952-12-26	AXK-5K-81
Continental Aerial Photo, Inc.	1949-07-14	13990-1-73

APPROX. SCALE
1" = 60'

EXPLANATION
(Locations Approximate)

- Exploratory Boring
- Percolation Test Boring

ALBUS & ASSOCIATES

GEOTECHNICAL MAP

Job No.: 2967.00	Date: 04/28/21	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>

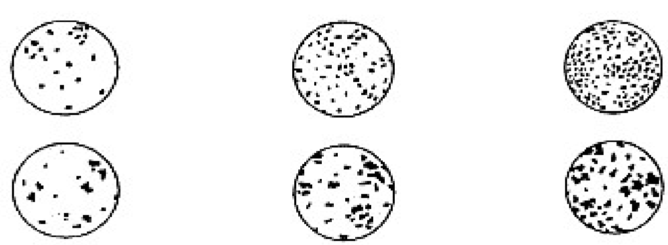
Fine grained soils	SPT	CA
Very Soft	2<	0-3
Soft	2-4	3-6
Medium Stiff	4-8	6-13
Stiff	8-15	13-24
Very Stiff	15-30	24-48
Hard	30>	48>

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

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

Depth (feet)	Lith- ology	Material Description	Water	Samples			Laboratory Tests		
				Blows Per Foot	Core	Bulk	Moisture Content (%)	Dry Density (pcf)	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) 200 = Percent Passing #200 Sieve Consol = Consolidation SE = Sand Equivalent Rval = R-Value ATT = Atterberg Limits							
20									

Albus & Associates, Inc.

EXPLORATION LOG

Project:		Location: B-1
Address: 777 W Orangethorpe Ave, Placencia, CA 92870		Elevation: 197.3
Job Number: 2967.00	Client: Orangethorpe Investment Partners, LLC	Date: 4/14/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: sdonyanavard

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
		Asphalt : 3 inch asphalt							
		Crushed Aggregate Base : 3 inch base							
		ARTIFICIAL FILL (Af) Silty Sand trace Clay (SM): Brown, moist, very loose, fine to medium grained sand, ac fragments, some silt spots, wood fragment		5			11.5	109.6	
		YOUNG ALLUVIAL FAN DEPOSIT (Qyf) Silty Sand trace Clay (SM): Brown, moist, very loose, fine to medium grained sand		8			4.4	95.2	
5		Sand (SP): Light gray to light brown, damp, loose, fine to medium grained sand							
		@ 6 ft, dry to damp, fine to coarse grained sand		10			2.1	99.7	
10		@ 10 ft, medium dense, some coarse gravel		22			2.8	100.1	200
15		@ 15 ft, light brown, medium to coarse grained sand		21			3.1	103.7	SA

EXPLORATION LOG

Project:		Location: B-1
Address: 777 W Orangethorpe Ave, Placencia, CA 92870		Elevation: 197.3
Job Number: 2967.00	Client: Orangethorpe Investment Partners, LLC	Date: 4/14/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: sdonyanavard

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
		@ 20 ft, dense, increased coarse grained sand		36			3.2		
25				19					SA
30		@ 30 ft, moist, medium dense, fine to coarse grained sand		13					
35		@ 35 ft, dense		26					200

Albus & Associates, Inc.

EXPLORATION LOG

Project:		Location: B-1
Address: 777 W Orangethorpe Ave, Placencia, CA 92870		Elevation: 197.3
Job Number: 2967.00	Client: Orangethorpe Investment Partners, LLC	Date: 4/14/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: sdonyanavard

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
		@ 40 ft, light gray, fine to medium grained sand		20					
45		Sand trace Silt (SP): Grayish brown, moist, very dense, fine grained sand		29					200
50		@ 50 ft, dense, fine to medium grained sand, rock fragments		23					
		End of Boring at depth of 51.5 feet. No groundwater encountered. Backfilled with soil cuttings and patched with cold patch asphalt.							

EXPLORATION LOG

Project:		Location: B-2
Address: 777 W Orangethorpe Ave, Placencia, CA 92870		Elevation: 201.4
Job Number: 2967.00	Client: Orangethorpe Investment Partners, LLC	Date: 4/14/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: sdonyanavard

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
		Asphalt (AC): 3 inch asphalt							
		Crushed Aggregate Base (CAB): 4 inch base							
		ARTIFICIAL FILL (Af) Silty Sand (SM): Brown, damp, loose, fine to medium grained sand, asphalt fragmens		6			7.9	101.2	
5		**YOUNG ALLUVIAL FAN DEPOSIT (Qyf)** Silty Sand (SM): Brown, damp, loose, fine to medium grained sand		6			7.6	99.3	
		Sand with Silt (SP-SM): Olive brown, moist, loose, fine to coarse grained sand, pin-hole pores, rootlets		14			2.7	98	
		Sand (SP): Light gray, dry to damp, medium dense, fine to coarse grained sand, some fine gravel							
10		@ 10 ft, medium to coarse grained sand		20			1.8	100.1	
15		light brown, damp, no gravel		19					200
20		@ 20 ft, light brown to light gray		32					

EXPLORATION LOG

Project:		Location: B-2
Address: 777 W Orangethorpe Ave, Placencia, CA 92870		Elevation: 201.4
Job Number: 2967.00	Client: Orangethorpe Investment Partners, LLC	Date: 4/14/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: sdonyanavard

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
				16					
		End of boring at depth of 26.5 feet. No groundwater encountered. Backfilled with soil cuttings and patched with cold patch asphalt.							

EXPLORATION LOG

Project:		Location: B-3
Address: 777 W Orangethorpe Ave, Placencia, CA 92870		Elevation: 198.4
Job Number: 2967.00	Client: Orangethorpe Investment Partners, LLC	Date: 4/14/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: sdonyanavard

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
		Asphalt (AC): 3 inch asphalt							
		Crushed Aggregate Base (CAB): 4 inch base							
		ARTIFICIAL FILL (Af) Silty Sand with Clay (SM): Brown, moist, very loose, fine to medium grained sand, asphalt fragments, rock fragments		5			14.7	100	
				5			12.6	95.2	Consol 200
5		**YOUNG ALLUVIAL FAN DEPOSIT (Qyf)** Sand trace Silt (SM): Brown, moist, very loose, fine to coarse grained sand		27			5.9		
		Sand with Gravel (SP): light gray to light brown, damp, medium dense, medium to coarse grained sand, fine gravel							
		Sand (SP): Light gray to light brown, damp, medium dense, medium to coarse grained sand							
10		@ 10 ft, moist, fine to coarse grained sand		22			2.3	104.4	
15		Sand trace Gravel (SP): Light brown, moist, medium dense, fine to coarse grained sand, fine gravel		24			2.9	107.8	
20		Sand (SP): Light brown to light gray, damp to moist, medium dense, medium to coarse grained sand		31			3.3	101.6	200

EXPLORATION LOG

Project:		Location: B-3
Address: 777 W Orangethorpe Ave, Placencia, CA 92870		Elevation: 198.4
Job Number: 2967.00	Client: Orangethorpe Investment Partners, LLC	Date: 4/14/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: sdonyanavard

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 ft, dense		22					
		End of boring at depth of 26.5 feet. No groundwater encountered. Backfilled with soil cuttings and patched with cold patch asphalt.							

EXPLORATION LOG

Project:		Location: B-4
Address: 777 W Orangethorpe Ave, Placencia, CA 92870		Elevation: 201.4
Job Number: 2967.00	Client: Orangethorpe Investment Partners, LLC	Date: 4/14/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: sdonyanavard

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
		Asphalt (AC): 3 inch asphalt							
		Crushed Aggregate Base (CAB): 4 inch base							
		ARTIFICIAL FILL (Af) Clayey Sand (SC): Brown, moist to very moist, loose, fine to medium grained sand, asphalt fragments, rock fragments		12			17.9	105.3	
5		**YOUNG ALLIVIAL FAN DEPOSITS (Qyf)** Silty Sand with Clay (SM): Brown, moist, very loose, fine to coarse grained sand, pin-hole pores		5			10.6	92.6	
		Sand (SP): Light gray, dry to damp, medium dense, medium to coarse grained sand, some fine gravel		13			2.6	99.9	
10		@ 10 ft, light brown		19			1.9		
15		@ 15 ft, light gray		30			2.1	109.9	
20		@ 20 ft, increased coarse grained sand		27			2.2	104	

Albus & Associates, Inc.

Plate A-9

EXPLORATION LOG

Project:		Location: B-4
Address: 777 W Orangethorpe Ave, Placencia, CA 92870		Elevation: 201.4
Job Number: 2967.00	Client: Orangethorpe Investment Partners, LLC	Date: 4/14/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: sdonyanavard

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 ft, fine to coarse grained sand		16					
30		@ 30 ft, dense, medium to coarse grained sand		18					
35		<u>Sand trace Clay (SP):</u> Light brown to brown, moist, medium dense, fine to coarse grained sand, increased clay content toward sampler tip		11					
		End of the boring at depth of 36.5 feet. No groundwater encountered. Backfilled with soil cuttings and patched with cold patch asphalt.							

EXPLORATION LOG

Project:		Location: B-5
Address: 777 W Orangethorpe Ave, Placencia, CA 92870		Elevation: 198.5
Job Number: 2967.00	Client: Orangethorpe Investment Partners, LLC	Date: 4/14/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: sdonyanavard

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
		Asphalt (AC): 3 inch asphalt							Max EI SO4 DS pH Resist Ch
		Crushed Aggregate Base (CAB): 3 inch base							
		ARTIFICIAL FILL (Af) Silty Sand trace Clay (SM): Brown, moist, loose, fine to coarse grained sand, asphalt fragments, rock fragments		6			14.3	100.6	
5		**YOUNG ALLUVIAL FAN DEPOSITS (Qyf)** Silty Sand trace Clay (SM): Brown, moist, loose, fine to coarse grained sand		9			3.9	98.7	
		Sand trace Silt (SP): Brown, moist, loose, fine to coarse grained sand		13			4.6	99.3	
		Sand (SP): Light brown to light gray, dry to damp, loose, fine to coarse grained sand							
		@ 6 ft, damp, medium dense, fine to medium grained sand							
10		@ 10 ft, dry to damp, medium to coarse grained sand		17			2.7	100.3	
15		@ 15 ft, light brown, damp to moist, fine to coarse grained sand		24			7	103.3	
20				29			3.1	99.7	

Albus & Associates, Inc.

EXPLORATION LOG

Project:		Location: B-5
Address: 777 W Orangethorpe Ave, Placencia, CA 92870		Elevation: 198.5
Job Number: 2967.00	Client: Orangethorpe Investment Partners, LLC	Date: 4/14/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: sdonyanavard

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
				13					
30		@ 30 ft, grayish brown, dense, fine grained sand		26					
35		@ 35 ft, fine to medium grained sand		23					
		End of boring at depth of 36.5 feet. No groundwater encountered. Backfilled with soil cuttings and patched with cold patch asphalt.							

Albus & Associates, Inc.

Plate A-12

EXPLORATION LOG

Project:	Location: B-6
Address: 777 W Orangethorpe Ave, Placencia, CA 92870	Elevation: 200.1
Job Number: 2967.00 / Client: Orangethorpe Investment Partners, LLC	Date: 4/14/2021
Drill Method: Hollow-Stem Auger / Driving Weight: 140 lbs / 30 in	Logged By: sdonyanavard

Depth (feet)	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
	Asphalt (AC): 4 inch asphalt							
	Crushed Aggregate Base (CAB): 6inch base							
	ARTIFICIAL FILL (Af) Silty Sand trace Clay (SM): Brown, moist, loose, fine to medium grained sand, asphalt fragments, some clay spots		6	■		13.6	102.5	
			11	■		2.4	99.8	
5	**YOUNG ALLUVIAL FAN DEPOSIT (Qyf)** Silty Sand trace Clay (SM): Brown, damp, loose, fine to medium grained sand, pin-hole pores		22	■		1.9	103.1	
	Sand (SP): Light gray, dry, medium dense, fine to medium grained sand							
	@ 6 ft, light gray to light brown, fine to coarse grained sand							
10	@ 10 ft, damp, medium to coarse grained sand		25	■		3	101.6	
15	@ 15 ft, some fine to coarse gravel		27	■		N.R.		
20			22	■				

EXPLORATION LOG

Project:		Location: B-6
Address: 777 W Orangethorpe Ave, Placencia, CA 92870		Elevation: 200.1
Job Number: 2967.00	Client: Orangethorpe Investment Partners, LLC	Date: 4/14/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: sdonyanavard

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 ft, dense		21					
		End of boring at depth of 26.5 feet. No groundwater encountered. Backfilled with soil cuttings and patched with cold patch asphalt.							

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 2488). The samples were re-examined in the laboratory, and classifications were 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 summarized on the Exploration Logs, Appendix A.

Maximum Dry Density and Optimum Moisture Content

Maximum dry density and optimum moisture content were performed on a representative sample 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-1.

Expansion Potential

Expansion index testing was performed on a selected sample. The test was performed in accordance with ASTM D4829. The test result and expansion potential are presented in Table B-1.

Soluble Sulfate Content

A chemical analysis was performed on a selected sample to determine soluble sulfate content. This test was performed in our soil laboratory in accordance with California Test Method No 417. The test result is included in Table B-1.

Particle Size Analyses

Particle size analyses were performed on representative samples of site materials in accordance with ASTM D 422. The results are presented graphically on the attached Plate B-1.

Consolidation

Consolidation tests were performed for a selected soil sample in general conformance with ASTM D 2435. Axial loads were applied in several increments to a laterally restrained 1-inch-high sample. Loads were applied in geometric progression by doubling the previous load, and the resulting deformations were recorded at selected time intervals. The specific test samples were inundated at selected loads to evaluate the effects of a sudden increase in moisture content (hydro-consolidation potential). The result of the test is graphically presented on Plate B-3.

<u>Direct Shear</u>

The Coulomb shear strength parameters, angle of internal friction and cohesion, were determined for a bulk sample obtained from one of our borings. The tests were performed in general conformance with Test Method ASTM D 3080. The bulk 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-4.

<u>Corrosion</u>

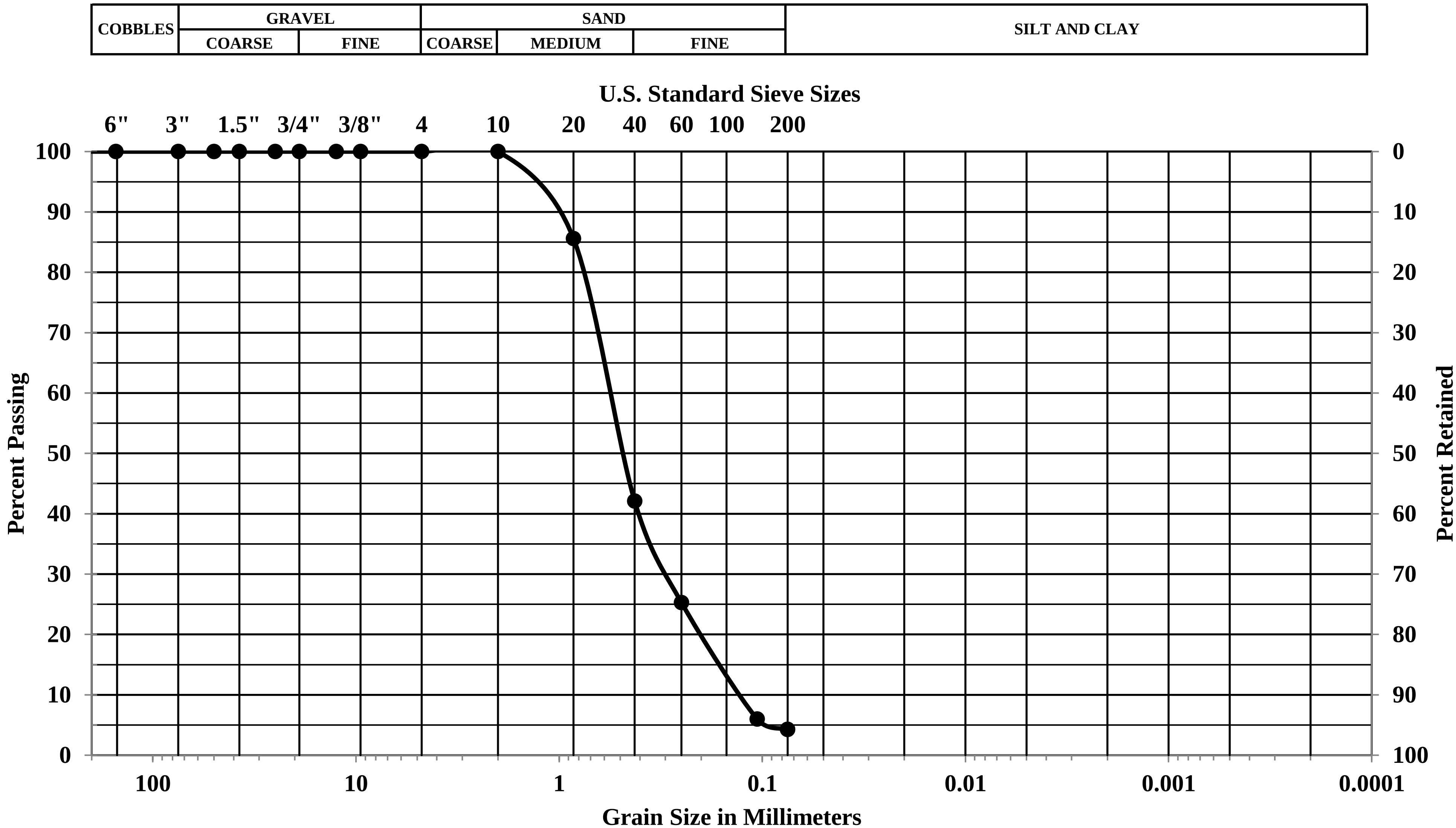
Select samples were tested for minimum resistivity, chloride, pH in accordance with California Test Method 643. Results of these tests are provided in Table B-1.

TABLE B-1
SUMMARY OF LABORATORY TEST RESULTS

Boring No.	Sample Depth (ft)	Soil Description	Test Results	
B-1	10	Sand (SP)	Passing No. 200 Sieve:	1.6
B-1	35	Sand (SP)	Passing No. 200 Sieve:	3.7
B-1	45	Sand trace Silt (SP)	Passing No. 200 Sieve:	6.3
B-2	15	Sand (SP)	Passing No. 200 Sieve:	3.7
B-3	4	Sand trace Silt (SP)	Passing No. 200 Sieve:	8.6
B-3	20	Sand (SP)	Passing No. 200 Sieve:	0.1
B-5	0-5	Sand with Silt trace Clay (SP)	Max. Dry Density (pcf): Opt. Moisture Content (%): Expansion Index: Expansion Potential: Soluble Sulfate Content: Sulfate Exposure: pH: Chloride content (ppm): Resistivity (ohms):	132.0 7.5 3 Very Low 0.003 % Negligible 5.65 30 3000

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

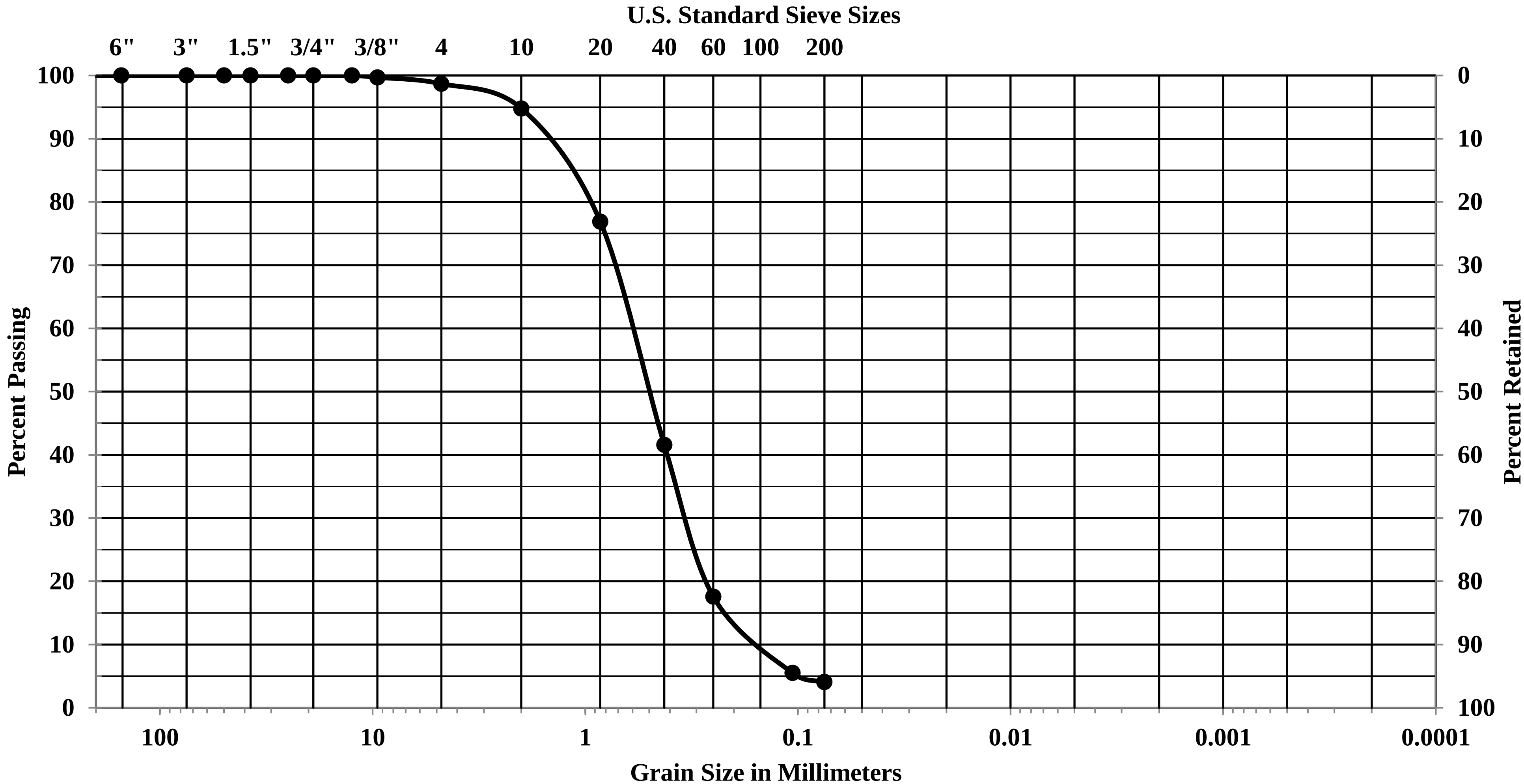
GRAIN SIZE DISTRIBUTION



Job Number	Location	Depth	Description
2967.00	B-1	15	Sand (SP)

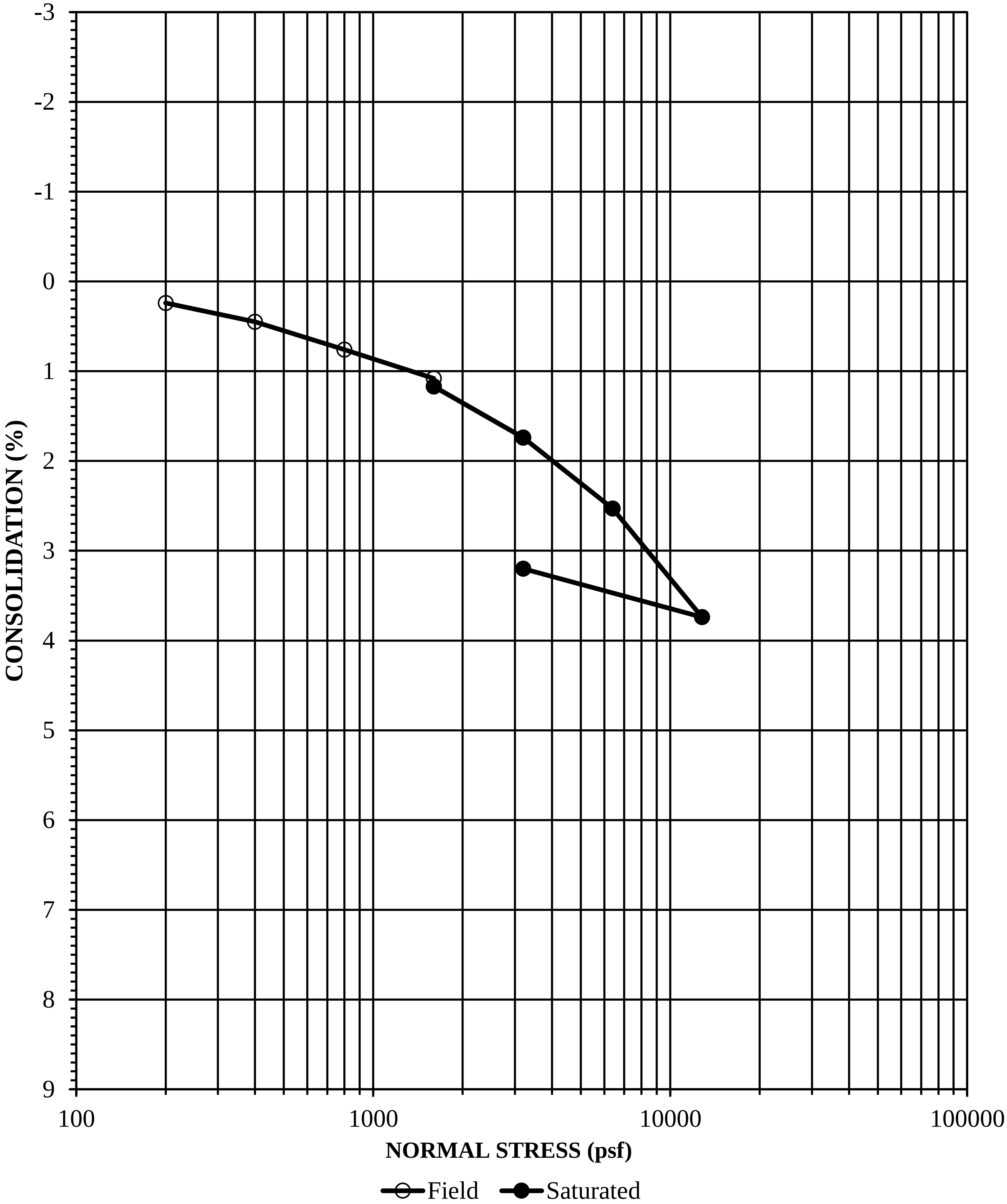
GRAIN SIZE DISTRIBUTION

COBBLES	GRAVEL		SAND			SILT AND CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	



Job Number	Location	Depth	Description
2967.00	B-1	25	Sand (SP)

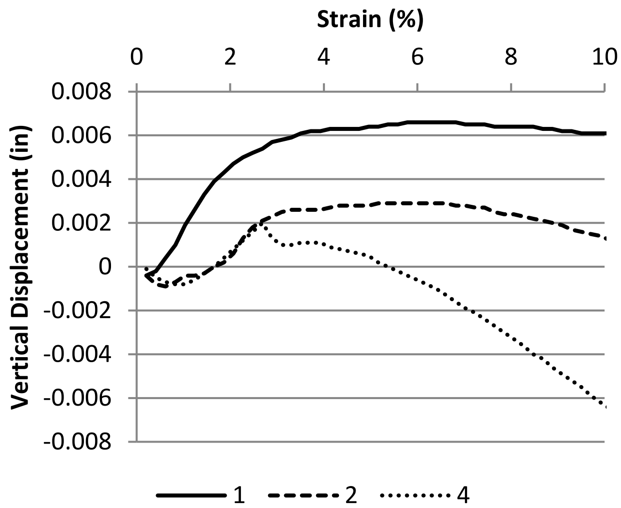
CONSOLIDATION

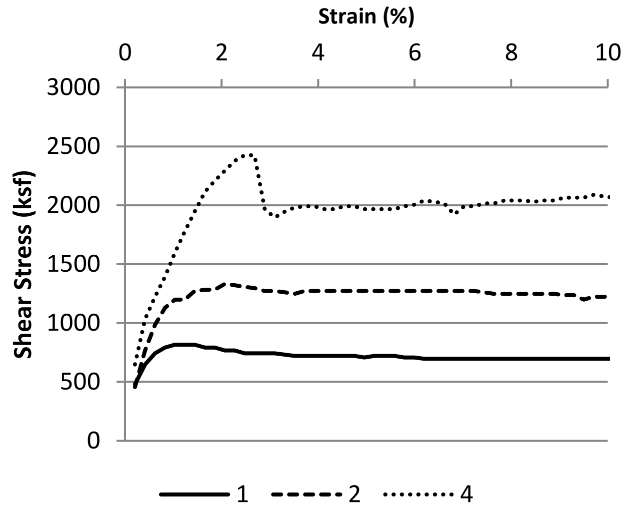


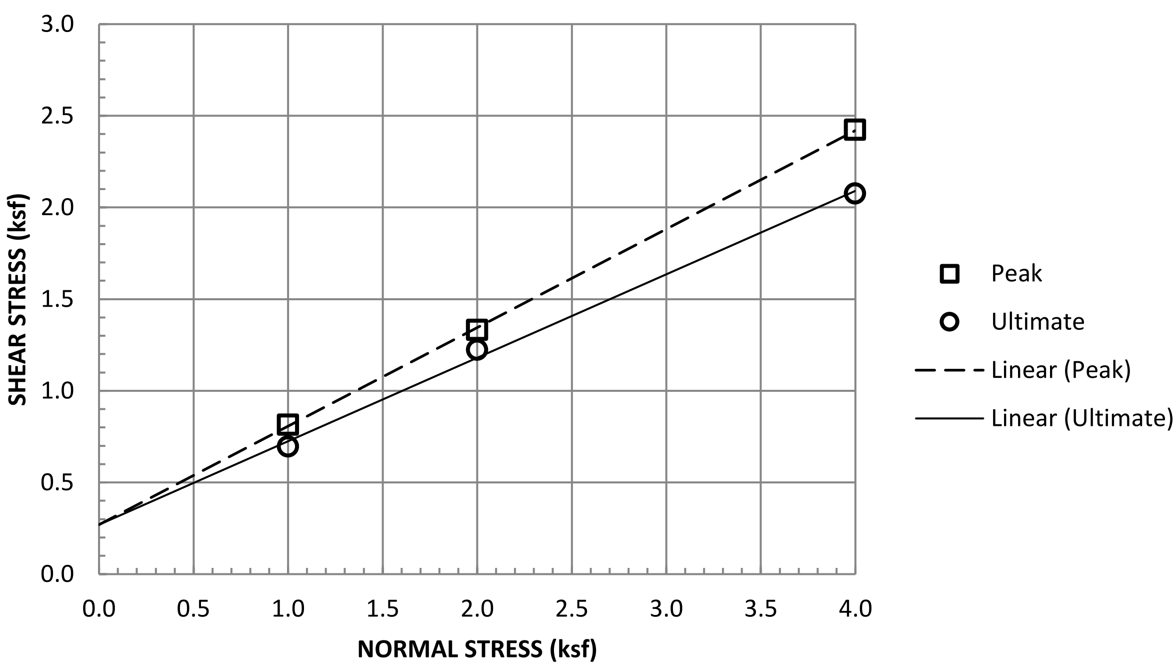
Job Number	Location	Depth	Description
2967.00	B-3	4	Sand trace Silt

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
98.9	9.9	20.6

DIRECT SHEAR







Sample Type:	Remolded, Saturated		
Normal Stress (ksf)	1	2	4
Peak Shear Stress (ksf)	0.816	1.332	2.424
Peak Displacement (in)	0.007	0.003	0.007
Ultimate Shear Stress (ksf)	0.696	1.224	2.076
Ultimate Displacement (in)	0.25	0.25	0.25
Initial Dry Density (pcf)	117.7	117.7	117.7
Initial Moisture Content (%)	7.4	7.4	7.4
Final Moisture Content (%)	14.1	13.8	13.3
Strain Rate (in/min)	0.005		

Job Number	Location	Depth	Description
2967.00	B-5	0-5	Sand with Silt trace Clay