ALBUS & ASSOCIATES

October 27, 2021
J.N.: 3016.00

Mr. Erik Pfahler
Borstein Enterprises
11766 Wilshire Boulevard, Suite 820
Los Angeles, CA 90025

Subject: Preliminary Geotechnical Investigation, Proposed Residential Development, 8601 Mission Drive, Rosemead, California

Dear Mr. Pfahler,

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.

David E. Albus
Principal Engineer

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1.0 INTRODUCTION

1.0 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

1.1 SITE LOCATION AND DESCRIPTION

The site is located at 8601 Mission Drive within the city of Rosemead, California. The APN parcel numbers for the current development are 5389-009-029, -030, and -031. The property is bordered by Mission Drive to the south, Walnut Grove Avenue, an existing easement for power lines, and a nursery to the west, and single-family residences to the north and east. The location of the site and its relationship to the surrounding areas are shown in Figure 1, Site Location Map.

The site consists of an irregularly shaped property containing approximately 3.35 acres of land. The site is relatively flat with elevations ranging from 357 to 363 feet above mean sea level (based on Google Earth). The site slopes gently down to the south. The site is currently vacant land with some improvements onsite. The perimeters of the site are bounded by chain-link fencing, masonry block walls, and plastic fencing. A short concrete driveway is located to the south and west. Along the southwest boundary of the property is existing overhead powerlines. Vegetation within the site consists of minor ground cover and some large palm trees within the southwest portion of the site.

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FIGURE 1-SITE LOCATION MAP

Proposed Residential Development
8601 Mission Drive,
Rosemead, California

NOT TO SCALE

1.2 PROPOSED DEVELOPMENT

Based on our understanding, site development is anticipated to consist of multi-story (2 to 3), wood-framed buildings at grade. Associated interior driveways, decorative hardscape, parking areas and underground utilities are also anticipated.

No grading or structural plans were available in preparing this proposal. However, we anticipate some minor cut and filling of the site will be required to achieve future surface configuration and we expect future foundation loads will be moderate.

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 1953, the site appears to have been graded but no improvements were constructed. By 1965, a large building had been constructed with a driveway entering from Walnut Grove Avenue to the western side of the building. An additional entrance from Mission drive runs to the south of the building and joins what appears to be an asphaltic parking area occupying the areas east and north of the building. From 1965 to 1988, the site appears relatively unchanged. By 1990, the building and associated improvements were gone and the site appears to have remained relatively unchanged since.

2.2 SUBSURFACE EXPLORATION

Subsurface exploration for this investigation was conducted on September 7, 2021, and consisted of drilling six (6) soil borings to depths ranging from approximately 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. 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 adjacent to boring B-1 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, Atterberg limits, 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

Review of the Diblee Map for the El Monte and Baldwin Park Quadrangles shows the site is designated as Quaternary Alluvium and falls within a flood plain and would have been subjected to seasonally-deposited materials associated with heavy rains from nearby mountain ranges to the north. Our exploration encounted artificial fills overlaying alluvial soils. 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.

Fills up to about 2 feet thick appear to be present on site due to previous site improvements and grading. Fill materials typically consisted of fine to medium-grained silty sands.

Alluvium was encountered underlying the artificial fill and generally consisted of interlayered silty sands, sands with silt, gravelly sands, and clayey sands. These materials were typically medium dense to very dense and damp. Alluvial soils were encountered to the maximum depth explored (51.5 feet).

Within borings B-2 and B-3, cobbles were encountered at depths of approximately 10 feet. Due to the size of the cobbles, the hollow stem could not extract all the cobbles. As such, the cobbles generally remained within the borings and floated within the cuttings and around the stem. Sizes were either measured or visually observed within the boring shaft and are estimated to be 4 to 6 inches in diameter.

3.2 GROUNDWATER

Groundwater was not encountered during this firm's subsurface exploration to a depth of 51.5 feet. The CDMG Special Report 024 suggests that historic high groundwater for the subject site is approximately 60 feet. However, review of the Los Angeles County groundwater level data for the nearby well 2920G indicates that groundwater for the area is 231 feet below ground surface as of 2018. Well readings have been recorded from 5/1/1949 to 4/26/2018, and during this period, groundwater has fluctuated, but has continued to increase in depth from 126 feet (bgs) to 231 feet (bgs) during this time period. The last recorded reading at the time of this report was April 26, 2018 and indicated a depth of 231 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)
Elysian Park (Upper)	1.74	1.3	50	reverse	3	20
Raymond	2.81	1.5	79	strike slip	0	22
Verdugo	5.15	0.5	55	reverse	0	29
Sierra Madre Connected	6.11	2	51	reverse	0	76
Sierra Madre	6.11	2	53	reverse	0	57
Elsinore;W+GI+T+J+CM	6.66	n/a	84	strike slip	0	241
Elsinore;W	6.66	2.5	75	strike slip	0	46
Elsinore;W+GI	6.66	n/a	81	strike slip	0	83
Elsinore;W+GI+T	6.66	n/a	84	strike slip	0	124
Elsinore;W+GI+T+J	6.66	n/a	84	strike slip	0	199
Clamshell-Sawpit	7.75	0.5	50	reverse	0	16
Puente Hills (LA)	8.61	0.7	27	thrust	2.1	22

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 84^{th} 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 coefficient (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 0.762g.

4.2 STATIC SETTLEMENT

Analyses were performed for the potential settlement of the underlying soils encountered during our investigation. The site is predominately granular in nature. As such, analyses of settlement were based on the elastic method using estimated modulli 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 1.2 feet wide and a wall load of 3,000 psf. The second model was based on a conventional square footing with 3 feet width and a column load of 3,000 psf. Both models yielded a total settlement of less than 0.5 inches. Both analyses assume the upper 2 feet of existing fill soils would be removed and replaced as compacted fill.

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 Elysian Park (Upper) fault located approximately 1.74 miles away. Therefore, 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 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.

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 zone of potentially liquefiable soils. Additionally, groundwater was not encountered to the maximum depth of 51.5 feet drilled during our site exploration. Furthermore, groundwater well measurements by the Los Angeles County in the vicinity of the project site since 1949 indicate that groundwater has been deeper than 50 feet for more than 70 years (well 1617N). Therefore, historical high groundwater is anticipated to be deeper than 50 feet below the ground surface. 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 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 on deleterious debris and oversized rocks (greater than 6 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. Such excavations will require laybacks where they are surcharged or where they exceed 4 feet in height.

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. We estimate that the existing upper 5 feet of soils will shrink approximately 8 to 14 percent when removed and replaced as compacted fill. Subsidence due to reprocessing of removal bottoms is anticipated to be on the order of 0.10 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. 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.

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 Rosemead, 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 artificial fill is considered unsuitable for support of the proposed development. Artificial fill is estimated to typically be 2 feet thick. These materials should be removed from within the limits of residential structures and retaining walls. The removals should extend a minimum distance of 5 feet beyond the foundations. Artificial fill should also be removed below subgrade for pavement and screen walls. Such removals should extend at least 2 feet beyond the edges of pavement and footings where feasible. In addition to the general removal of existing fills, the alluvial soils should be over-excavated to a minimum depth of 1 foot below the bottoms of footings for the residential buildings.

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.

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 6 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

The upper 2 to 4 feet of site materials generally consist of silt sands that may be temporarily cut vertical up to a maximum height of 4 feet provided they are not surcharged. Temporary excavations greater than 4 feet or exposing friable sandy soils 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. Based on the anticipated removal depths discussed herein and the current minimum setback of 15 feet for buildings from property lines, we anticipate that removals can be accomplished with open cuts. However, if deeper removals are required during grading or final building locations are closer to property lines and thereby result in insufficient room for recommended lay back cuts, shoring may be required. Additional recommendations for such conditions can be provided after reviewing final site plans and during grading.

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	1.997
Mapped MCE_R Spectral Response Acceleration, at 1-sec. period, S_1	0.722
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.7*
Adjusted MCE_R Spectral Response Acceleration, short periods, S_{MS}	1.997
Adjusted MCE_R Spectral Response Acceleration, at 1-sec. period, S_{M1}	1.227
Design Spectral Response Acceleration, short periods, S_{DS}	1.331
Design Spectral Response Acceleration, at 1-sec. period, S_{D1}	0.818
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 Class	D
Site Coefficient, F_a	1.0
Site Coefficient, F_v	2.5
Adjusted MCE Spectral Response Acceleration, short periods, S_{MS}	1.837
Adjusted MCE Spectral Response Acceleration, at 1-sec. period, S_{M1}	1.444
Design Spectral Response Acceleration, short periods, S_{DS}	1.225
Design Spectral Response Acceleration, at 1-sec. period, S_{D1}	1.087

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 3,000 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 300 psf and 800 psf for each additional foot in width and depth, respectively, up to a maximum value of 4,000 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 250 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 0.48 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 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.

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 12 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.

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 0.44 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.

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.

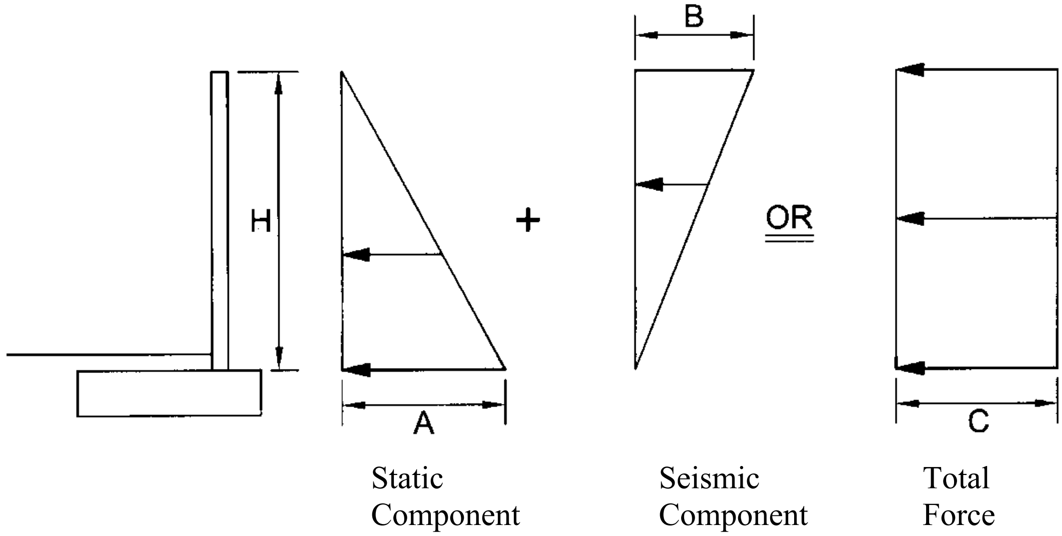
Retaining walls supporting backfill should also be coated with a moisture-proofing compound or covered with such material to inhibit infiltration of moisture through the walls. Moisture-proofing material should cover any portion of the back of wall that will be in contact with soil and should lap over 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.

TABLE 6.3

SEISMIC EARTH PRESSURES
Pressure Diagram



Pressure Values
Walls Up To 10 Feet High

Value	Backfill Condition	
	Level	2H:1V Slope
A	28H	41H
B	14H	14H
C	21H	28H

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

6.5 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 10 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 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.

6.6 CONCRETE MIX DESIGN

Laboratory testing of near-surface soils for soluble sulfate content indicates soluble sulfate concentration of 0.001%. We recommend following the procedures provided in ACI 318, Section 4.3, Table 4.3.1 for **negligible** sulfate exposure. Upon completion of rough grading, an evaluation of as-graded conditions and further laboratory testing should be completed for the site to confirm or modify the recommendations provided in this section.

6.7 CORROSION

Results of preliminary testing of soils for pH, chloride, and minimum resistivity indicate the site is potentially **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
		--	--	6.5	--
		--	80.0	--	8.0
Parking Stalls	--	3.0	--	--	4.0

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 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.

6.9.2 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 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 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 **Borstein Enterprises** 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.

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0 50 100 200

APPROX SCALE : 1" = 100'

EXPLANATION
(Locations Approximate)

- Exploratory Boring
- Percolation Test Boring

ALBUS & ASSOCIATES

GEOTECHNICAL MAP

Job No.: 3016.00	Date: 10/27/2021	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>

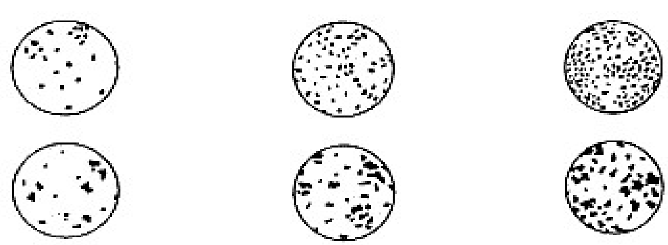
Fine grained soils		
Very Soft	2<	0-3
Soft	2-4	3-6
Medium Stiff	4-8	6-13
Stiff	8-15	13-24
Very Stiff	15-30	24-48
Hard	30>	48>

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

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

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

EXPLORATION LOG

Project:		Location: B-1
Address: 8601 Mission Dr, Rosemead, CA		Elevation: 358
Job Number: 3016.00	Client: Borstein Enterprises	Date: 9/7/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: ddalbus

Depth (feet)	Material Description	Water	Samples: Blows Per Foot	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
	ARTIFICIAL FILL (Af) Silty Sand (SM): Light brown, dry, fine to medium grained sand					Max EI SO4 DS pH Resist Ch
	ALLUVIUM (Qal) Sand with Gravel trace Silt (SP): Brown to yellowish brown, damp, medium dense, fine to coarse grained sand		17	2.9	112.4	
5			20	3.2	103.9	
	Sand with Gravel (SP): Yellowish brown, damp to moist, medium dense, fine to coarse grained sand		28	2.6	112.8	
	Gravelly Sand (SW): Yellowish brown, damp to moist, medium dense, fine to coarse grained sand					
10	@ 10 ft, more coarse grained sand		34	3.6	117.4	
15	@ 15 ft, dry to damp, very dense		37			SA Hydro
	Silty Sand (SM): Brown to yellowish brown, moist, very dense, fine grained sand					
20	@ 20 ft, medium dense		10			200
25	Sand with Gravel trace Silt (SP): Brown, moist, dense, more fine grained sand		21			

EXPLORATION LOG

Project:		Location: B-1
Address: 8601 Mission Dr, Rosemead, CA		Elevation: 358
Job Number: 3016.00	Client: Borstein Enterprises	Date: 9/7/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: ddalbus

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
		Yellowish brown, very dense, more fine to coarse grained sand		52					SA Hydro
35		Gravelly Sand (SP): Light reddish brown, dry to damp, very dense, fine to coarse grained sand		49					200
40		Sand with Silt (SP): Light reddish brown, damp to moist, very dense, fine grained sand		36					
45		Silty Sand (SM): Light reddish brown, damp to moist, very dense, fine grained sand, moderate fines		61					200
		Clayey Sand (SC): Brown to reddish brown, moist, hard, fine grained sand							
50				30					ATT
		Total Depth 51.5 feet No Groundwater Boring backfilled with soil cuttings							

Albus & Associates, Inc.

EXPLORATION LOG

Project:		Location: B-2
Address: 8601 Mission Dr, Rosemead, CA		Elevation: 357.9
Job Number: 3016.00	Client: Borstein Enterprises	Date: 9/7/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: ddalbus

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ARTIFICIAL FILL (Af) Silty Sand (SM): Brown, dry, fine grained sand							
		ALLUVIUM (Qal) Silty Sand with Gravel (SM): Brown, damp, medium dense, fine to coarse grained sand		28			2.7		
		Gravelly Sand (SP): Yellowish brown, moist, medium dense, fine to coarse grained sand		25			3.3	112.4	
5									
		@ 6 ft, dense, gravel up to 1 inch dia		30					
10		@ 10 ft, medium dense, cobbles observed in cuttings up to 6 inch dia		15					200
		Total Depth 11.5 feet No Groundwater Boring backfilled with soil cuttings							

Albus & Associates, Inc.

EXPLORATION LOG

Project:		Location: B-3
Address: 8601 Mission Dr, Rosemead, CA		Elevation: 357.9
Job Number: 3016.00	Client: Borstein Enterprises	Date: 9/7/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: ddalbus

Depth (feet)	Lithology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ARTIFICIAL FILL (Af) Silty Sand (SM): Brown, dry, fine grained sand							
		ALLUVIUM (Qal) Gravelly Sand trace Silt (SP): Yellowish brown, dry to damp, medium dense, fine to coarse grained sand		19	■		2.3		
		@ 4 ft, damp, loose, no silt, gravel up to 1 inch dia		12	■		3.1	104.9	
5									
		@ 6 ft, moist, medium dense		27	■		3.4	109.8	Consol
10									
		@ 10 ft, 4 inch dia cobble observed in cuttings				■			
		Total Depth 11.5 feet No Groundwater Boring backfilled with soil cuttings							

Albus & Associates, Inc.

Plate A-5

EXPLORATION LOG

Project:		Location: B-4
Address: 8601 Mission Dr, Rosemead, CA		Elevation: 358
Job Number: 3016.00	Client: Borstein Enterprises	Date: 9/7/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: ddalbus

Depth (feet)	Lith-ology	Material Description	Water	Samples			Laboratory Tests		
				Blows Per Foot	Core	Bulk	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
		ARTIFICIAL FILL (Af) Silty Sand (SM): Brown, dry, fine grained sand							
		ALLUVIUM (Qal) Silty Sand with Gravel (SM): Brown, moist, loose, fine to coarse grained sand		15			3.8	107.2	
5		Sand with Gravel (SP): Yellowish brown, moist, loose, fine to coarse grained sand		13			2.8		
		@ 6 ft, medium dense		17			4.5	105.9	Consol
10		@ 10 ft, more coarse grained sand		31			3.7	112.9	
		Total Depth 11.5 feet No Groundwater Boring backfilled with soil cuttings							

EXPLORATION LOG

Project:			Location: B-5
Address: 8601 Mission Dr, Rosemead, CA			Elevation: 361
Job Number: 3016.00	Client: Borstein Enterprises		Date: 9/7/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in		Logged By: ddalbus

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ARTIFICIAL FILL (Af) Silty Sand (SM): Brown, dry, fine grained sand							
		ALLUVIUM (Qal) Silty Sand with Gravel (SM): Brown, damp, medium dense, fine to medium grained sand		17			3.4	108.7	
		@ 4 ft, Yellowish brown		20			4.6	-31.3	
5									
		Sand with Gravel (SP): Yellowish brown, moist, medium dense, fine to coarse grained sand		34			3.1	111.1	
10									200
		Total Depth 11.5 feet No Groundwater Boring backfilled with soil cuttings							

EXPLORATION LOG

Project:		Location: B-6
Address: 8601 Mission Dr, Rosemead, CA		Elevation: 361.2
Job Number: 3016.00	Client: Borstein Enterprises	Date: 9/7/2021
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: ddalbus

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ARTIFICIAL FILL (Af) Silty Sand (SM): Brown, dry, fine grained sand							
		ALLUVIUM (Qal) Silty Sand with Gravel (SM): Brown, damp, medium dense, fine to coarse grained sand		16			2.1	108.5	
		Gravelly Sand (SP): Yellowish brown, damp, loose, fine to coarse grained sand		15			2.4		200
5									
		@ 6 ft, medium dense, more coarse grained sand		24			2.4	109.8	
10		@ 10 ft, dense, more gravel		46			3.4	103.2	
		Total Depth 11.5 feet No Groundwater Boring backfilled with soil cuttings							

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 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 on 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 on Table B-1.

Atterberg Limits

Atterberg Limits (Liquid Limit, Plastic Limit, and Plasticity Index) were performed in accordance with Test Method ASTM D 4318. Pertinent test values are presented within 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 Plates B-1 and B-2.

Direct Shear

The Coulomb shear strength parameters, angle of internal friction and cohesion, were determined for a bulk sample and intact samples obtained from one 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-3.

Corrosion

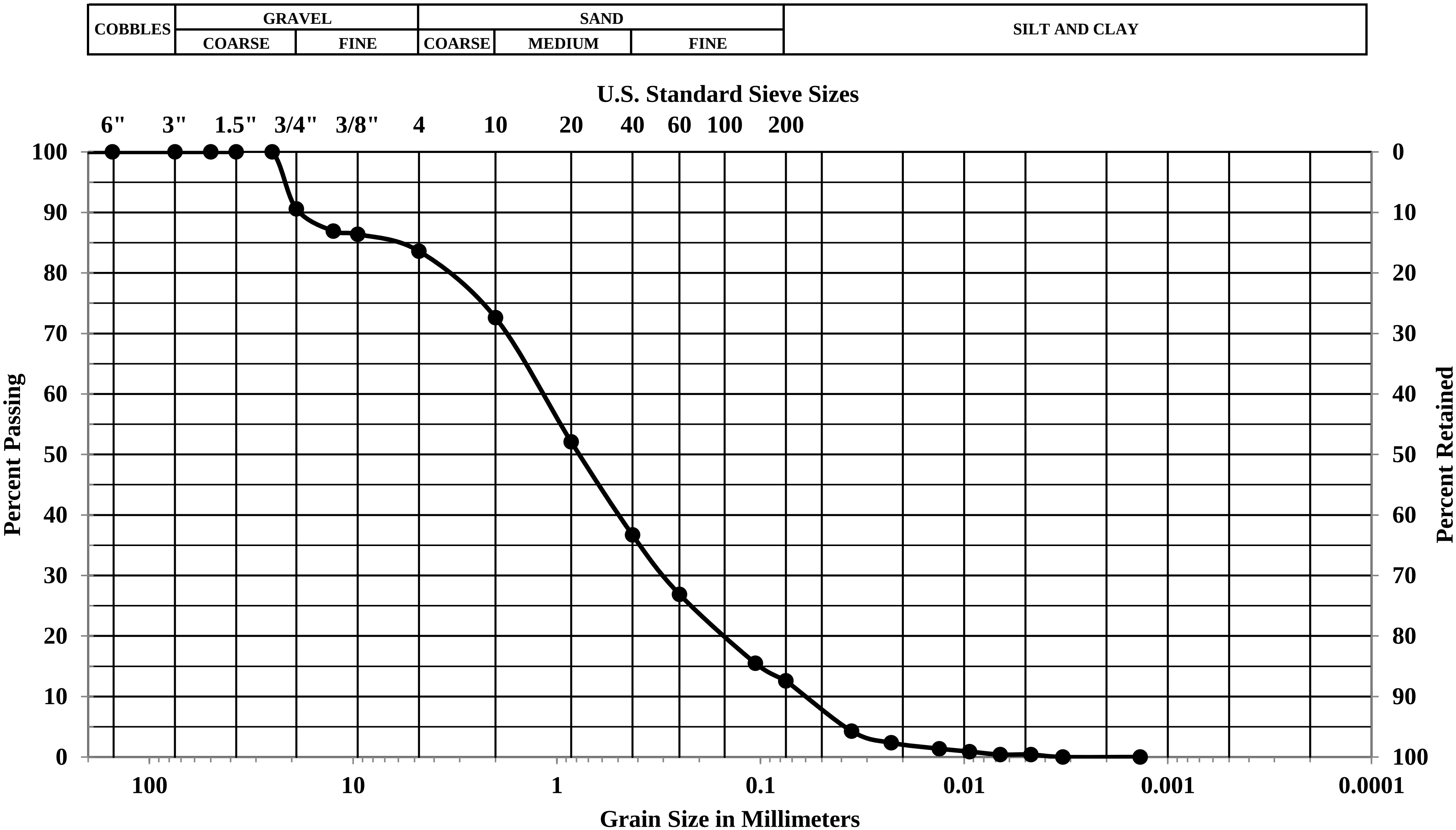
Select samples were tested for minimum resistivity, chloride, pH in accordance with California Test Method 643. Results of these tests are provided in Table B-1.

TABLE B-1
SUMMARY OF LABORATORY TEST RESULTS

Boring No.	Sample Depth (ft)	Soil Description	Test Results	
B-1	0-5	Silty Sand with Gravel	Max. Dry Density (pcf): Opt. Moisture Content (%): Expansion Index: Expansion Potential: Soluble Sulfate Content: Sulfate Exposure: PH: Chloride content (ppm): Resistivity (ohms):	132.5 6.5 0 Very Low 0.005% Negligible 6.86 75 3900
B-1	20	Silty Sand	Passing No. 200 Sieve:	43.4
B-1	35	Gravelly Sand trace Silt	Passing No. 200 Sieve:	11.1
B-1	45	Sand with Silt	Passing No. 200 Sieve:	17.7
B-1	50	Sandy Clay with Silt	Liquid Limit (%): Plasticity Index (%):	25 7.2
B-2	10	Gravelly Sand trace Silt	Passing No. 200 Sieve:	10.7
B-5	10	Gravelly Sand with Silt	Passing No. 200 Sieve:	20.3
B-6	4	Sand trace Silt	Passing No. 200 Sieve:	6.2

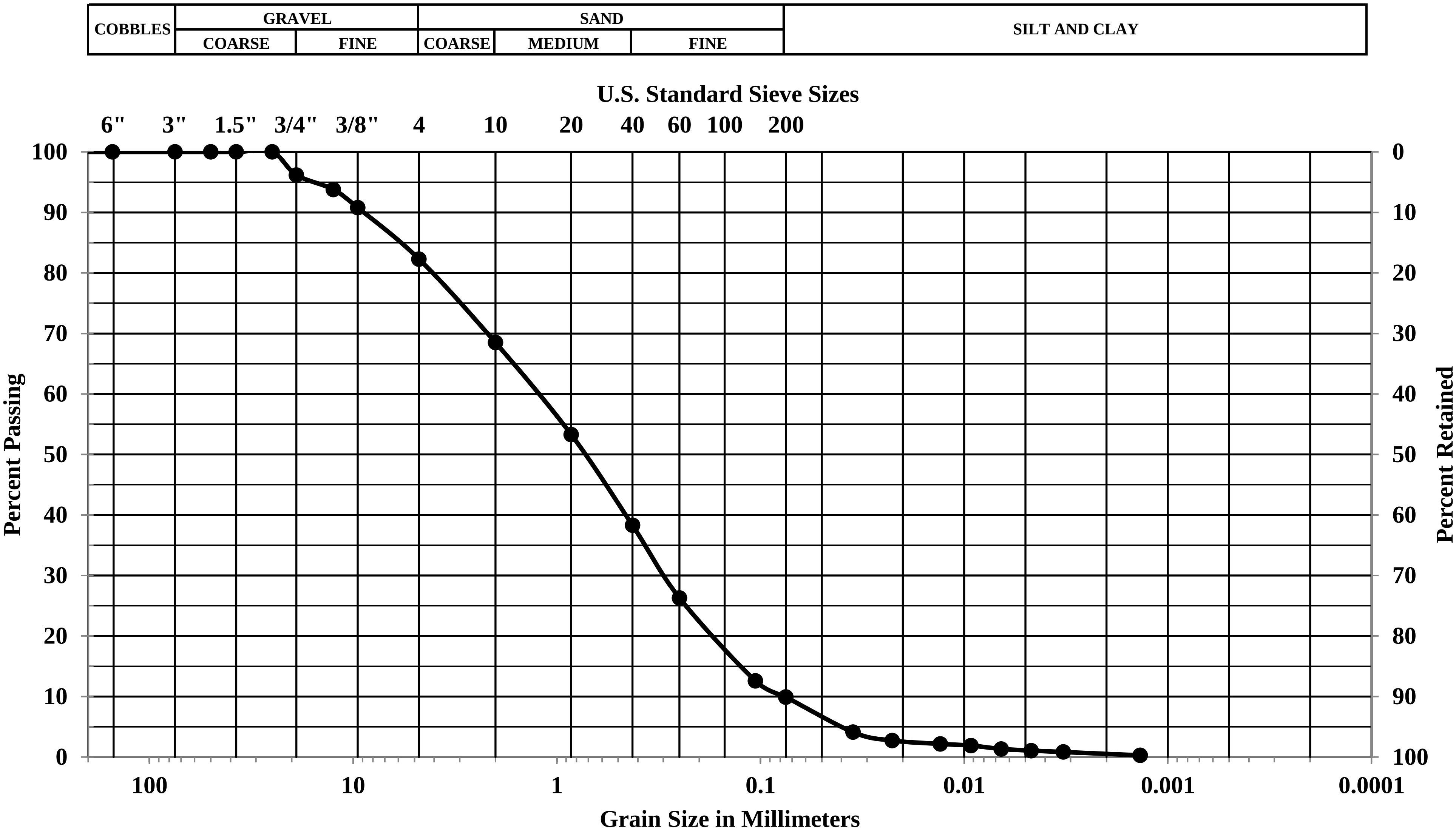
Note: Additional laboratory test results are provided on the boring logs provided in Appendix A.

GRAIN SIZE DISTRIBUTION



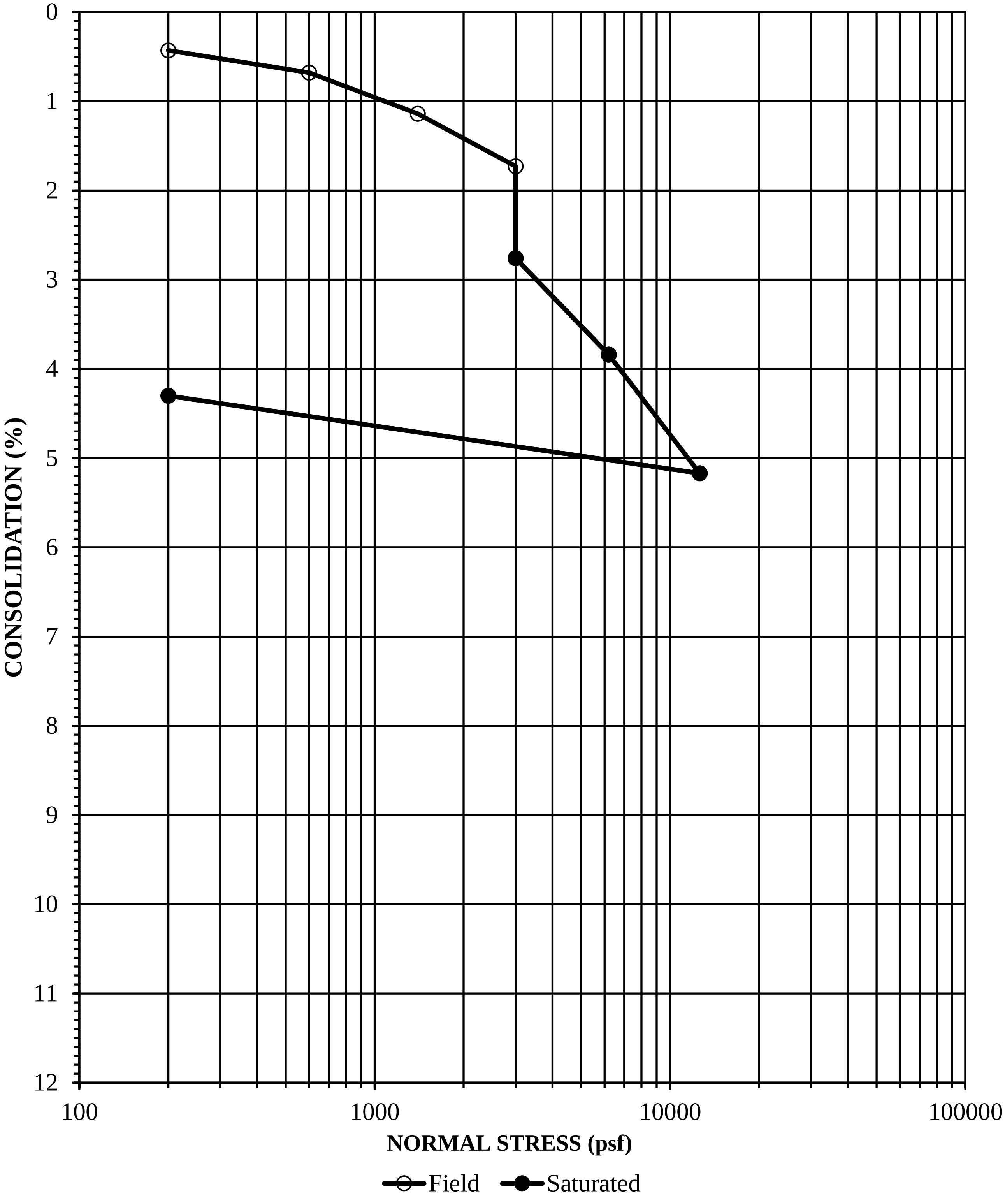
Job Number	Location	Depth	Description
3016.00	B-1	15	Sand with Gravel trace Silt

GRAIN SIZE DISTRIBUTION



Job Number	Location	Depth	Description
3016.00	B-1	30	Sand with Gravel trace Silt

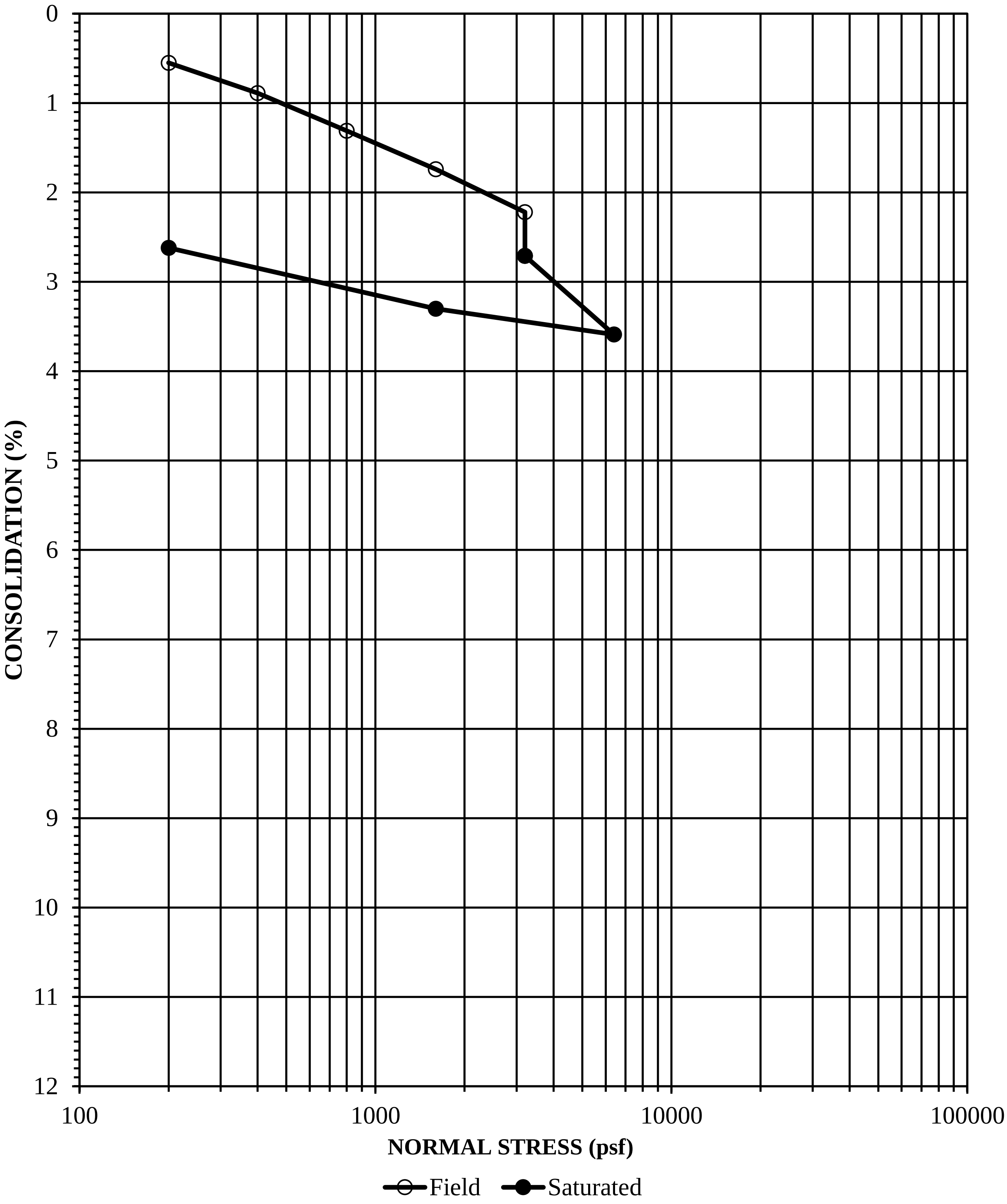
CONSOLIDATION



Job Number	Location	Depth	Description
3016.00	B-3	6	Sand

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
102.5	4	18.2

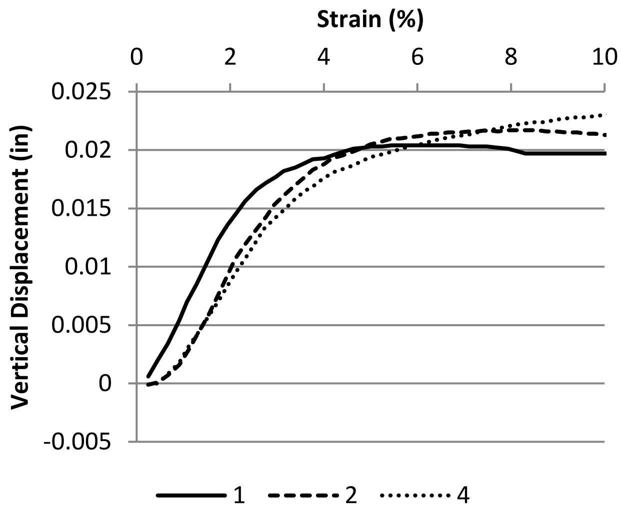
CONSOLIDATION

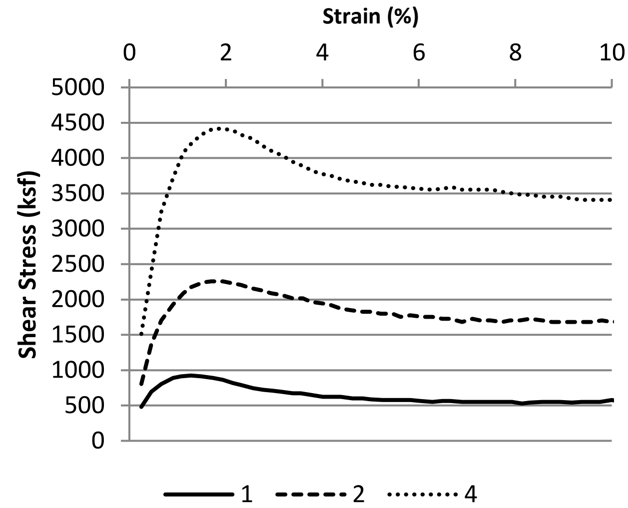


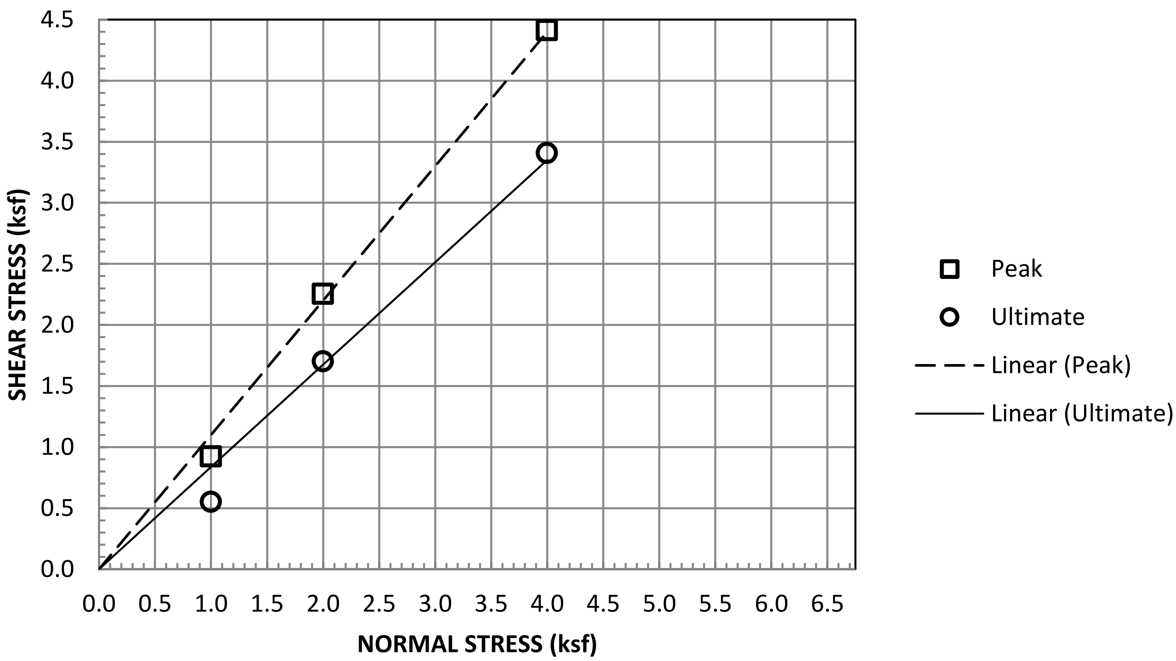
Job Number	Location	Depth	Description
3016.00	B-4	6	Sand

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
106.5	4.9	18.7

DIRECT SHEAR







Sample Type:	Saturated, Remolded		
Normal Stress (ksf)	1	2	4
Peak Shear Stress (ksf)	0.924	2.256	4.416
Peak Displacement (in)	0.02	0.022	0.023
Ultimate Shear Stress (ksf)	0.552	1.704	3.408
Ultimate Displacement (in)	0.24	0.25	0.25
Initial Dry Density (pcf)	124.5	124.5	124.5
Initial Moisture Content (%)	2	2	2
Final Moisture Content (%)	7.2	7.1	6.7
Strain Rate (in/min)	0.05		

Job Number	Location	Depth	Description
3016.00	B-1	0-5	Silty Sand with Gravel