APPENDIX E

GEOTECHNICAL REPORT

ALBUS-KEEFE & ASSOCIATES, INC
GEOTECHNICAL CONSULTANTS

January 10, 2020
J.N.: 2859.00

Ms. Sarah Walker
National Community Renaissance
4322 Piedmont Drive
San Diego, CA 92107

Subject: **Preliminary Geotechnical Investigation, Proposed Residential Development, 1314 Angelina Drive, Placentia, California.**

Dear Ms. Walker,

Pursuant to your request, ***Albus-Keefe & Associates, Inc***. is pleased to present to you our preliminary geotechnical investigation report for the subject development. This report presents the results of our field investigation, laboratory testing, engineering analyses, as well as our preliminary geotechnical recommendations for design and construction of the subject development.

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

Sincerely,

ALBUS-KEEFE & ASSOCIATES, INC.

Paul Kim
Associate 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 referenced conceptual site plan
- Review of published geologic and seismic data for the site and surrounding area
- Review of historical aerial photographs
- 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, and settlement potential
- Preparation of this report

1.2 SITE LOCATION AND DESCRIPTION

The site is located at 1314 North Angelina Drive within the city of Placentia, California. The property is bordered by North Angelina Drive to the West, single-family residences to the North and East, and Morse Avenue to the South. The location of the site and its relationship to the surrounding areas is shown on Figure 1, Site Location Map.

The site consists of a rectangular-shaped property containing approximately 4 acres of land. The site is relatively flat with elevations ranging from EL. 294 to EL. 297 above mean sea level (based on Google Earth) descending to the south-west. The site is currently occupied by Blessed Sacrament Episcopal Church. There are currently two existing structures and it appears that the structure located westerly is used for church gatherings. The easterly structure is used as a school facility. Associated parking areas are located along the southern boundary with vegetation occupying the remainder to the site. Perimeter walls run along the North and East boundaries and appear to be associated with the single-family residences.

Vegetation includes general landscaping in and around the structures, planters within the parking areas, grass and moderate to large sized trees within the open spaces.

N

SITE LOCATION MAP

Proposed Residential Development
1314 Angelina Drive
Placentia, California

NOT TO SCALE

FIGURE 1

1.3 PROPOSED DEVELOPMENT

Based on the conceptual site plan by RRM Design Group, dated September 5, 2019, the proposed project includes the development of two residential buildings accommodating 65 units. Building 1, at the north end of the site, is a linear two-story structure. Building 2 is a two-story, L-shaped building located interior to the site with a three-story element at the northern end of the building transitioning to two-stories toward the single-family neighborhood along the eastern property line. Associated parking, underground utilities and a storm water disposal system are also planned.

No grading or structural plans were available in preparing of this report. However, we anticipate that minor rough grading of the site will be required to achieve future surface configuration. We expect the proposed above-grade portion will be of wood-frame construction yielding relatively light foundation loads.

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 as early 1946, the subject site was part of a larger site and used for agricultural purposes. By 1967, the site was cleared of vegetation and the south half of the existing Church structure was constructed. Additionally, the north- and east-adjacent single-family residences have been constructed. By 1980, the north half of the existing Church structure was constructed. Also, at this time, the parking lot has likely been developed with asphalt. By 2002, the additional asphalt-paved parking appears east of the Church structure. By 2005, the school structure is present. The site has remained relatively unchanged since 2005.

2.2 SUBSURFACE EXPLORATION

Subsurface exploration for this investigation was conducted on December 17, 2019, and consisted of the drilling of four (4) soil borings to depths ranging from approximately 31.5 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-Keefe & 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.

One additional boring was drilled adjacent to boring B-1 for percolation testing. An additional percolation well was also installed in B-3. Details and results of percolation tests are reported under a separate cover.

2.3 LABORATORY TESTING

Selected samples of representative earth materials from our borings were tested in our laboratory. Tests consisted of USCS classification, in-situ moisture content and dry density, expansion index, maximum dry density and optimum moisture content, consolidation/collapse, direct shear strength, grain size analysis, percent passing No. 200 sieve, soluble sulfate content, and corrosivity testing (pH, chloride, and resistivity). Descriptions of laboratory testing and the test results are presented in Appendix B and on the Exploration Logs in Appendix A.

3.0 GEOLOGIC CONDITIONS

3.1 SOIL CONDITIONS

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

Soil materials encountered at the subject site generally consisted of Quaternary-aged alluvium (Qal). However, artificial fill materials were encountered within the parking lot at B-1 with an approximate thickness of 4 feet. The artificial fill consists of a sandy clay, grayish brown, moist, very stiff with fine to medium grained sand.

The alluvial materials were encountered to the maximum depth explored of 51.5 feet and are comprised of interbedded layers of damp to moist, reddish brown and light reddish-brown sandy clay, silty sand, clayey sand, silty clay, and sand. The granular alluvial soils are typically medium dense while the fine-grained alluvial soils are typically very stiff to hard.

A more detailed description of the interpreted soil profile at each of the boring locations, based upon the soil cuttings and soil samples, are presented in Appendix A. The stratigraphic descriptions in the logs represent the predominant materials encountered during investigation. Relatively thin, often discontinuous layers of different material may occur within the major divisions.

3.2 GROUNDWATER

Groundwater was not encountered during this firm's subsurface exploration to the maximum depth of 51.5 feet. Based on a review of the referenced CDMG Special Report, the historical groundwater for the site is not available. Additional review of the Department of Water Resources groundwater level data for the nearby well 338950N1178554W001 (approximately 2,600 feet to the northeast) indicates that groundwater for the area is below 150 feet in depth between 1970 to present. Review of well data from the State Water Resources Board GeoTracker database indicates groundwater levels in excess of 110 feet below the ground surface. These wells are estimated to be in generally similar geologic conditions based on review available geologic maps.

3.3 FAULTING

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

TABLE 3.1
Summary of Active Faults

Name	Distance (miles)	Slip Rate (mm/yr.)	Preferred Dip (degrees)	Slip Sense	Rupture Top (km)	Fault Length (km)
Puente Hills (Coyote Hills)	0.91	0.7	26	thrust	2.8	17
Elsinore;W+GI	2.97	n/a	81	strike slip	0	83
Elsinore;W+GI+T+J+CM	2.97	n/a	84	strike slip	0	241
Elsinore;W	2.97	2.5	75	strike slip	0	46
Elsinore;W+GI+T	2.97	n/a	84	strike slip	0	124
Elsinore;W+GI+T+J	2.97	n/a	84	strike slip	0	199
Puente Hills (Santa Fe Springs)	9.57	0.7	29	thrust	2.8	11
Puente Hills (Coyote Hills)	0.91	0.7	26	thrust	2.8	17
Elsinore;W+GI	2.97	n/a	81	strike slip	0	83
Elsinore;W+GI+T+J+CM	2.97	n/a	84	strike slip	0	241
Elsinore;W	2.97	2.5	75	strike slip	0	46
Elsinore;W+GI+T	2.97	n/a	84	strike slip	0	124

4.0 ANALYSES

4.1 SEISMICITY AND SEISMIC DESIGN PARAMETERS

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

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

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

In accordance with Section 21.2.2 (Supplement 1), the deterministic spectral response acceleration at each period was calculated as the 84th percentile, 5% damped, response acceleration, using the NGA-West2 GMPE Worksheet. For this, the information from at least three causative faults with the greatest contribution per deaggregation analysis were used, and the larger acceleration spectrum among these was selected as the deterministic response spectrum. The deterministic spectrum was adjusted per requirements in Section 21.2.2 (Supplement 1) where applicable. Both probabilistic and deterministic spectra were subjected to the maximum direction scale factors specified in Section 21.2 to produce the maximum acceleration spectra.

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

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

4.2 STATIC SETTLEMENT

Analyses were performed to evaluate the potential for static settlement of the underlying alluvial soils. Our analyses were based on the results of consolidation tests performed on selected samples from our borings as well as the recorded blow counts during the exploration. Results of our testing indicate the native site materials have low to moderate compressibility. In its current state, the native materials would result in excessive settlement due to the weight of new foundations.

The artificial fill soils were not considered in our settlement analysis as it is considered unsuitable for support of the proposed site development.

Provided remedial removals are performed, total and differential static settlement can likely be limited to a maximum of 1 inch and ½-inch over 30 feet, respectively. These estimated magnitudes of static settlements are considered within tolerable limits for the proposed structures.

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 the opinion that the proposed development, if constructed in accordance with the recommendations provided in our referenced report, will be safe against hazards from settlement, slippage, or landslides. The proposed site development will have no adverse effects on the stability of adjacent property if graded in accordance with this firm's recommendations and the approved rough grading plans.

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 active faults are known to project through the site nor does the site lie within the bounds of an "Earthquake Fault Zone" as defined by the State of California in the Los Angeles Earthquake Fault Zoning Act. As such, the potential for ground rupture due to fault displacement beneath the site is considered very low. The nearest zoned fault is the Whittier Fault located 3.5 miles to the northeast.

5.2.2 Ground Shaking

The site is located in a seismically active area that has historically been affected by 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 development, the property will probably experience 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 of proposed structures in accordance with the current CBC is anticipated to adequately mitigate concerns with ground shaking.

5.2.3 Landsliding

Geologic hazards associated with landsliding are not anticipated at the site due to not being located within an area identified by the California Geologic Survey (CGS) as having potential for seismic slope instability. Additionally, 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 liquefaction susceptibility of the onsite soils was evaluated by analyzing the potential of concurrent occurrence of the above-mentioned three basic factors. The liquefaction evaluation for the site was completed under the guidance of Special Publication 117A: Guidelines for Evaluating and Mitigating Seismic Hazards in California (CDMG, 2008).

Based on the historically low groundwater level, the potential for liquefaction at the site is considered to be low. Additionally, the site is not mapped within a State-designated zone of potentially liquefiable soils.

5.3 STATIC SETTLEMENT

The existing artificial fills are considered unsuitable for support of the proposed development. Additionally, the near-surficial alluvial soils are compressible which would result in excessive settlements for the proposed development in its current condition. Therefore, removal and recompaction of the existing surficial soils to provide a uniform compacted blanket will be necessary. Provided grading and construction are performed in accordance with the recommendations provided herein, estimated total and differential settlement of proposed site improvements are anticipated to be less than 1 inch and ½ inch over 30 feet, respectively. These magnitudes of settlement are considered within tolerable limits of proposed site development.

5.4 EARTHWORK AND MATERIAL CHARACTERISTICS

All artificial fill is considered unsuitable to support proposed site development. This condition can be mitigated by the removal and re-compaction of the unsuitable soils. The non-engineered fill is estimated to be approximately 3 feet in depth and located in the southwest corner of the site. Although, locally deeper conditions may exist and likely throughout the site, particularly in the vicinity of the existing structures.

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

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

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. Specific recommendations to provide for stable temporary cuts are provided later in this report. The use of appropriate shoring or lay backs will be essential to protect workers and prevent delays due to caving during trenching or temporary backcut activities. These materials will also be very prone to erosion during periods of rain until they are covered by pavement or mature landscaping. Appropriate protection during the rainy season will be required to avoid costly repairs due to erosion.

If encountered, portions of concrete debris and asphalt can likely be reduced in size (4” minus) and incorporated within fill soils during earthwork operations.

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

Off-site improvements exist near the property lines. The presence of the existing offsite improvements may limit removals of unsuitable materials adjacent the property lines. Special grading techniques, such as slot cuttings, will be required adjacent to property lines where offsite structures are nearby. Construction of perimeter site walls may require deepened footings where removals are restricted by property boundaries.

5.5 SHRINKAGE AND SUBSIDENCE

Volumetric changes in earth quantities will occur when excavated onsite soil materials are replaced as properly compacted fill. We estimate that the existing surficial soils will shrink approximately up to 10 percent. Subsidence due to reprocessing of removal bottoms is anticipated to be negligible. 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 USCS visual manual classification, the near-surface soils and the anticipated soils at basement subgrade within the site are generally anticipated to possess a **Low to medium** expansion potential. Additional testing for soil expansion will be required subsequent to rough grading and prior to construction of foundations and other concrete flatwork 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 all applicable requirements of the grading codes of the City of Placentia, California and CAL OSHA, in addition to recommendations presented herein.

6.1.2 Pre-Grade Meeting and Geotechnical Observation

Prior to commencement of earthwork operations and foundation installation, we recommend a meeting be held between the City Inspector, general contractor, civil engineer, and geotechnical consultant to discuss proposed earthwork and logistics.

We also recommend that a geotechnical consultant be retained to provide soil engineering and engineering geologic services during site development. This is to observe compliance with the design specifications and recommendations, and to allow design changes in the event that subsurface conditions differ from those anticipated. If conditions are encountered during construction that appears 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

Site improvements, such as asphaltic pavement, structural foundations and underground utilities, should be removed from the areas to be developed prior to any grading activities. 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.

In general, seepage pits that are open should be cleared of any fluids and then filled with 2-sack cement slurry up to within 5 feet of proposed grades. Any brick lining that remains in the upper 5 feet should be removed and the remainder of the pit filled with engineered fill in accordance with Section 6.1.5. Seepage pits that are presently backfilled with soil should be removed to a depth of 10 feet below pad grade and be capped with 2-sack cement slurry. The slurry cap should be at least 5 feet thick and should extend at least 12 inches outside the perimeter of the seepage pit. The remaining 5 feet should be filled with engineered fill in accordance with Section 6.1.5.

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 Site Preparation (Removals and Overexcavations)

In general, the artificial fill and the near-surface compressible materials are considered unsuitable for support of the proposed development at the site. These materials should be removed from proposed building, street and other "structural" areas, and replaced as engineered compacted fill. The removal depth is anticipated to be up to 4 feet and existing soils should be over-excavated to at least a depth of 2 feet below the bottom of footings for structures supported by conventional spread footings at grade. The actual depth of removal should be determined by the geotechnical consultant during grading.

The removals should extend laterally a distance of at least 5 feet beyond the limits of the proposed structures or a 1:1 projection down and away from the bottom of the footings, whichever is greater. Removals for retaining walls less than 3 feet in height and screen walls may be limited to the edge of the foundations or pavement. Upon review of more detailed site development plans, the depth of removals for short retaining walls and screen walls may be lessened from the general removals described above.

Where removals are limited by existing structures, protected trees or property lines, special considerations may be required in the construction of affected improvements. Under such conditions, specific recommendations should be provided by this firm based on review of site-specific development plans.

Following removals/excavation, the exposed grade should first be scarified to a depth of 6 inches, brought to at least 120 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 over 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 50 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 or trench excavations in site materials may be cut vertically up to a height of 4 feet provided that no surcharging of the excavations is present. Temporary slopes over 4 feet in height but no more than 10 feet in height should be laid back to 1:1 (H:V) or flatter and evaluated by the geotechnical consultant.

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 1:1 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 SEISMICITY

Following ASCE7-16, Section 21.5.3, we have estimated site-specific Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration PGA_M = 0.745g. Per Section 11.2, this value should be used for evaluation of liquefaction, lateral spreading, seismic settlements, and other soil-related issues. Based on the results of deaggregation analysis performed using USGS Unified Hazard Tool, the mean event associated with a probability of exceedance equal to 2% over 50 years has a moment magnitude of 6.68 and the mean distance to the seismic source is 5.6 miles.

6.3 SEISMIC DESIGN PARAMETERS

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

TABLE 6.1
CBC 2019 SEISMIC DESIGN PARAMETERS

Parameter	Value
Site Class	D
Mapped MCE Spectral Response Acceleration, short periods, S_S	1.730
Mapped MCE Spectral Response Acceleration, at 1-sec. period, S_1	0.609
Site Coefficient, Fa	**1.0**
Site Coefficient, Fv	**2.5**
Adjusted MCE Spectral Response Acceleration, short periods, S_{MS}	**1.891**
Adjusted MCE Spectral Response Acceleration, at 1-sec. period, S_{M1}	**1.465**
Design Spectral Response Acceleration, short periods, S_{DS}	**1.261**
Design Spectral Response Acceleration, at 1-sec. period, S_{D1}	**0.977**
Long-Period Transition Period, T_L (sec.)	8
Seismic Design Category for Risk Categories I-IV	**D**
MCE = Maximum Considered Earthquake	

Boldface values: Site-specific values per ASCE7-16; other values are mapped values.

6.4 FOUNDATION DESIGN

6.4.1 General

The following recommendations are provided for preliminary design purposes. These recommendations have been based on the site materials exposed during our investigation, our understanding of the proposed development, and the assumption that the recommendations presented herein are incorporated into the design and construction of the project. Final recommendations should be provided by the project geotechnical consultant following review of final foundation plans as well as observation and testing of site materials during grading. Depending upon the design plans and actual site conditions, the recommendations provided herein may require modification.

6.4.2 Soil Expansion

The recommendations presented herein are based on soils with a **Low to Medium** expansion potential (EI≤60). Following site grading, additional testing of site soils should be performed by the project geotechnical consultant to confirm the basis of these recommendations. If site soils with higher expansion potentials are encountered or imported to the site, the recommendations contained herein may require modification.

6.4.3 Settlement

Under normal static conditions, the foundation system should be designed to tolerate a total settlement of 1 inch and a differential settlement of 1/2-inch over 30 feet. These estimated magnitudes of settlement should be considered by the structural engineer in design of the proposed structures at the site.

6.4.4 Allowable Bearing Value

Foundations for the basement may utilize a bearing value of 2,100 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 230 psf and 650 psf for each additional foot in width and depth, respectively, up to a maximum value of 3,400 psf. Recommended allowable bearing values include both dead and live loads, and may be increased by one-third for wind and seismic forces.

6.4.5 Lateral Resistance

Provided site grading is performed and that foundations are founded in engineered fill, a passive earth pressure of 240 pounds per square foot per foot of depth (psf/ft) up to a maximum value of 2,000 pounds per square foot (psf) may be used to determine lateral bearing for footings. This value may be increased by one-third when designing for wind and seismic forces. A coefficient of friction of 0.31 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. Footings against property lines should have the above-noted values reduced by 50 percent.

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.

6.4.6 Conventional Spread Foundations and Slabs on Grade

All exterior and interior continuous footings should have a minimum width of 12 inches and minimum embedment of 12 inches below lowest adjacent grade. All continuous footings for habitable structures should be reinforced with a minimum of one No. 4 bar on top and one No. 4 bar on the bottom.

All spread footings used to support columns should have a minimum width of 18 inches and minimum embedment of 12 inches below lowest adjacent grade. All spread footings in habitable structures should be tied in both directions with a grade beam having a minimum depth and width of 12 inches. The grade beams should be reinforced with a minimum of one No. 4 bar on top and one No. 4 bar on the bottom. Reinforcing of the grade beams should hook into the footings.

Interior concrete slabs constructed on grade should be a nominal 4 inches thick and should be reinforced with 6-inch by 6-inch, W4 X W4 reinforcing wire mesh or No. 3 bars spaced 12 inches on center, each way. Care should be taken to ensure the placement of reinforcement at mid-slab height. Slabs on grade in habitable structures should be hooked to the underlying grade beams on a minimum spacing of 24 inches or poured monolithically with the grade beams.

Interior grade beams as required by the WRI method should be provided in both directions at a maximum spacing of 20 feet. Design of the slab in accordance with the WRI method may use an effective PI of 23. This value already accounts for the factors for ground slope and over-consolidation.

All slabs on grade that may have moisture sensitive coverings should be underlain with a minimum of 10-mil moisture vapor retarder conforming to ASTM E 1745, Class A. A minimum of four (4) inches of clean sand having a sand equivalent (SE) of at least 30 should be placed under the membrane. An additional one inch of the sand (SE>30) may be placed over the vapor barrier to aid in the uniform curing of the slab if preferred. This vapor barrier system is anticipated to be suitable for most flooring finishes that can accommodate some vapor emissions. However, this system may emit more than 4 pounds of water per 1000 sq. ft. and therefore, may not be suitable for all flooring finishes. Additional steps should be taken if such vapor emission levels are too high for anticipated flooring finishes.

Prior to placing concrete, the subgrade below all floor slab areas should be moisture-conditioned to achieve a moisture content that is at least 120 percent of the optimum moisture content. This moisture content should be maintained a minimum depth of 12 inches below the bottoms of the slabs.

6.4.7 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.

6.5 RETAINING AND SCREENING WALLS

6.5.1 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.

6.5.2 Allowable Bearing Value and Lateral Resistance

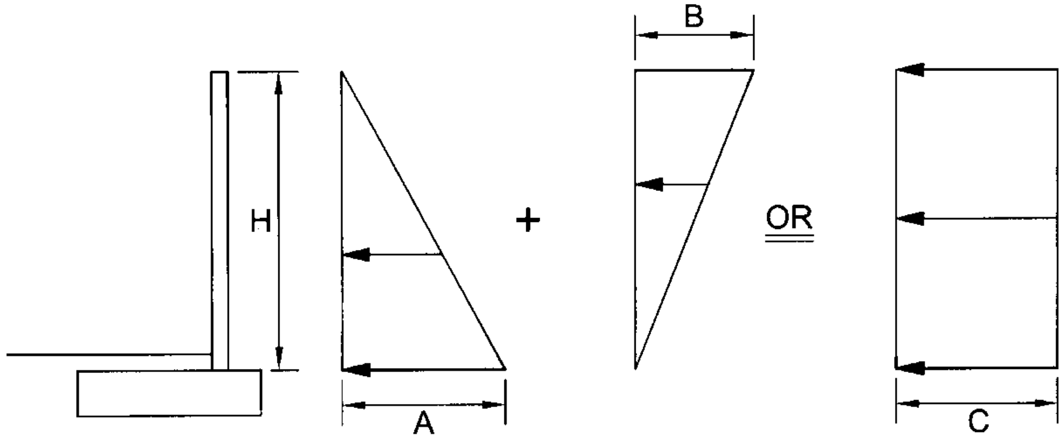
Design of retaining and screen walls may utilize the bearing and lateral resistance values provided in Section 0 and 6.4.5. Lateral resistance for walls along property lines, where lateral removals are restricted should be reduced by 50%.

6.5.3 Active Earth Pressures

Static and seismic earth pressures for level and 2:1 (H:V) backfill conditions are provided in Table 6.2. Seismic earth pressures provided herein are based on the method provided by Seed & Whitman (1970) for active condition and Wood (1973) for at-rest condition, both using a peak ground acceleration (PGA) of 0.38g for probability of exceedance of 10% in 50 years. Active condition relates to the un-restrained retaining wall condition where the wall is free to rotate about its base. The at-rest condition should apply to cases where the wall is restrained from rotation, such as the subterranean walls where the movement is restricted by the structural floor members. 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. In addition, the values are based on drained backfill conditions and do not consider hydrostatic pressure. Furthermore, retaining walls should be designed to support adjacent surcharge loads imposed by other nearby footings or traffic loads in addition to the earth pressure.

TABLE 6.2

SEISMIC EARTH PRESSURES
Pressure Diagram



Static Component | Seismic Component | Total Force

Pressure Values
Walls Up To 10 Feet High

Value	Backfill Condition	
	Level Active (Unrestrained)	Level At-Rest (Restrained)
A	43H	80H
B	12H	12H
C	28H	46H

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.

6.5.4 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.

6.5.5 Footing Reinforcement and Wall Jointing

All continuous footings should be reinforced with a minimum of two No. 4 bars, one top and one bottom. Walls should be provided with cold joints spaced no more than 40 feet apart. Wall finishes and capping materials should not extend across the cold joint. The structural engineer may require different reinforcement or jointing and should dictate if greater than the recommendations provided herein. Where recommended removals are limited due to space restrictions, greater reinforcement and closer jointing may be recommended. Specific recommendations should be provided by the geotechnical consultant during grading based on as-built conditions exposed in the field.

6.5.6 Footing Observations

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

6.5.7 Retaining Wall Backfill

Onsite soils may generally be used for backfill of retaining walls. The project geotechnical consultant should approve all backfill used for retaining walls. Wall 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. Hand-operated compaction equipment should be used to compact the backfill placed immediately adjacent the wall to avoid damage to the wall. Flooding or jetting of backfill material is not recommended.

6.6 EXTERIOR FLATWORK

Exterior flatwork should be a minimum 4 inches thick. Cold joints or saw cuts should be provided at least every 7 feet in each direction. Flatwork more than 7 feet in width across the minimum dimension should be reinforced with 6” by 6”, W4 by W4 welded wire mesh or No 3 bars spaced 12 inches center to center in both directions. Special jointing 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 moistened to at least 120 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 1 percent away from building foundations and retaining walls.

6.7 CONCRETE MIX DESIGN

Laboratory testing of onsite soil indicates **negligible** soluble sulfate content. Concrete designed to follow the procedures provided in ACI 318, Section 4.3, Table 4.3.1 for **negligible** sulfate exposure are anticipated to be adequate for mitigation of sulfate attack on concrete. Upon completion of rough grading, an evaluation of as-graded conditions and further laboratory testing will be required for the site to confirm or modify the conclusions provided in this section.

6.8 CORROSION

Results of preliminary testing of soils for pH, chloride, and minimum resistivity indicate the site is potentially **Moderately 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.

6.9 PRELIMINARY PAVEMENT DESIGN

6.9.1 Subgrade Preparation

Prior to placement of paving elements, subgrade soils should be moisture-conditioned to at least 120 percent of the optimum moisture content then compacted to at least 90 percent compaction for asphaltic concrete pavement areas and to at least 95 percent compaction for concrete pavement areas. Areas observed to pump or yield under vehicle traffic should be removed and replaced with firm and unyielding compacted soil or aggregate base materials.

6.9.2 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 10 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 6.3
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	--	--	12.0 9.0
		--	--	8.0	--
		--	80.0	--	13.0
Parking Stalls	--	3.0	--	--	6.0

6.9.1 Subgrade Preparation

Prior to placement of paving elements, subgrade soils should be scarified 6 inches, moisture-conditioned to at least 120 percent of 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.

6.9.2 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.

6.9.3 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.

6.9.4 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 6.4.

TABLE 6.4
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

6.9.5 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.

6.10 POST GRADING CONSIDERATIONS

6.10.1 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. 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.

6.10.2 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 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.

6.11 PLAN REVIEW AND CONSTRUCTION SERVICES

We recommend ***Albus-Keefe & Associates, Inc*.** be engaged to review any future development plans, including foundation plans 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, described in other literature, 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 **National Community Renaissance** and his 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-KEEFE & ASSOCIATES, INC

Paul Hyun Jin Kim
Associate Engineer
G.E. 3106

REGISTERED PROFESSIONAL ENGINEER
HYUN JIN KIM
No. 3106
GEOTECHNICAL
STATE OF CALIFORNIA

8.0 REFERENCES

Publications

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

CDMG, "Seismic Hazard Zone Report for the Yorba Linda 7.5-Minute Quadrangle, Los Angeles, Orange and San Bernardino County, California," Seismic Hazard Zone Report 010, 2005.

NCEER, "Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils", Technical Report NCEER-97-0022, December 31, 1997.

Seed, HB, and Whitman, RV. "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.

Southern California Earthquake Center (SCEC), University of Southern California, "Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction Hazards in California", March 1999.

U.S. Geologic Survey. Unified Hazard Tool, https://earthquake.usgs.gov/hazards/interactive/

U.S. Geologic Survey. U.S. Seismic Design Maps, https://seismicmaps.org/

Tokimatsu, K. & Seed, H.B., "Evaluation of Settlement in Sands Due to Earthquake Shaking," Journal of Geotechnical Engineering, Vol. 113, No. 8, August, 1987.

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

Plans

Conceptual Site Plan, Placentia, California, prepared by rrm design group Architect, dated September 05, 2019, scale: 1" = 30'

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

GEOTECHNICAL MAP

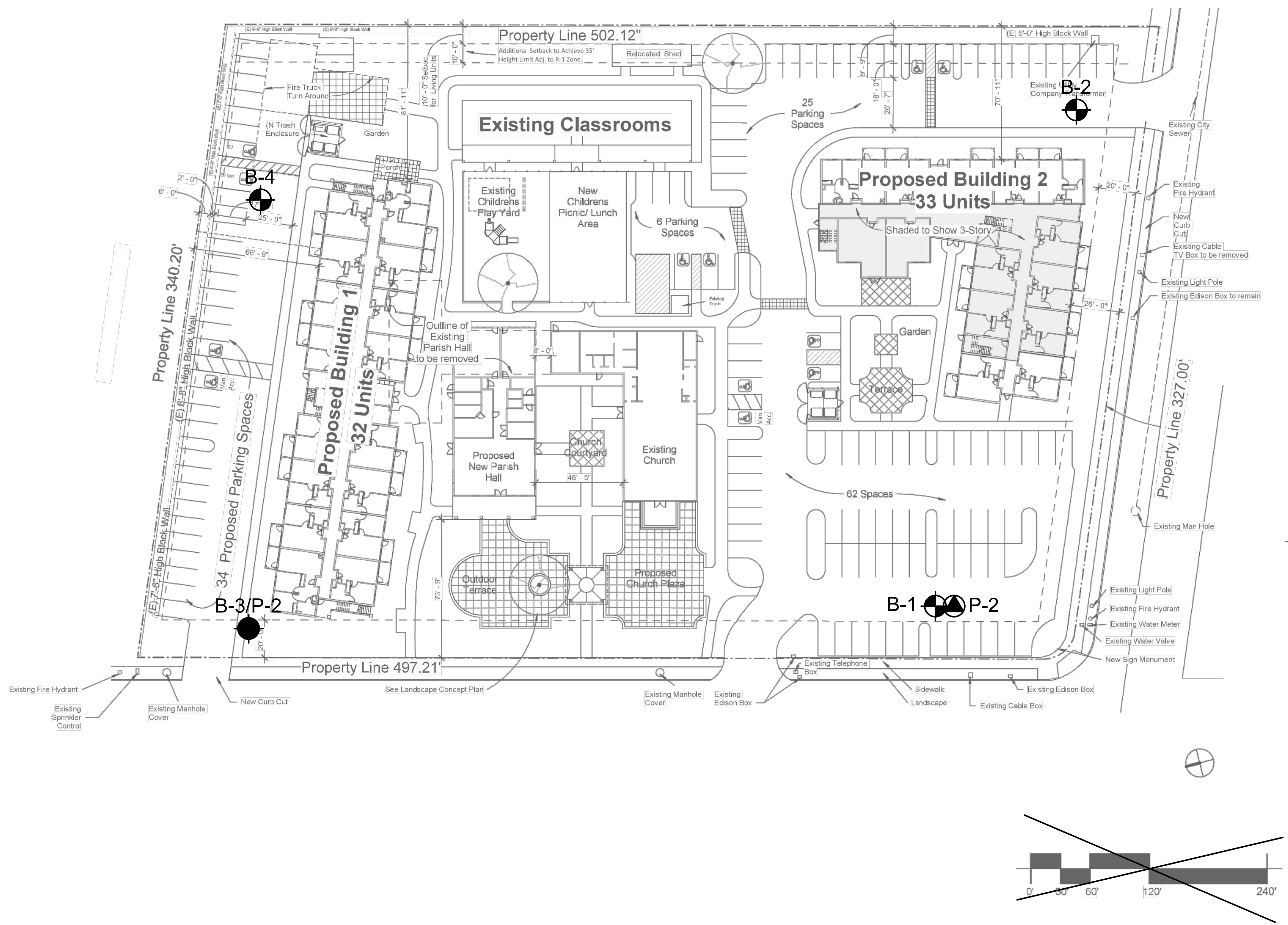
Job No.: 2859.00	Date: 12/27/19	Plate: 1

EXPLANATION
(Locations Approximate)

- Exploratory Boring

- Exploratory Percolation Test Boring

- Exploratory Boring and Percolation Test Boring



Site Coverage

Name	Area	Percentage
Lot Area (SF) :		169,716 SF/ 3.90 acres
Maximum Lot Coverage allowed:		60% (101,830 SF)
Proposed Lot Coverage:		55%
Building Footprints (Existing and Proposed)		35,631 SF
Parking and Driveways		53,824 SF
Covered Patios		3,678 SF
Total Proposed Lot Coverage		93,133 SF (55%)
Percentage Open Space Required:		40%
Percentage Open Space Provided:		45%

Residential Unit Count

	One Bedroom	Two Bedroom
Building 1	28	4
Building 2	31	2
	59 units	6 units

Total Residential Units: 65

rrm design group

SITE PLAN

1314 N. ANGELINA DR. - PLACENTIA, CA

09/05/19
#0934-02-RS17

A4

APPENDIX A

EXPLORATION BORING LOGS

EXPLORATION LOG

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

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests

EXPLANATION

Solid lines separate geologic units and/or material types.

Dashed lines indicate unknown depth of geologic unit change or material type change.

5

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.

Other Laboratory Tests:

15

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

Albus-Keefe & Associates, Inc.

Plate A-1

EXPLORATION LOG

Project: Santa Angelina Senior Community		Location: B-1
Address: 1314 N Angelina Dr, Placentia, CA		Elevation: 294
Job Number: 2859.00	Client: National Community Renaissance	Date: 12/17/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: DDA

Depth (feet)	Material Description	Water	Blows Per Foot	Core	Bulk	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
	Asphalt = 3.5"							
	Base = 5"							
	ARTIFICIAL FILL (Af) Sandy Clay (CL): Grayish brown, moist, very stiff, fine to medium grained sand		25			15.9	113.2	
5	**ALLUVIUM (Qal)** Sandy Clay (CL): Reddish brown, moist, very stiff, fine to medium grained sand, more sand		34			14.4	115	Consol
	Clayey Sand (SC): Reddish brown, moist, medium dense, fine to coarse grained sand, trace pinhole pores		28			12.7	119.3	
10	@ 10 ft, trace pinhole pores		21			12.8	117.3	
15	Sand (SP): Reddish brown, moist, medium dense, fine to medium grained sand		10					
	Clayey Sand (SC): Reddish brown, moist, medium dense, fine to medium grained sand							
20	Sandy Clay (CL): Reddish brown, moist, hard, fine grained sand		28					

Albus-Keefe & Associates, Inc.

EXPLORATION LOG

Project: Santa Angelina Senior Community		Location: B-1
Address: 1314 N Angelina Dr, Placentia, CA		Elevation: 294
Job Number: 2859.00	Client: National Community Renaissance	Date: 12/17/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: DDA

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
		Silty Clay with Sand (CL): Light reddish brown, moist, hard, fine grained sand		19					
30		Silty Sand trace Clay (SM): Light reddish brown, moist, medium dense, fine grained sand		12					200
35		Clayey Sand (SC): Light reddish brown, moist, medium dense, fine to medium grained sand		10					SA Hydro
		Sand with Silt (SP): Light reddish brown, moist, medium dense, fine to medium grained sand							
40		Silty Sand trace Clay (SM): Light reddish brown, moist, dense, fine grained sand		16					200
45		Sandy Clay (CL): Reddish brown, moist, hard, fine grained sand		20					

EXPLORATION LOG

Project: Santa Angelina Senior Community		Location: B-1
Address: 1314 N Angelina Dr, Placentia, CA		Elevation: 294
Job Number: 2859.00	Client: National Community Renaissance	Date: 12/17/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: DDA

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
		Clayey Sand (SC): Light reddish brown, moist, very dense, fine to coarse grained sand		28					
		Total Depth 51.5 feet No Groundwater Boring backfilled with soil cuttings Percolation Well (10ft offset): 0-30' solid 3" pipe 30-35' perforated 3" pipe caved to 25', no gravel added							

Albus-Keefe & Associates, Inc.

EXPLORATION LOG

Project: Santa Angelina Senior Community		Location: B-2
Address: 1314 N Angelina Dr, Placentia, CA		Elevation: 296
Job Number: 2859.00	Client: National Community Renaissance	Date: 12/17/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: DDA

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
		Grass							
		ALLUVIUM (Qal) Sandy Clay (CL): Light reddish brown, dry to damp, hard, fine grained sand, trace pinhole pores and fine roots		58			5.7	115.1	
5		@ 4 ft, some medium grained sand, trace pinhole pores and fine roots		38			10.1	120	Consol
		Silty Sand with Clay (SM): Light reddish brown, moist, medium dense, fine to medium grained sand, some coarse grained sand, trace pinhole pores		20			7.3	110.6	Consol
10		Silty Clay with Sand (CL-ML): Light reddish brown to reddish brown, moist, very stiff, fine grained sand, trace pinhole pores		28			14.8	109.1	
15		Silty Clay (CL-ML): Light reddish brown to light gray, moist, stiff		8					
20				11					

EXPLORATION LOG

Project: Santa Angelina Senior Community		Location: B-2
Address: 1314 N Angelina Dr, Placentia, CA		Elevation: 296
Job Number: 2859.00	Client: National Community Renaissance	Date: 12/17/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: DDA

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
				10					
30		Sandy Clay (CL): Reddish brown, moist, very stiff, fine grained sand		8					
		Total Depth 31.5 feet No Groundwater Boring backfilled with soil cuttings							

EXPLORATION LOG

Project: Santa Angelina Senior Community		Location: B-3
Address: 1314 N Angelina Dr, Placentia, CA		Elevation: 297
Job Number: 2859.00	Client: National Community Renaissance	Date: 12/17/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: DDA

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
		Grass							
		ALLUVIUM (Qal) Sandy Clay (CL): Light reddish brown, dry to damp, very stiff, fine grained sand, trace pinhole pores		38			10	112.1	
		@ 4 ft, moist, hard		74			11.1	119.4	
5									
		@ 6 ft, Gray to reddish brown, very stiff, less sand		32			14.4	117	
10		@ 10 ft, hard, less gray, more sand		37			14.3	113.6	
15		@ 15 ft, very stiff		10					
20				14					

Albus-Keefe & Associates, Inc.

EXPLORATION LOG

Project: Santa Angelina Senior Community		Location: B-3
Address: 1314 N Angelina Dr, Placentia, CA		Elevation: 297
Job Number: 2859.00	Client: National Community Renaissance	Date: 12/17/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: DDA

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, hard, more sand		17					
30		Silty Sand / Sandy Silty trace Clay (SM/ML): Light reddish brown, moist, medium dense / very stiff		8					200
35		Silty Sand trace Clay (SM): Light reddish brown, moist, very stiff		13					200
		Total Depth 36.5 feet No Groundwater Boring backfilled with soil cuttings Percolation Well: 0-30' solid 3" pipe 30-35' perforated 3" pipe caved to 27', no gravel added							

Albus-Keefe & Associates, Inc.

EXPLORATION LOG

Project: Santa Angelina Senior Community		Location: B-4
Address: 1314 N Angelina Dr, Placentia, CA		Elevation: 297
Job Number: 2859.00	Client: National Community Renaissance	Date: 12/17/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: DDA

Depth (feet)	Material Description	Water	Blows Per Foot	Core	Bulk	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
0	Grass				X			Max EI SO4 DS ATT pH Resist Ch
	ALLUVIUM (Qal) Sandy Clay with Silt (CL): Reddish brown, damp to moist, stiff, fine grained sand, trace pinhole pores and fine roots		16	X	X	10.6	103.2	
	@ 4 ft, hard		41	X	X	10.3	114.5	Consol
5	Clayey Sand (SC): Light reddish brown, moist, dense, fine to medium grained sand							
	Sandy Clay with Silt (CL): Reddish brown, moist, very stiff, fine grained sand, trace pinhole pores		35	X		19.9	103.7	
10	@ 10 ft, trace pinhole pores		29	X		22.2	98	
15	Silty Clay trace Sand (CL): Light reddish brown to light gray, damp, very stiff, fine grained sand		13	X				
20	Silty Sand / Sandy Silt trace Clay (SM/ML): Light reddish brown, damp, medium dense / very stiff, fine grained sand		15	X				

EXPLORATION LOG

Project: Santa Angelina Senior Community		Location: B-4
Address: 1314 N Angelina Dr, Placentia, CA		Elevation: 297
Job Number: 2859.00	Client: National Community Renaissance	Date: 12/17/2019
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: DDA

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Core	Bulk	Laboratory Tests: Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
		Silty Sand trace Clay (SM): Light reddish brown, damp, medium dense, fine grained sand		11					
30		Sand (SP): Light reddish brown, damp, very dense, fine to coarse grained sand		32					
		Total Depth 31.5 feet No Groundwater Boring backfilled with soil cuttings							

APPENDIX B

LABORATORY TEST PROGRAM

LABORATORY TESTING PROGRAM

Soil Classification

Soils encountered within the exploratory borings were initially classified in the field in general accordance with the visual-manual procedures of the Unified Soil Classification System (ASTM D2488). The samples were re-examined in the laboratory and classifications reviewed and then revised where appropriate. The assigned group symbols are presented in the Boring Logs provided in Appendix A.

In Situ Moisture and Density

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

Laboratory Maximum Dry Density

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

Expansion Potential

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

Grain-Size Analyses

Grain size analyses were performed on selected samples of site materials. These tests were performed in accordance with ASTM D 422. Results are graphically presented on Plate B.

Consolidation

Consolidation tests were performed for selected soil samples 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 test samples were inundated at selected loads to evaluate the effects of a sudden increase in moisture content (hydro-consolidation potential). Results of the tests are graphically presented on Plates B-2 to B-5.

Direct Shear

The Coulomb shear strength parameters, angle of internal friction and cohesion, were determined for a bulk sample obtained from one our borings. The tests were performed in general conformance with Test Method ASTM D 3080. The sample was remolded to 90 percent of maximum dry density and at the optimum moisture content. Three specimens were prepared for each test, artificially saturated, and then sheared under varied loads at an appropriate constant rate of strain. Results are graphically presented on Plate B-6.

Atterberg Limits

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

Corrosion

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

Soluble Sulfate Content

A chemical analysis was performed on a selected soil sample to determine soluble sulfate content. The test was performed in accordance with California Test Method (CTM) 417. The test result is included in Table B.

Percent Passing No. 200 Sieve

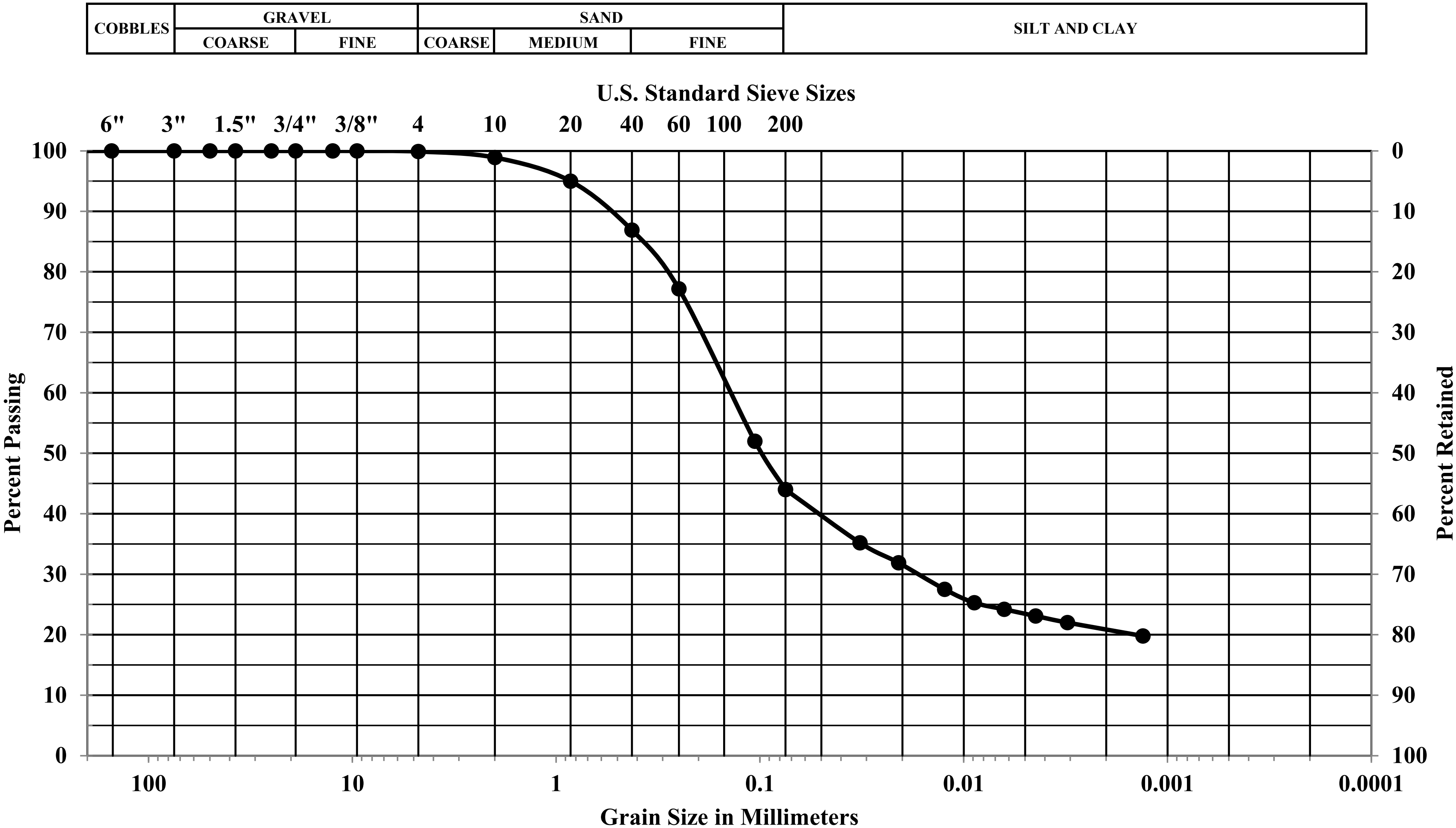
Percent of material passing the No. 200 sieve was determined on selected samples to verify visual classifications performed in the field. These tests were performed in accordance with ASTM D 1140. Test results are presented on Table B.

TABLE B
SUMMARY OF LABORATORY TEST RESULTS

Boring Number	Depth (feet)	Soil Type	Test Results	
B-1	30	Silty Sand (SM)	Percent Passing #200 Sieve:	45.3%
B-1	40	Silty Sand (SM)	Percent Passing #200 Sieve:	30.5%
B-3	30	Silty Sand/ Sandy Silt (SM/ML)	Percent Passing #200 Sieve:	53.7%
B-3	35	Silty Sand (SM)	Percent Passing #200 Sieve:	33.2 %
B-4	0-5	Sandy Clay (CL)	Maximum Dry Density (pcf): Optimum Moisture (%): Liquid Limit: Plastic Index: Soluble Sulfate Content (%): Sulfate Exposure: pH: Minimum Resistivity: Chloride: Expansion Index: Expansion Potential:	122.5 11.5 32 16 0.000 Negligible 7.36 2500 Ohm-cm 24.2 ppm 49 Low

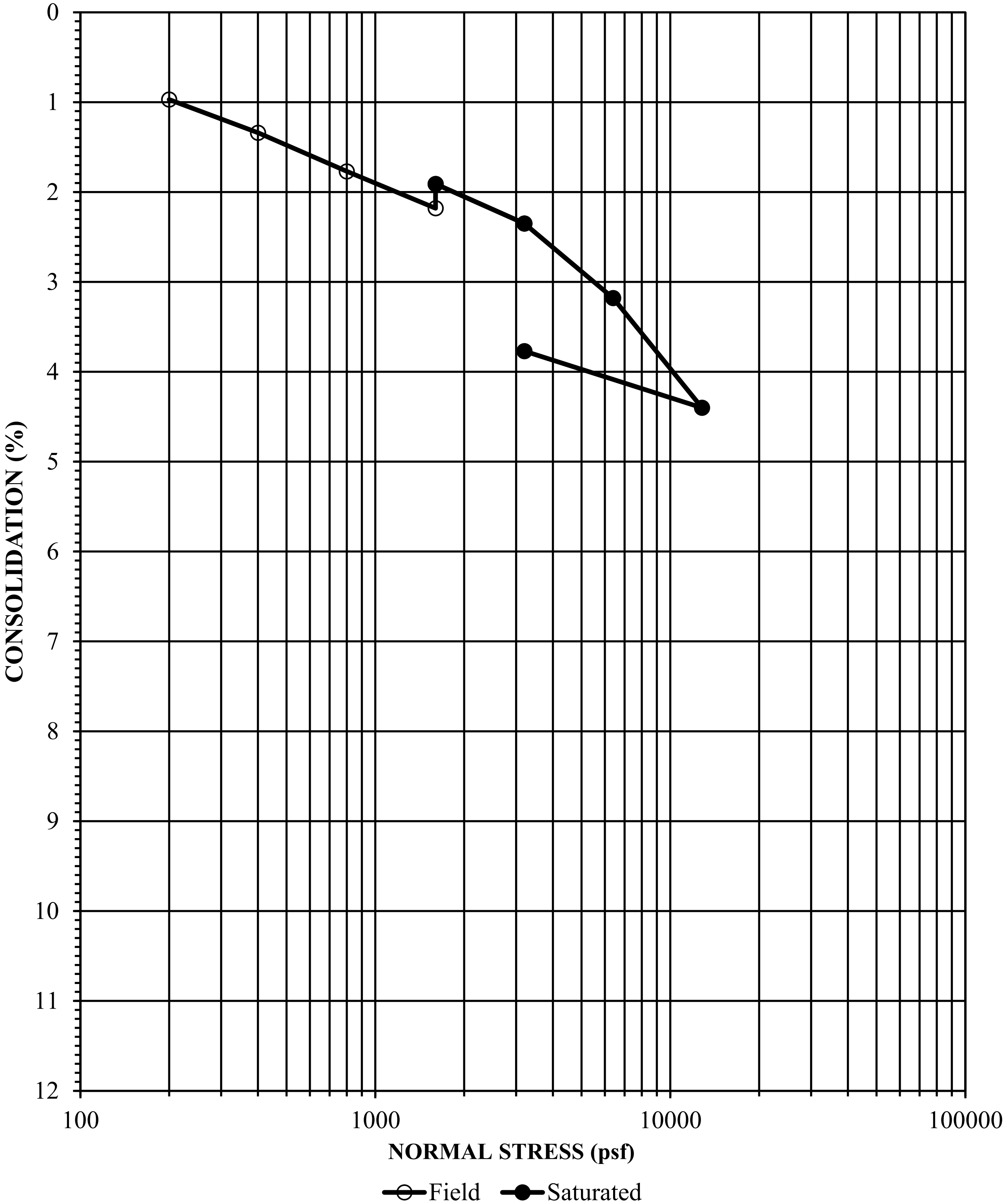
Additional laboratory test results are provided on the boring logs provided in Appendix A and on the Plates that follow.

GRAIN SIZE DISTRIBUTION



Job Number	Location	Depth	Description
2859.00	B-1	35-36.2	Clayey Sand (SC)

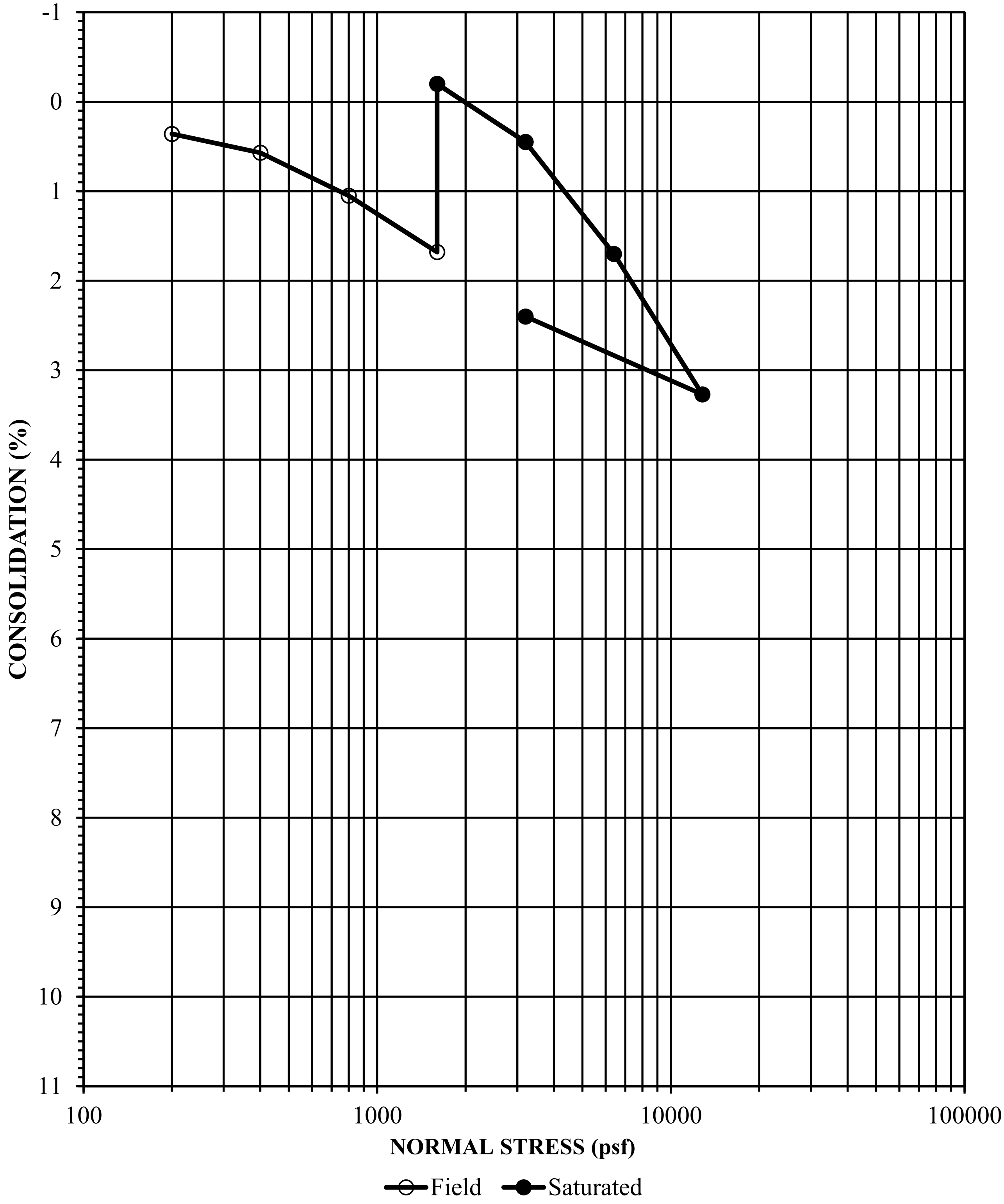
CONSOLIDATION



Job Number	Location	Depth	Description
2859.00	B-1	4	Sandy Clay (CL)

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
117.9	11.2	12

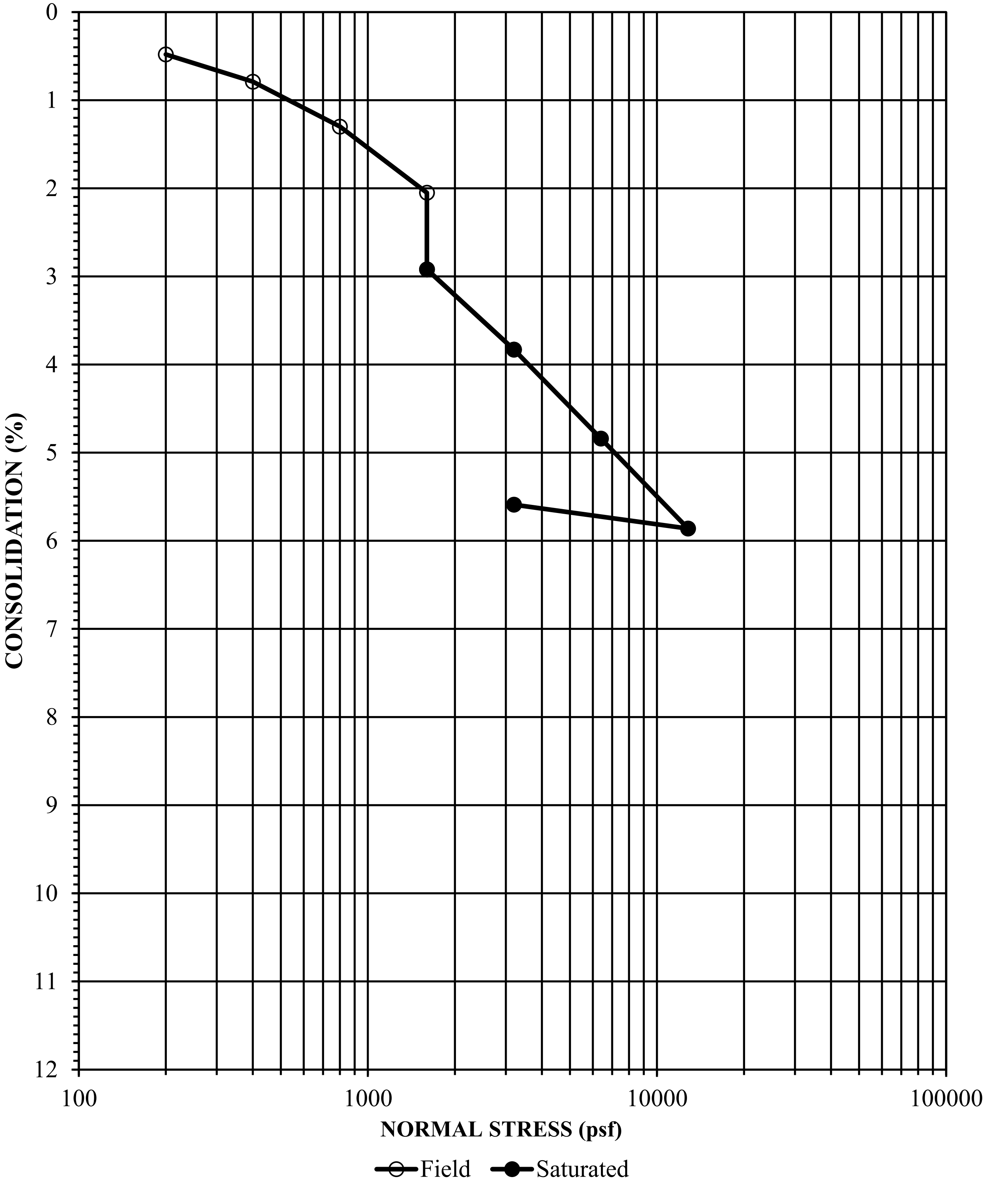
CONSOLIDATION



Job Number	Location	Depth	Description
2859.00	B-2	4	Sandy Clay (CL)

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
118.7	7.7	12.7

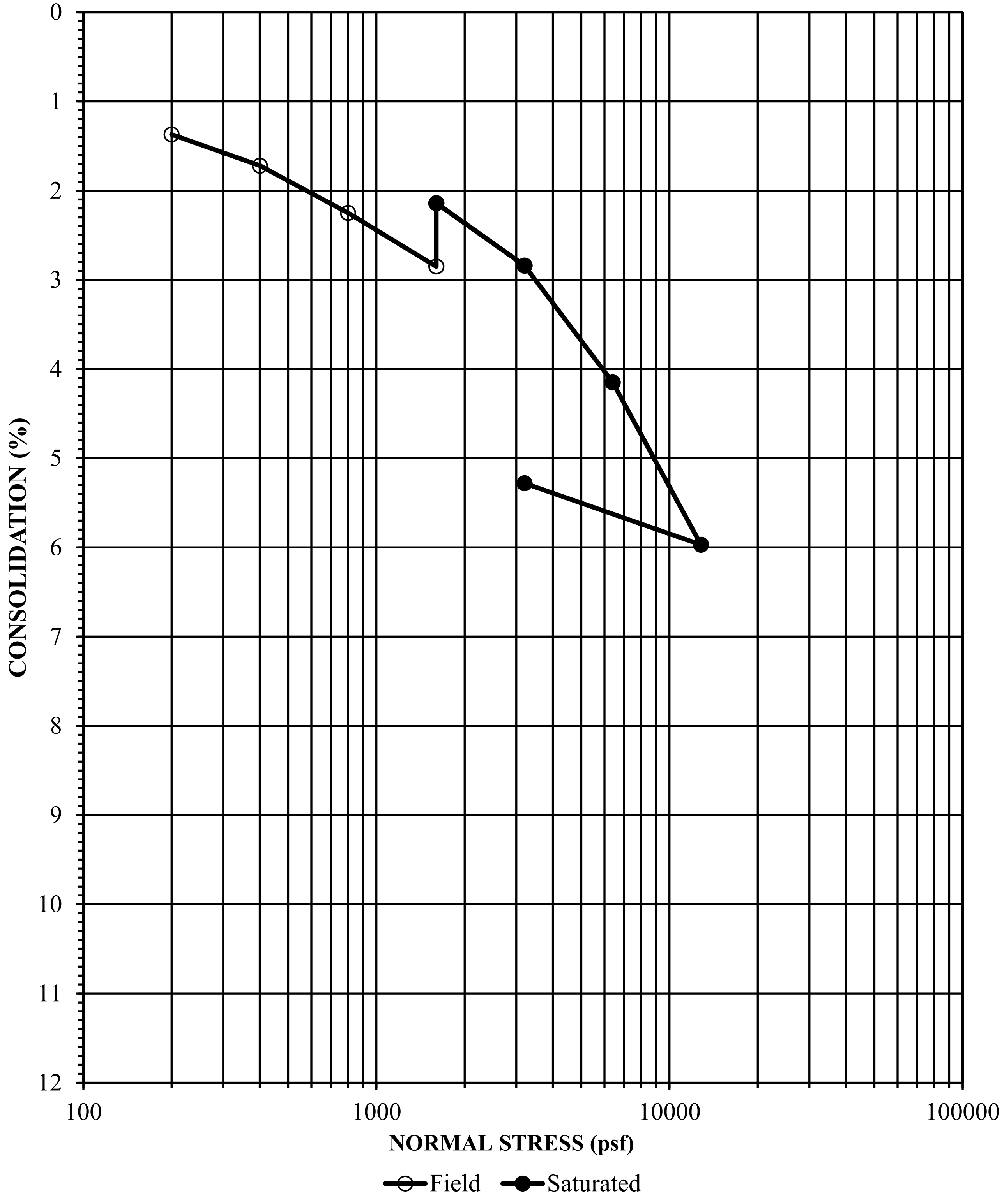
CONSOLIDATION



Job Number	Location	Depth	Description
2859.00	B-2	6	Silty Sand trace Clay (SM)

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
110	8.1	14.1

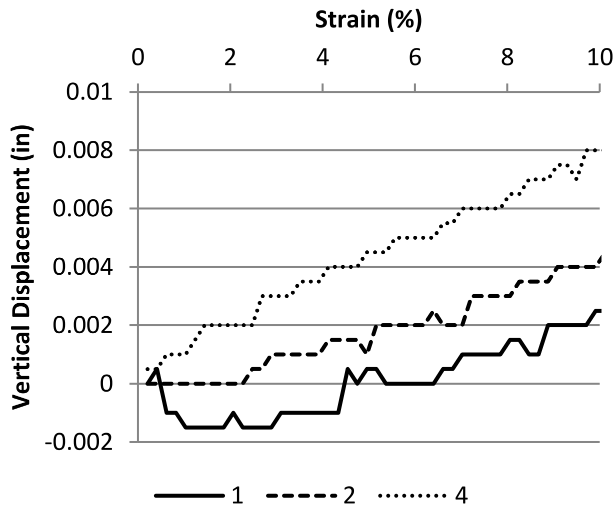
CONSOLIDATION

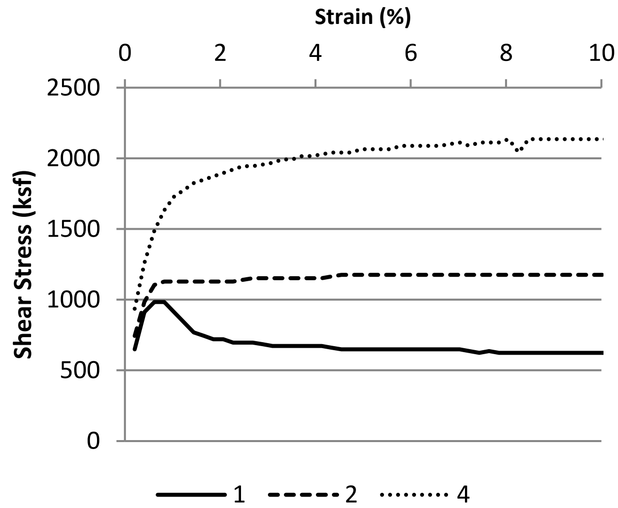


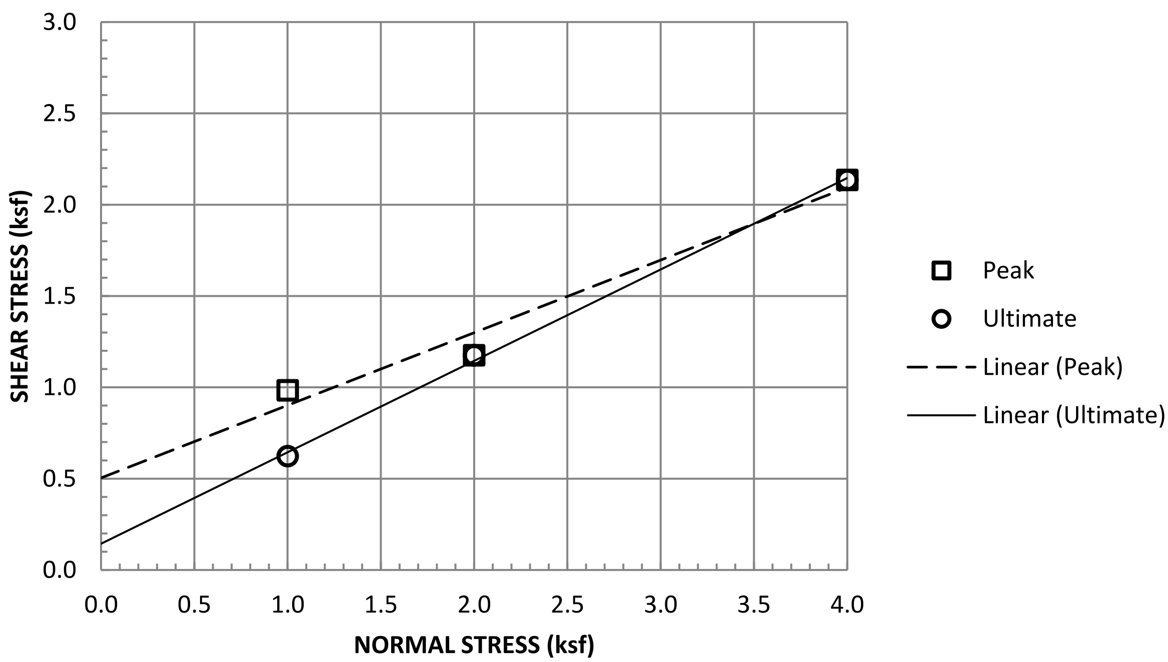
Job Number	Location	Depth	Description
2859.00	B-4	4	Sandy Clay (CL)

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
112	7.8	15.6

DIRECT SHEAR







Sample Type:	Remolded 90% of 122.5 @ 11.5%, Saturate		
Normal Stress (ksf)	1	2	4
Peak Shear Stress (ksf)	0.984	1.176	2.136
Peak Displacement (in)	0.003	0.005	0.008
Ultimate Shear Stress (ksf)	0.624	1.176	2.136
Ultimate Displacement (in)	0.25	0.25	0.25
Initial Dry Density (pcf)	110.3	110.3	110.3
Initial Moisture Content (%)	11.5	11.5	11.5
Final Moisture Content (%)	15.4	16.8	17
Strain Rate (in/min)	.005		

Job Number	Location	Depth	Description
2859.00	B-4	0-5	Sandy Clay (CL)