Appendix D: Geotechnical Investigation, Percolation Study, and Paleontological Records Search Results

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D.1 - Geotechnical Investigation

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ALBUS-KEEFE & ASSOCIATES, INC
GEOTECHNICAL CONSULTANTS

November 9, 2018
J.N.: 2761.00

Mr. Greg McCafferty
McEb LLC
2390 E Orangewood, Suite 510
Anaheim, California 92806

Subject: Highland & Valencia Mixed-Use Project Preliminary Geotechnical Investigation and Percolation Study, 415 S. Highland Ave., Fullerton, California.

Dear Mr. McCafferty,

Pursuant to your request, ***Albus-Keefe & Associates, Inc***. is pleased to present to you our preliminary geotechnical investigation report, for the proposed mixed-use development at the subject site. This report presents the results of our aerial photo, subsurface exploration, laboratory testing, and engineering analyses. Conclusions relevant to the feasibility of the proposed site development are also presented herein based on the findings of our work.

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

Sincerely,

ALBUS-KEEFE & ASSOCIATES, INC.

Patrick M. Keefe
Principal Engineering Geologist

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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 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,
- Review of the site development plans provided to us at the time of our work.
- Preparation of this report

1.2 SITE LOCATION AND DESCRIPTION

The site is located at 415 South Highland Avenue in the City of Fullerton, California. Two properties (032-181-18 and 032-181-20) comprise the site. The site is bounded by West Valencia Drive to the south, South Highland Avenue to the east, a multi-family two-story residential structure to the west, and an alley way followed by a parking lot as well as a residential structure to the north. The location of the site and its relationship to the surrounding areas are shown on Figure 1, Site Location Map.

The site is semi-rectangular in shape and consists of 0.62 acres of land. The site is currently occupied by a car wash facility with an associated asphalt paved surface lot. Minor improvements related to the car wash facility were located west of the existing structure. The remaining portion of the site consist of an asphalt paved lot with limited underground utilities. A landscaped area is located at the southeast portion of the site. The site is also bounded by a masonry-built wall to the northwest.

Topography within the site is relatively flat with elevations approximately 147 to 151 feet above mean sea level (MSL), based on google earth. Site drainage appears to be directed as sheet flow towards the south and east to the adjacent streets. Vegetation within the site consist of grass within the southeast portion of the site and scattered trees near the west, south, and southeast border of the site.

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Site Location Map

McEb LLC
Proposed Mixed Use Development
415 South Highland Avenue
Fullerton, California

NOT TO SCALE

FIGURE 1

1.3 PROPOSED DEVELOPMENT

Based on the plans by IDS Group, the proposed development for the site will consist of 24 two- to three-story townhomes with a three-story commercial building located at the southeast portion of the property.

Improvements will also consist of interior driveways and parking areas, underground utilities, and landscaping. Structural or grading plans regarding the proposed mixed-use development were not provided to us at the time of this report. We anticipate the proposed mixed-use structure will be wood-framed structures with concrete slabs on grade yielding relatively light foundation loads.

2.0 INVESTIGATION

2.1 RESEARCH

We have reviewed the referenced geologic publications, maps, and historical aerial photos of the vicinity. Data from these sources were utilized to the development of some of our findings and conclusions presented in this report. Since 1953, the site appears to have been utilized for agricultural purposes. A structure is present at the southern portion of the property. In 1963, constructed residential and commercial structures are seen adjacent to the site property. In addition, the site is currently a vacant lot due to the residential structure at the southern portion of the site being demolished. In 1972, the site contains a structure on the northeastern portion of the property that appears to be present day car wash. Between 1980 to 1995, a possible structure appears on the south eastern portion of the site. In 2003, the possible structure within the southeastern portion of the site is no longer present, being replaced by landscaping. Between 2003 and 2018, there does not appear to be any major alterations to the site.

2.2 SUBSURFACE EXPLORATION

Subsurface exploration for this investigation was conducted on October 3, 2018. Our exploration consisted of drilling three (3) exploratory borings utilizing a hollow-stem auger drill rig to depths ranging from approximately 21.5 to 51.5 feet below the existing ground surface (bgs). An engineer of *Albus-Keefe & Associates, In*c. logged the exploratory excavations. Visual and tactile identifications were made of the materials encountered, and their descriptions are presented in the Exploration Logs in Appendix A. The approximate locations of the exploratory borings completed by this firm are shown on the enclosed Geotechnical Map, Plate 1. Upon completion of sampling, a 3-inch pipe was installed within exploratory boring B-3 for percolation testing. The boring was later backfilled and the pipe removed after testing. Details and results of percolation tests at the site are the subject of a separate report and are not included in the report in-hand.

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 and capped with asphalt cold patch.

2.3 LABORATORY TESTING

Selected samples obtained from our subsurface exploration were tested in our soil laboratory. Tests consisted of maximum dry density and optimum moisture content, in-situ moisture content and dry density, expansion index, corrosivity (pH. resistivity, and chloride) testing, soluble sulfate content, direct shear, consolidation/collapse potential, grain-size distribution analysis, R-value, percent passing No. 200 sieve, and Atterberg limits. A description of laboratory test criteria and test results are presented in Appendix B.

3.0 SUBSURFACE CONDITIONS

3.1 SOIL CONDITIONS

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

Soil materials encountered at the subject site mainly consisted of interlayered alluvial deposits. Locally undocumented artificial fill was observed within the southern portion and expected to be within the eastern portion of the site. The artificial fill was observed to the depth of 2 feet below existing ground surface. Thicker amounts of artificial fill could possibly be present within the site.

The artificial fill is comprised of medium brown silty sand and sandy silt. These materials are typically slightly damp and loose or medium stiff. Alluvial deposits were encountered below the artificial fill materials to the maximum depth of exploration, 51.5 feet below the ground surface. The alluvial soils are typically comprised of interlayered light, medium, and dark brown sandy clay, clayey sand, sand with clay, and occasional sand layers. Silt and sandy silt deposits were also encountered generally below depths of 20 feet. All materials observed are generally moist and medium dense to dense / stiff to very stiff.

3.2 GROUNDWATER

Groundwater was not encountered during this firm’s subsurface exploration reaching depths of approximately 51.5 feet below the existing ground surface.

A review of the referenced CDMG Seismic Hazard Zone Report 03 indicates that historical high groundwater levels for the general site area have been recorded at approximately 45 feet below the existing ground surface.

3.3 FAULTING

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

TABLE 3.1
Summary of Faults

Name	Dist. (miles)	Slip Rate (mm/yr.)	Preferred Dip (degrees)	Slip Sense	Rupture Top (km)	Fault Length (km)
Puente Hills (Coyote Hills)	0.37	0.7	26	thrust	2.8	17
Elsinore;W+GI	5.56	n/a	81	strike slip	0	83
Elsinore; W+GI+T+J+CM	5.56	n/a	84	strike slip	0	241
Elsinore;W	5.56	2.5	75	strike slip	0	46
Elsinore; W+GI+T	5.56	n/a	84	strike slip	0	124
Elsinore; W+GI+T+J	5.56	n/a	84	strike slip	0	199
Puente Hills (Santa Fe Springs)	6.81	0.7	29	thrust	2.8	11

4.0 ANALYSES

4.1 SEISMICITY

We have performed probabilistic seismic analyses utilizing the U.S. Seismic Design Maps web application by the U.S. Geological Survey (USGS). From our analyses, we obtain a PGA of 0.639 g in accordance with Figure 22-7 of ASCE 7-10. The Site Coefficient, F_{PGA}, for site class D at this range of PGA is 1.0. Therefore, the $PGA_M = 1.0 \times 0.639$ g = 0.64 g. The mean event associated with a probability of exceedance of 2% over 50 years has a moment magnitude of 6.64 and the mean distance to the seismic source is 6.5 miles.

4.2 STATIC SETTLEMENT

Results of our subsurface investigation indicated limited amounts of fill were observed within the site. Visually, the artificial fill was noted to possess variable engineering characteristics with no documentation as to its placement. The artificial fill is not considered suitable for support of engineered fill or foundation loads in its existing state.

The near-surface alluvial soils exhibit slight to moderate compressibility. Provided rough grading and foundation support is designed in accordance with the recommendations provided herein and based on the anticipated foundation loads, total and differential settlements are anticipated to be less than 1 inch and ½ inch over 30 feet, respectively. The estimated magnitudes of settlement are considered within tolerable limits for the proposed structures.

4.3 LIQUEFACTION

We have performed engineering analyses to evaluate the potential for liquefaction at the site if the design earthquake event were to occur. Our analyses followed the guidelines presented in the CGS Special Publication 117A (2008) and the procedures by Youd, et al. (2001). These analyses are based on field test data and laboratory test results from this investigation.

Our liquefaction analyses were based on the soil profile from boring B-1 as provided on Plate C-1 (Appendix C). Historically high groundwater was assumed at a depth of 45 feet below the existing ground surface based on our discussion in Section 3.2. Fine-grained soils that do not have a Plasticity Index (PI) less than 12 and field moisture contents greater than 85% of liquid limit (LL) or soils with corrected blow counts greater than 30 per foot were assumed to be not susceptible to liquefaction. Based on our analysis, we confirmed that a thin layer below depths 45 feet has a factor of safety below 1.3 and as such, is prone to liquefaction during the design earthquake event. Details of the liquefaction analyses are shown on Plate C-2.

Analyses were performed to evaluate the potential magnitude of settlement resulting from seismic shaking of saturated soils with a liquefaction safety factor less than 1.3. The estimated settlement caused by soil liquefaction was evaluated for the site based on the empirical procedures developed by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992), which compare the volumetric strain in the soil with the induced cyclic stress ratios/liquefaction safety factors. Taking the average of these three methods, we estimate a liquefaction-induced settlement of 0.22 inch. Liquefaction induced-settlement analyses are provided in Appendix C on Plate C-3.

In addition to liquefaction settlement, seismic-induced settlement can occur above groundwater table during a strong seismic event. We have estimated the dry seismic settlement using the Tokumatsu and Seed (1987) Method. The analyses indicate a total dry seismic settlement of 1.22 inch. Martin and Lew (1999) recommend that the dry seismic settlement estimate be multiplied by two to account for multi-direction shaking. Therefore, the total estimated dry seismic settlement is 2.45 inches. Details of seismic settlement above groundwater are shown on Plate C-4.

Seismic-induced differential settlement is not expected to exceed one half the total settlement according to Martin and Lew (1999). The differential dry seismic settlement can be less than one half the total dry seismic settlement at sites with relatively uniform soil conditions and deep sediments. We estimate that differential dry seismic settlement of the proposed structure will not exceed 1.2 inch in 30 horizontal feet during the design event.

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 Whittier fault located approximately 5.6 miles. 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 relative 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. Potential ground accelerations have been estimated for the site and are presented in Section 4.1 of this report. 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 due to the relatively flat nature of the site. Furthermore, the site is not located within an area identified by the California Geologic Survey (CGS) as having potential for seismic slope instability.

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 subsurface soils was evaluated by analyzing the potential concurrent occurrence of the above-mentioned three basic factors. The liquefaction evaluation for this site was completed under the guidance of Special Publication 117A: Guidelines for Evaluating and Mitigating Seismic Hazards in California (CDMG, 2008). The site is located within a mapped liquefaction hazard zone by the California Geologic Survey. Historic groundwater is determined to at 45 feet below the existing ground surface.

Our analyses indicate liquefaction could lead to a total seismic settlement (saturated and dry) of the ground surface of up to approximately 2.7 inches due to seismic consolidation during liquefaction. The differential settlement due to seismic settlement would likely be on the order of ½ of the total seismic settlement or approximately 1.4 inch over 30 feet. Evaluations presented in reports for the adjacent sites indicate that lateral spreading is not a significant risk at the site.

Based on the State of California Special Publication 117A, hazards from liquefaction should be mitigated to the extent required to reduce seismic risk to “acceptable levels”. The acceptable level of risk means, “that level that provides reasonable protection of the public safety” [California Code of Regulations Title 14, Section 3721 (a)]. The use of well-reinforced foundations, such as post-tensioned slabs, grade beams with structural slabs, or mat foundations have been proven to adequately provide basal support for similar structures during comparable liquefaction events.

5.3 STATIC SETTLEMENT

Provided rough grading is performed in accordance with the recommendations provided herein and based on the anticipated relatively light foundation loads, total and differential static settlements are anticipated to be less than approximately 1 inch and ½-inch over 30 feet, respectively, for the proposed structures. The estimated magnitudes of static settlements are considered within tolerable limits for the proposed structures. Our office should be provided with foundation plans and structural loads as soon as these become available, in order to confirm our assessment of static settlement.

5.4 EXCAVATION AND MATERIAL CHARACTERISTICS

The earth materials beneath the site are anticipated to be relatively easy to excavate with conventional heavy earthmoving equipment. Generally, the site materials possess moisture contents near or above optimum moisture content. As such, fill soils derived from onsite soils that exhibit elevated moisture contents may require blending or drying prior to compaction.

Buried debris, clarifiers and other underground improvements may be present beneath the site. If encountered during future rough grading, these improvements will require proper abandonment or removal.

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 near-surface earth materials will shrink approximately 5 to 20 percent with an anticipated average near 13 percent. 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 within the site are generally anticipated to possess a **Very Low** expansion potential. There is a possibility of a higher expansion potential due to the interlayered nature of the site. Additional testing for soil expansion will be required subsequent to rough grading and prior to construction of foundations and other concrete work to confirm these conditions.

6.0 RECOMMENDATIONS

6.1 EARTHWORK

6.1.1 General Earthwork and Grading Specifications

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

All existing site improvements, including asphaltic concrete paving, 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.

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.

The presence of the existing offsite improvements may limit removals of unsuitable materials adjacent the property lines. Special grading techniques, such as slot cutting, may be required adjacent to the property lines were offsite structures are nearby.

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, all artificial fill and near-surface compressible alluvium is considered unsuitable for support of proposed engineered fill and site improvements. These materials should be removed from proposed building pads and any other “structural” areas, and replaced as engineered compacted fill. The depth of removal is anticipated to be about 4 feet below existing grades. In addition to general removal of unsuitable soils above, the existing soils should be over-excavated to a depth of at least 2 foot below the bottom of footings for the structure. Locally deeper removal may be required in the areas of previously existing improvements. The actual depth of removal should be determined by the geotechnical consultant during grading.

Within the limits of pavement and free-standing retaining walls over 3 feet in height, the existing fill soils should be removed (approximately 2 feet in thickness) or to a minimum depth of 1 foot below subgrade or footing, whichever is deeper.

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 pavement and free-standing retaining walls may be limited to the edge of the foundations or pavement where lateral restrictions to removals are present such as property lines. The actual depth of removals should be verified by the geotechnical consultant during site grading.

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

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

Following removals/overexcavation, the exposed grade should first be scarified to a depth of 6 inches, brought to at least 110 percent of the optimum moisture content, and then compacted to at least 90 percent of the laboratory standard (ASTM D 1557).

6.1.5 Fill Placement

Materials excavated from the site may be reused as fill provided they are free of deleterious materials and particles greater than 4 inches in maximum dimension (oversized materials). Asphaltic and concrete debris generated during site demolition can be incorporated within 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 110 percent of 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.

Excavations into site materials may expose soils with very differing characteristics. If such differing materials are created through excavation, they should be blended to create a relatively uniform soil mix when reused as fill below the structures. The blending of each lift should be observed and approved by the geotechnical consultant prior to placement of additional lifts of fill.

6.1.6 Import Materials

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

6.1.7 Temporary Excavations

Temporary construction slopes 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 feet in height but no greater than 10 feet 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 SEISMIC DESIGN PARAMETERS

For design of the project in accordance with Chapter 16 of the 2016 CBC, the following table presents the seismic design factors:

TABLE 6.1
2016 CBC Seismic Design Parameters

Parameter	Value
Site Class	D
Importance Factor	I, II, III
Mapped MCE_R Spectral Response Acceleration, short periods, S_S	1.724
Mapped MCE_R Spectral Response Acceleration, at 1-sec. period, S_1	0.625
Site Coefficient, Fa	1.0
Site Coefficient, Fv	1.5
Adjusted MCER Spectral Response Acceleration, short periods, S_{MS}	1.724
Adjusted MCER Spectral Response Acceleration, at 1-sec. period, S_{M1}	0.937
Design Spectral Response Acceleration, short periods, S_{DS}	1.149
Design Spectral Response Acceleration, at 1-sec. period, S_{D1}	0.625
MCE_R = Risk-Targeted Maximum Considered Earthquake	

6.3 FOUNDATION DESIGN

6.3.1 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. Our preliminary recommendations include conventional shallow spread footings and post-tension slabs on grade. 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.

6.3.2 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

Foundations should be designed for static total and differential settlement up to 1 inch and ½-inch over 30 feet, respectively. Seismic settlements could be up to 2.7 inches and 1.4 inch over 30 feet for total and differential settlements, respectively.

6.3.4 Allowable Bearing Value

A bearing value of 1,800 pounds per square foot (psf) can be used for continuous and isolated footings founded at a minimum depth of 12 inches below the lowest adjacent grade and having a minimum width of 12 inches and 24 inches, respectively. The bearing 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 3,500 psf. Recommended allowable bearing values include both dead and live loads, and may be increased by one-third for wind and seismic forces.

6.3.5 Lateral Resistance

Provided site grading is performed in accordance with the recommendations provided by the project geotechnical consultant, a passive earth pressure of 220 pounds per square foot per foot of depth up to a maximum value of 1,100 pounds per square foot may be used to determine lateral bearing for beams. This value may be increased by one-third when designing for wind and seismic forces. A coefficient of friction of 0.31 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.

Where lateral removals may be restricted, such as along property lines, the above-noted values should be reduced by 50%.

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 (ASTM D 1557).

6.3.6 Footings and Slabs on Grade

Exterior and interior continuous 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 four No. 4 bars, two top and two bottom. The structural engineer may require different reinforcement and should dictate if greater than the recommendations provided herein.

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

Interior concrete slabs constructed on grade should be a nominal 4 inches thick and should be reinforced with 6-inch by 6-inch, W4 X W4 reinforcing wire mesh or No. 3 bars spaced 18 inches on center, each way. Care should be taken to ensure the placement of reinforcement at mid-slab height. Slabs on grade should be provided with stiffening beams in accordance with the WRI method. An Effective PI of 20 may be used in design of the slab system. As a minimum, stiffening beams should be provided at a spacing of 15 feet in each direction. The structural engineer may recommend a greater slab thickness and reinforcement based on proposed use and loading conditions and such recommendations should govern if greater than the recommendations presented herein.

Concrete floor slabs in areas to receive carpet, tile, or other moisture sensitive coverings should be underlain with a minimum of 10-mil moisture vapor retarder conforming to ASTM E 1745, Class A. The membrane should be properly lapped, sealed, and underlain with at least 2 inches of sand having a SE no less than 30. One inch of sand may be placed over the membrane to aid in the curing of the concrete and protection of the membrane. 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.

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

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

6.3.7 Post-Tension Slab

Perimeter edge beams should be founded at a minimum depth of 12 inches below the lowest adjacent final ground surface. If a post-tensioned mat is used, the outer 12 inches should be thickened to provide a minimum embedment of 8 inches below lowest grade, or to the depth of the underlying sand, whichever is deeper. Interior beams may be founded at a minimum depth of 12 inches below the tops of the finish floor slabs.

The thickness of the floor slab/mat should be determined by the project structural engineer; however, we recommend a minimum slab thickness of 4 inches. Design of the mat may be based on a modulus of subgrade reaction (Kv1) of 35 pounds per cubic inch (pci). The modulus is based on an effective loading area of 1 foot by 1 foot. The modulus may be adjusted for other effective loading areas using the equation provided below.

$k_b(pci) = 35\left\{\frac{b+1}{2b}\right\}^2$ where "b" is the effective width of loading (minimum dimension) in feet.

All dwelling area floor slabs constructed on-grade should be underlain with a minimum of 10-mil moisture vapor retarder conforming to ASTM E 1745, Class A. The membrane should be properly lapped, sealed, and underlain with at least two (2) inches of sand having a sand equivalent (SE) no less than 30. One inch of this sand may be placed over the membrane to aid in the uniform curing of the concrete slab. 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. Where a mat is utilized, the sand may be reduced to 2 inches provided the mat is at least 8 inches thick.

Prior to placing concrete, subgrade soils below slab-on-grade/mat areas should be thoroughly moistened to provide at least 110 percent of the optimum moisture content to a depth of 12 inches. Based on the guidelines provided in the "Design of Post-Tensioned Slabs-on-Ground" 3rd Edition by Post-Tensioning Institute, the em and ym values for expansive soil conditions are summarized in Table 6.2. These values also consider the estimated potential differential settlement due to seismic settlement discussed previously.

TABLE 6.2
PTI Design Parameters

Parameter	Value
Edge Lift Moisture Variation Distance, em	4.2 feet
Edge Lift, ym	0.946 inches
Center Lift Moisture Variation Distance, em	8.0 feet
Center Lift, ym	0.60 inches

6.3.8 Foundation Observations

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

6.4 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 0 and 6.3.5.

6.4.3 Footing Reinforcing and Wall Jointing

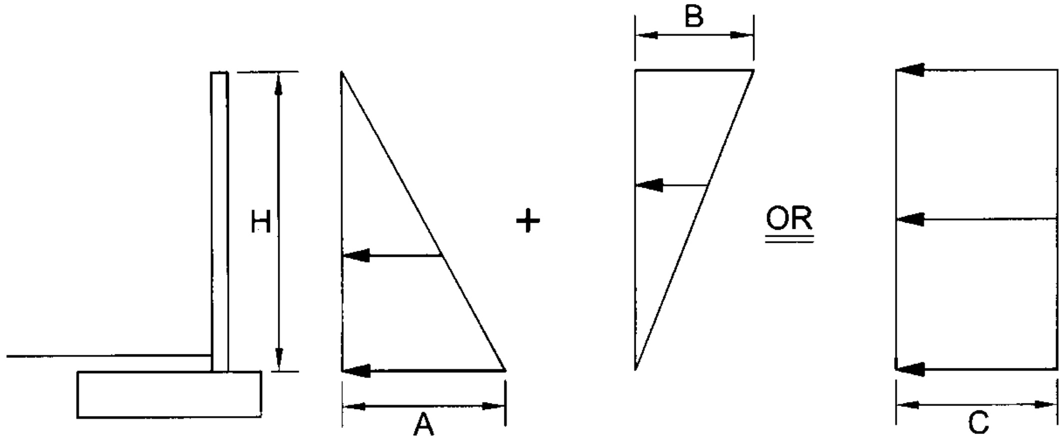
All continuous footings should be reinforced with a minimum of four No. 4 bars, two top and two bottom. The structural engineer may require different reinforcement and should dictate if greater than the recommendations herein.

Retaining and screen walls should be provided with cold joint through the wall stem at a spacing of approximately 20 feet on center. The joint should not continue through the footing.

6.4.4 Active Earth Pressure

Static and seismic earth pressures for level and 2:1 (H:V) backfill conditions are provided in the 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 0.40g. This acceleration is based on a 10% probability of exceedance in 50 years. Based on the 2016 CBC, walls that retain less than 6 feet need not be designed for seismic earth pressures. The values provided in the following table are based on typical site materials on drained backfill conditions and do not consider hydrostatic pressure. 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 6.3
SEISMIC EARTH PRESSURES



Static Component Seismic Component Total Force

Active Earth Pressure Values

<table>
<tr><th rowspan="2">Value</th><th colspan="2">Backfill Condition</th></tr>
<tr><th>Level</th><th>2H:1V Slope</th></tr>
<tr><td>A</td><td>37H</td><td>65H</td></tr>
<tr><td>B</td><td>13H</td><td>13H</td></tr>
<tr><td>C</td><td>25H</td><td>39H</td></tr>
<tr><td colspan="3">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.</td></tr>
</table>

6.4.5 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 cover the top of footing. A drainage panel should be provided between the water proofing and soil backfill. The panel should extend from the top of the subdrain gravel 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.

6.4.6 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.

6.4.7 Retaining Wall Backfill

Onsite soils may be used to backfill 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.

6.5 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. 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 achieve a minimum of 110 percent of 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 or other appropriate collection devices designed to carry runoff water to the street or other approved drainage structures. Flatwork adjacent entry points to structures should have a minimum slope of 1% away from the structure.

6.6 CONCRETE MIX DESIGN

Laboratory testing of near-surface soils for soluble sulfate content indicates soluble sulfate concentration of up to 0.000%. 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 content, 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 buried below ground surface at the site.

6.8 PRELIMINARY PAVEMENT DESIGN

6.8.1 Preliminary Pavement Structural Sections

Based on the soil conditions present at the site and estimated traffic index, preliminary pavement structural sections are recommended in the table below. Considering soil variability at the site, "R-value" of 25 was utilized 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.

6.8.2 Subgrade Preparation

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

TABLE 6.4
PRELIMINARY PAVEMENT STRUCTURAL SECTIONS

Location	Traffic Index	Asphaltic Concrete (inches)	Portland Cement Concrete (inches)	Concrete Pavers (mm)	Aggregate Base (inches)
Entryway and Driveway	5.5	3.0 4.0	-- --	-- --	9.0 6.0
		--	--	80	10.0
		--	6.5	--	--
Parking Stalls	N/A	3.0	--	--	6.0

6.8.3 Aggregate Base

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

6.8.4 Asphaltic Concrete

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

6.8.5 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 and provided with a key or dowels. Where traffic will traverse over edges of concrete paving (not including joints), the edges should be thickened by 20% of the design thickness toward the edge over a horizontal distance of 5 feet.

Trash pickup areas should be provided with a concrete slab where the bins will be picked up and extend at least 3 feet past the front wheel landing areas. The slab should be at least 6.5 inches thick and be reinforced with No. 4 bars spaced at 24 inches on centers, both ways. The slabs should be provided transverse and longitudinal joints spacing as specified above. Dowels or a keyway should be provided at all cold joints.

6.9 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 2016 CBC, Section 1804.3. 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. Trench backfill should be brought to moisture content slightly over optimum, placed in lifts no greater than 12 inches in thickness, and then mechanically compacted with appropriate equipment to at least 90 percent of the laboratory standard. The project geotechnical consultant should perform density testing, along with probing, to test compaction. Site conditions are generally not suitable for jetting of trench backfill and jetting should not be completed without prior approval from the project geotechnical consultant.

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

6.10 PLAN REVIEW AND CONSTRUCTION SERVICES

We recommend *Albus-Keefe & 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 **McEb LLC** and their project consultants in the planning and design of the proposed development. This report has not been prepared for use by parties or projects other than those named or described herein. This report may not contain sufficient information for other parties or other purposes.

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

Respectfully submitted,

ALBUS-KEEFE & ASSOCIATES, INC.

Mark Principe
Staff Engineer

Bidjan Ghahreman
Associate Engineer
G.E. 3111

8.0 REFERENCES

Publications

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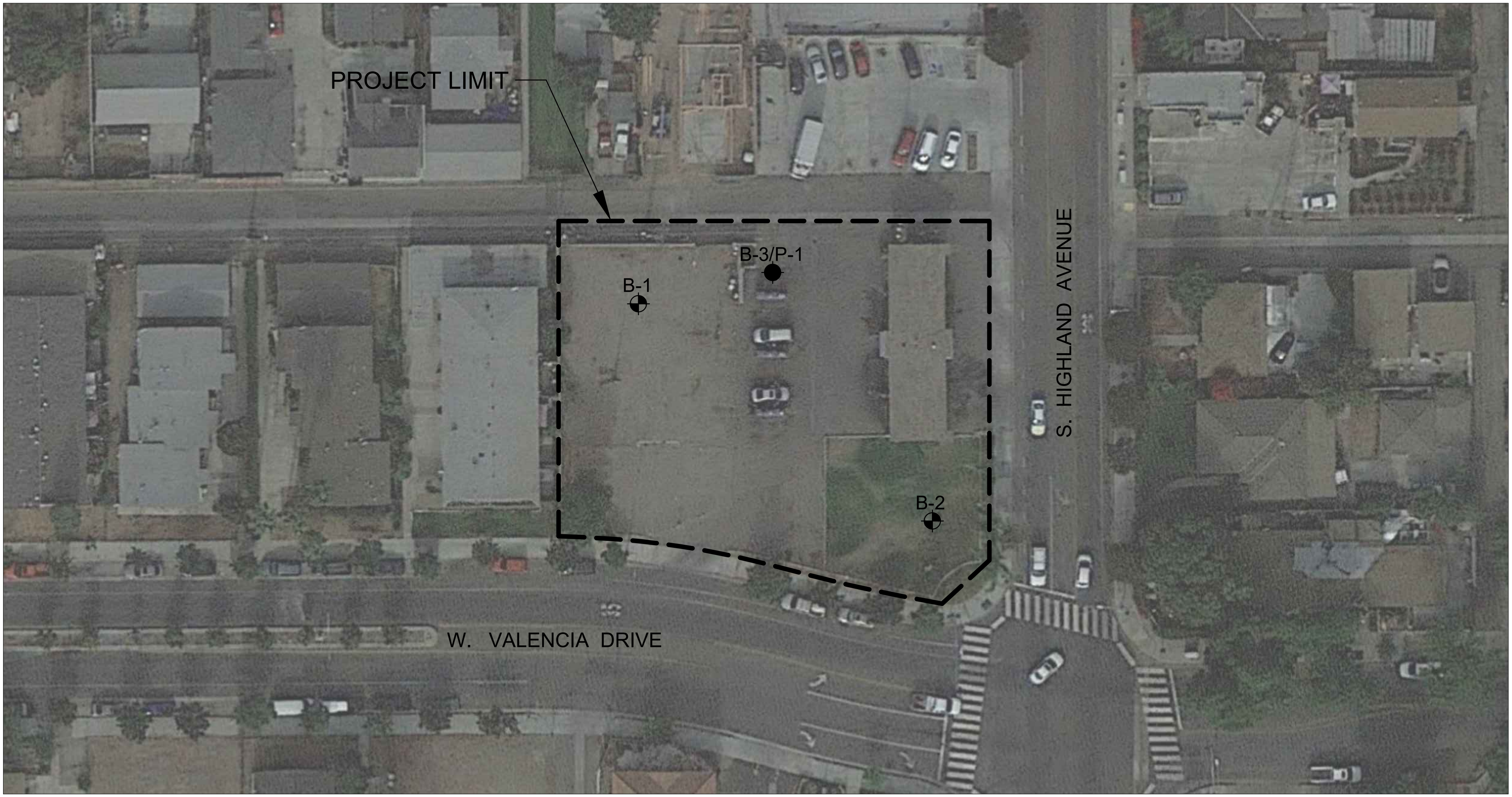
Plans

Highland & Valencia Mixed-Use, 415 S. Highland Ave., Fullerton, CA 92832, prepared by IDS Group, dated May 18, 2018, Project No. 16x056



EXPLANATION
(Locations Approximate)

- Exploratory Boring
- Exploratory Percolation Boring

APPROX. SCALE
1" = 40'

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

GEOTECHNICAL MAP

Job No.: 2761.00	Date: 11/9/18	Plate: 1

APPENDIX A

EXPLORATION LOGS

EXPLORATION LOG

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

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

Albus-Keefe & Associates, Inc.

Plate A-1

EXPLORATION LOG

Project: Highland & Valencia Mixed-Use Project		Location: B-1
Address: 415 S Highland Ave, Fullerton, CA		Elevation: 150.9
Job Number: 2761.00	Client: McEb LLC	Date: 10/3/2018
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: MP

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
0		Asphalt (AC): 4 Inches							
		ALLUVIUM (Qal) Silty Sand (SM): Medium brown, moist, medium dense, fine grained sand, trace medium grained sand, micaceous, few clay.		25			8.6	95.7	DS RVal pH Resist Ch
5		Clayey Sand (SC): Mottled light and medium to dark brown, moist, medium dense, fine grained sand, trace medium grained sand, nodules of clay present.		19			13.3	107.8	SO4 Consol ATT
10		Clayey Sand / Sandy Clay (SC/CL): Dark brown, moist, medium dense / very stiff, fine grained sand.		36			16.3	109.2	
15		Clayey Sand (SC): Medium brown, moist, medium dense, fine grained sand.		13					
20		Sand with Clay (SP-SC): Light brown, moist, medium dense, fine grained sand, lenses of sandy clay.		7					
		Silt (ML): Medium brown, very moist, stiff, few fine grained sand, micaceous, with clay.							

EXPLORATION LOG

Project: Highland & Valencia Mixed-Use Project	Location: B-1
Address: 415 S Highland Ave, Fullerton, CA	Elevation: 150.9
Job Number: 2761.00 / Client: McEb LLC	Date: 10/3/2018
Drill Method: Hollow-Stem Auger / Driving Weight: 140 lbs / 30 in	Logged By: MP

Depth (feet)	Material Description	Water	Samples: Blows Per Foot	Core	Bulk	Laboratory Tests: Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
	Sand (SP): Grayish brown, moist, dense, fine to coarse grained sand.		20					
	Sandy Silt (ML): Medium grayish brown, moist, hard, fine grained sand, iron oxide staining.							
30	Silt (ML): Medium brown, moist, stiff, few fine sand, with clay, lenses of sandy silt / silty sand.		9					
35	Clayey Sand (SC): Grayish brown, moist, medium dense, medium grained sand, lenses of sand.		13					
	Sand (SP): Brown, moist, medium dense, medium to coarse grained sand, trace fines.							
40	Sandy Clay (CL): Light grayish brown, moist, very stiff, lenses of sand, iron oxide staining.		15					
45	Clayey Sand (SC): Brownish gray, moist, dense, fine to medium grained sand.		19					200
	Clay (CL): Brownish gray, moist, hard.							

EXPLORATION LOG

Project: Highland & Valencia Mixed-Use Project		Location: B-1
Address: 415 S Highland Ave, Fullerton, CA		Elevation: 150.9
Job Number: 2761.00	Client: McEb LLC	Date: 10/3/2018
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: MP

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
		@ 50 ft, with fine grained sand.		19			22.1		ATT
		Total Depth 51.5 feet. No Groundwater Encountered. Backfilled with Cuttings. Patched with A.C. Cold Patch.							

EXPLORATION LOG

Project: Highland & Valencia Mixed-Use Project		Location: B-2
Address: 415 S Highland Ave, Fullerton, CA		Elevation: 148.8
Job Number: 2761.00	Client: McEb LLC	Date: 10/3/2018
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: MP

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
		ALLUVIUM (Qal) Silty Sand / Sandy Silt (SM/ML): Medium brown, slightly damp, loose / medium stiff, fine grained sand.							
		Sand with Silt (SP-SM): Tan to brown, moist, loose, fine grained sand, trace medium grained sand.		8			3.2	92.4	
5		Clay (CL): Mottled light, medium, and dark brown, moist, medium stiff, fine grained sand, rootlets present, with sand, with silt.		10			17.6	93.1	Consol
		Clayey Sand (SC): Medium brown, moist, loose, fine grained sand, possible pores, rootlets present.		13			17.8	100.6	
10		Sandy Clay (CL): Medium brown light brown, moist, very stiff, fine to medium grained sand, trace coarse gravel, trace pores.		26			17.7	108.8	
15		Clayey Sand / Sandy Clay (SC/CL): Medium brown light brown, moist, medium dense / very stiff, fine grained sand.		26			16.1	110.5	
20		Sand with Clay (SP-SC): Brown, moist, dense, fine to medium grained sand, trace coarse grained sand.		24					
		Total Depth 21.5 feet. No Groundwater Encountered. Backfilled with Cuttings.							

Albus-Keefe & Associates, Inc.

EXPLORATION LOG

Project: Highland & Valencia Mixed-Use Project		Location: B-3
Address: 415 S Highland Ave, Fullerton, CA		Elevation: 149.8
Job Number: 2761.00	Client: McEb LLC	Date: 10/3/2018
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: MP

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		Asphalt (AC): 4 Inches							
		ALLUVIUM (Qal) Sand with Clay (SP-SC): Light brown, moist, medium dense, fine grained sand. @ 4 ft, Increased clay.							
5		Silty Sand / Clayey Sand (SM/SC): Mottled light brown and medium brown, moist, medium dense, fine to coarse grained sand.		19			13.3	107.8	Consol
		@ 6 ft, Medium brown. @ 8 ft, Increased clay.							
10				27			13.2	111.6	SA Hydro
		Sandy Clay (CL): Medium brown, moist, stiff, fine grained sand, few silt.							
15				9					
		Sand with Silt (SP-SM): Reddish brown, moist, medium dense, lenses of sandy silt.							
20				16					
		Sandy Silt (ML): Medium brown, moist, very stiff, fine grained sand, iron oxide staining, few coarse grained sand, few clay.							

EXPLORATION LOG

Project: Highland & Valencia Mixed-Use Project		Location: B-3
Address: 415 S Highland Ave, Fullerton, CA		Elevation: 149.8
Job Number: 2761.00	Client: McEb LLC	Date: 10/3/2018
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: MP

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Core	Bulk	Laboratory Tests: Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
		Sand (SP): Grayish brown, moist, medium dense, fine to medium grained sand.		19					
		Silt (ML): Medium grayish brown, moist, very stiff, with clay, few fine grained sand.							
30		@ 30 ft, Stiff.		7					
		Sand with Silt (SP-SM): Grayish brown, moist, dense, fine to coarse grained sand.							
35				24					
		Sand (SP): Grayish brown, moist, dense, fine to coarse grained sand.							

Total Depth 36.5 feet.
No Groundwater was Encountered.
Backfilled with Cuttings.
Patched with A.C. Cold Patch.
Installed percolation well P-1 within Boring.

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.

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.

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

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

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 samples were undisturbed or remolded to 90 percent of maximum dry density and 2 percentage points over optimum. Three specimens were prepared for each test, artificially saturated, and then sheared under varied loads at an appropriate constant rate of strain. Results are graphically presented on Plate B-5.

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 specific 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-4.

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.

Particle Size Analyses

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

Corrosion

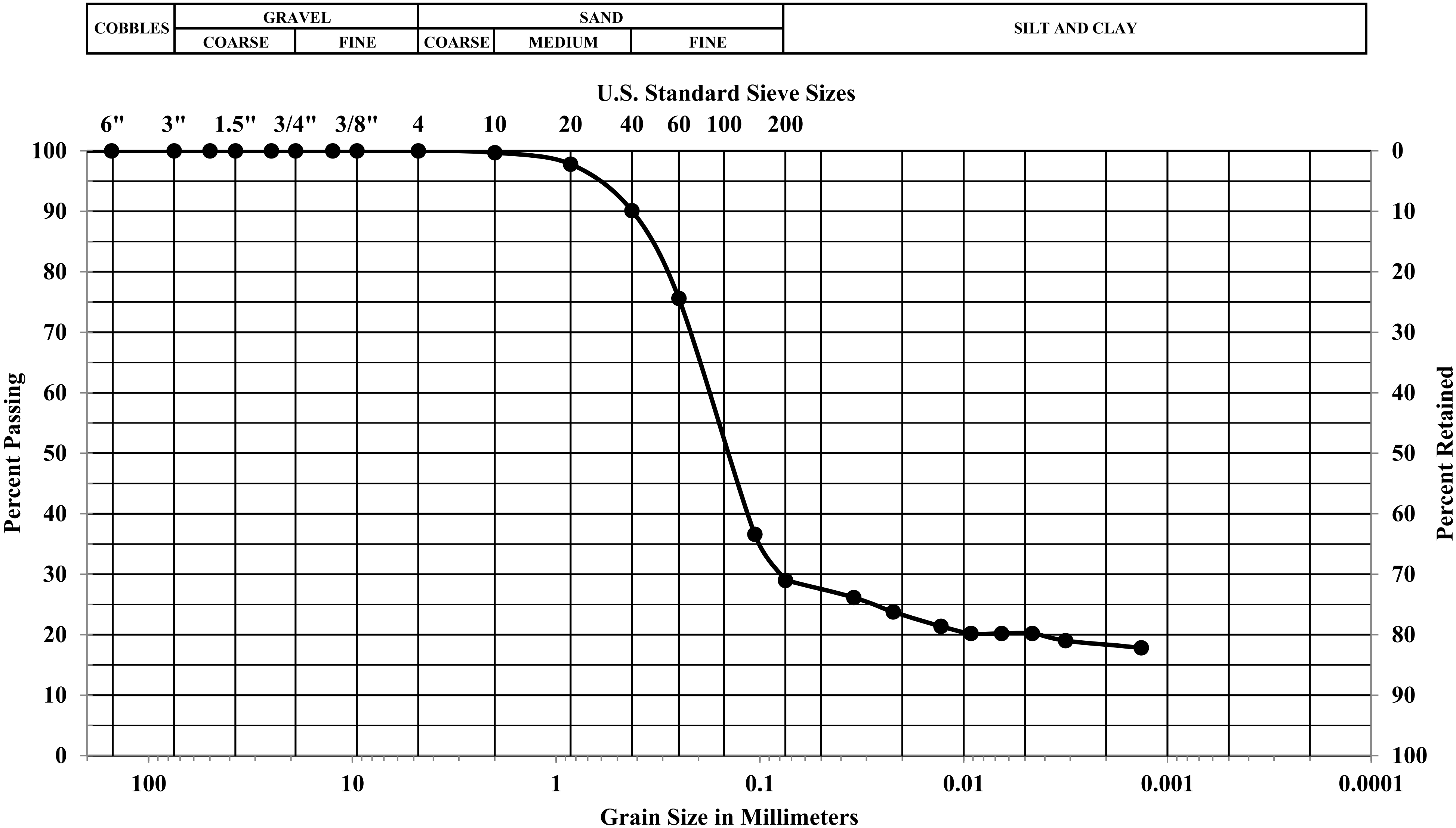
Select samples were tested for minimum resistivity, chloride, and pH in accordance with California Test Method 643. Results of these tests are provided in Table B-1.

TABLE B-1
SUMMARY OF LABORATORY TEST RESULTS

Boring No.	Sample Depth (ft.)	Soil Description	Test Results	
B-1	0-5	Silty Sand (SM)	Maximum Dry Density: Optimum Moisture Content: PH: Resistivity: Chloride: Expansion Index: Expansion Potential: R-Value:	122.0 pcf 12.0 % 7.86 5900 ohm-cm 3.7 ppm 4 Very Low 68
B-1	6	Clayey Sand (SC)	Soluble Sulfate Content: Sulfate exposure: Liquid Limit: Plasticity Index:	0 % Negligible 28 9
B-1	45	Clayey Sand (SC)	Passing No. 200 Sieve:	13.5 %
B-1	50	Clay (CL)	Liquid Limit: Plasticity Index:	31 12

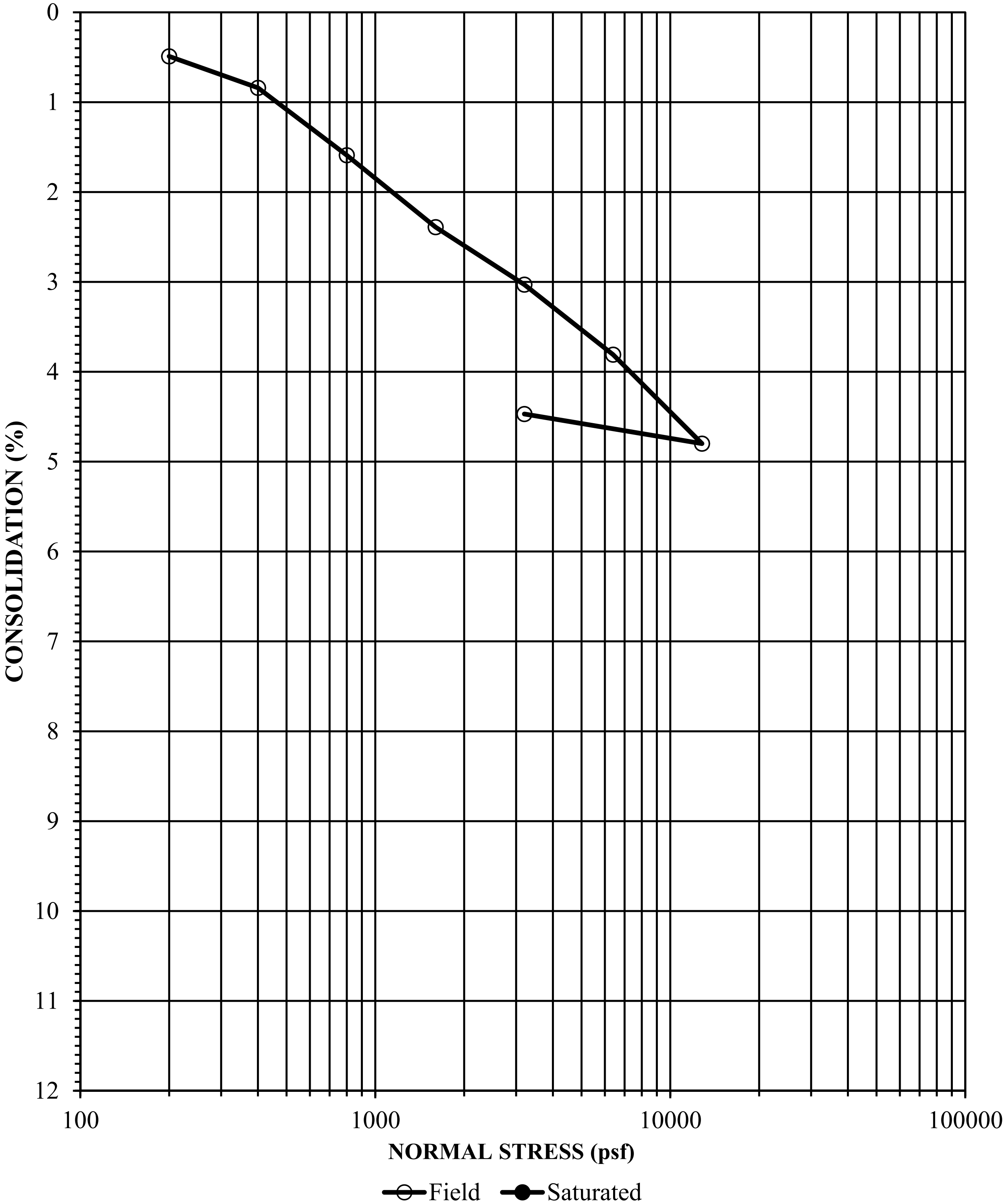
Note: Additional laboratory test results are provided on the boring logs provided in Appendix A.

GRAIN SIZE DISTRIBUTION



Job Number	Location	Depth	Description
2761.00	B-3	10	Silty Sand / Clayey Sand (SM/SC)

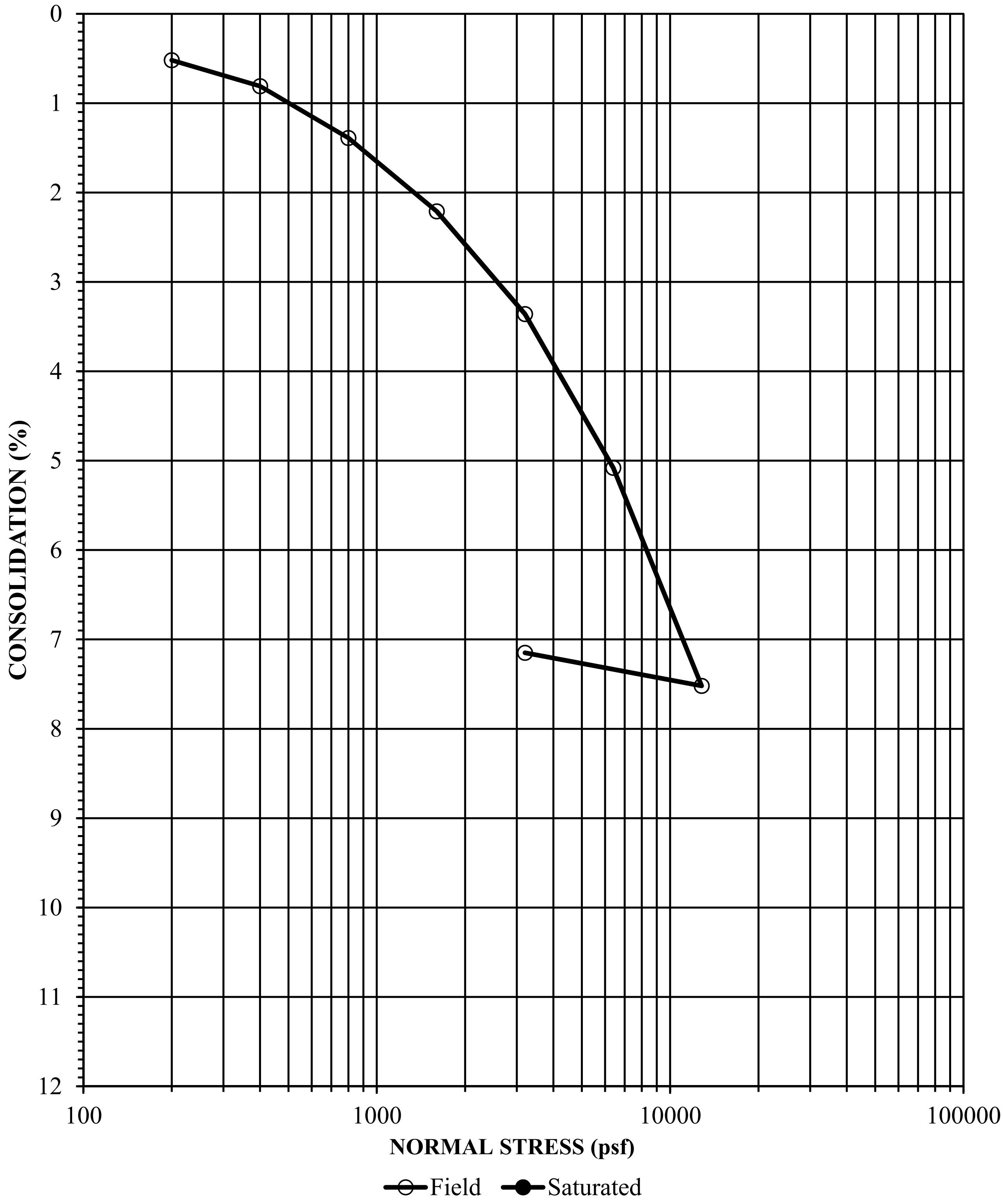
CONSOLIDATION



Job Number	Location	Depth	Description
2761.00	B-1	6	Clayey Sand (SC)

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
107.8	13.9	12.1

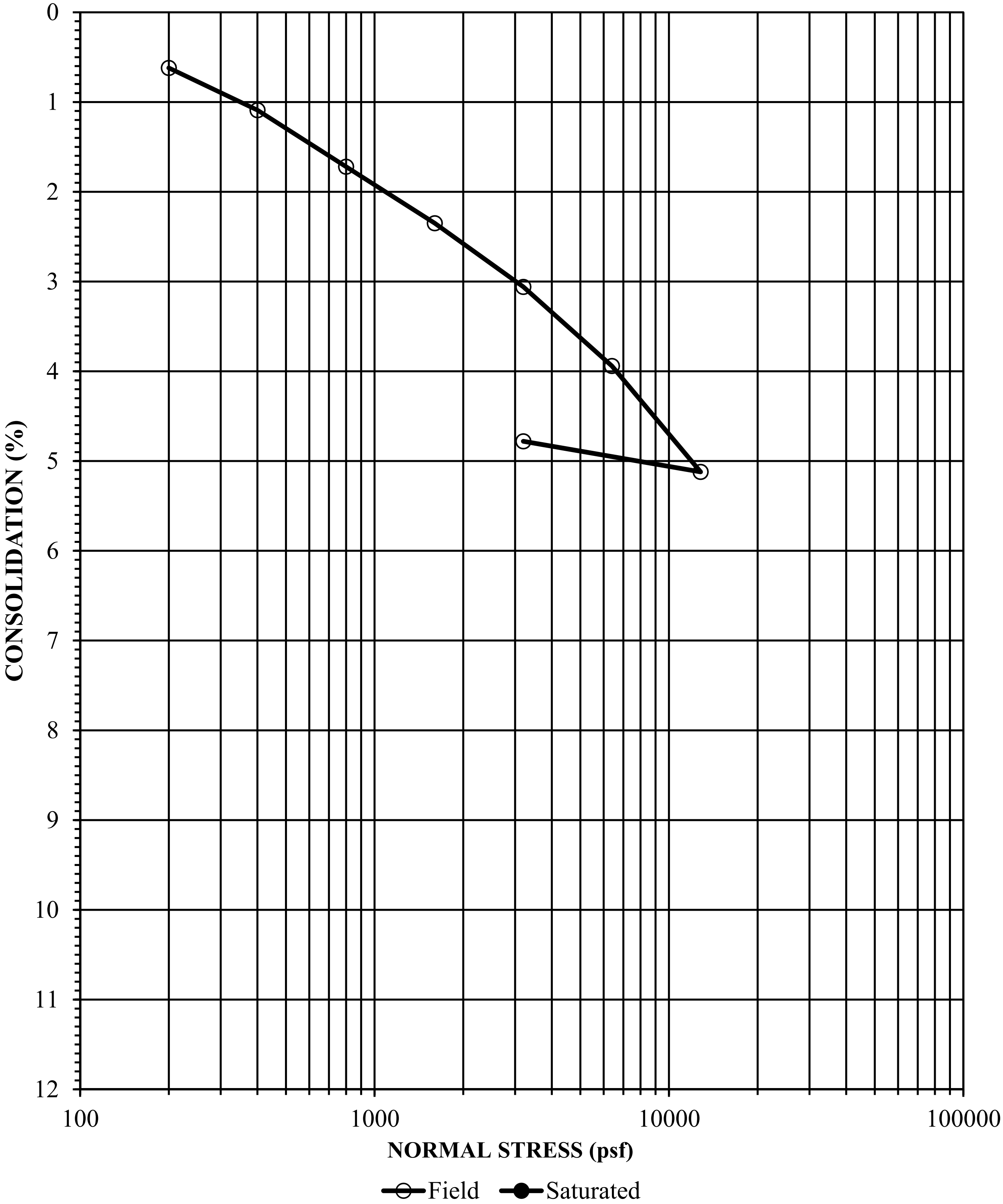
CONSOLIDATION



Job Number	Location	Depth	Description
2761.00	B-2	4	Clay (CL)

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
93	21.2	12.8

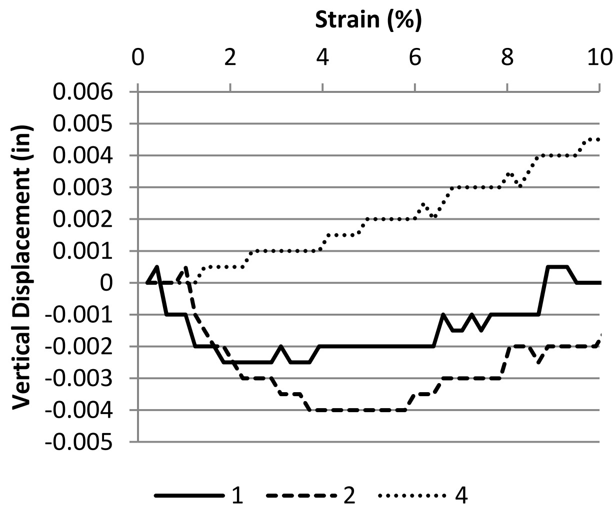
CONSOLIDATION

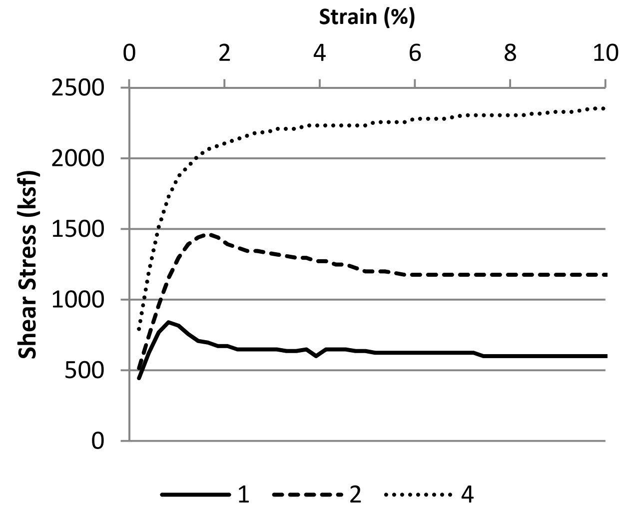


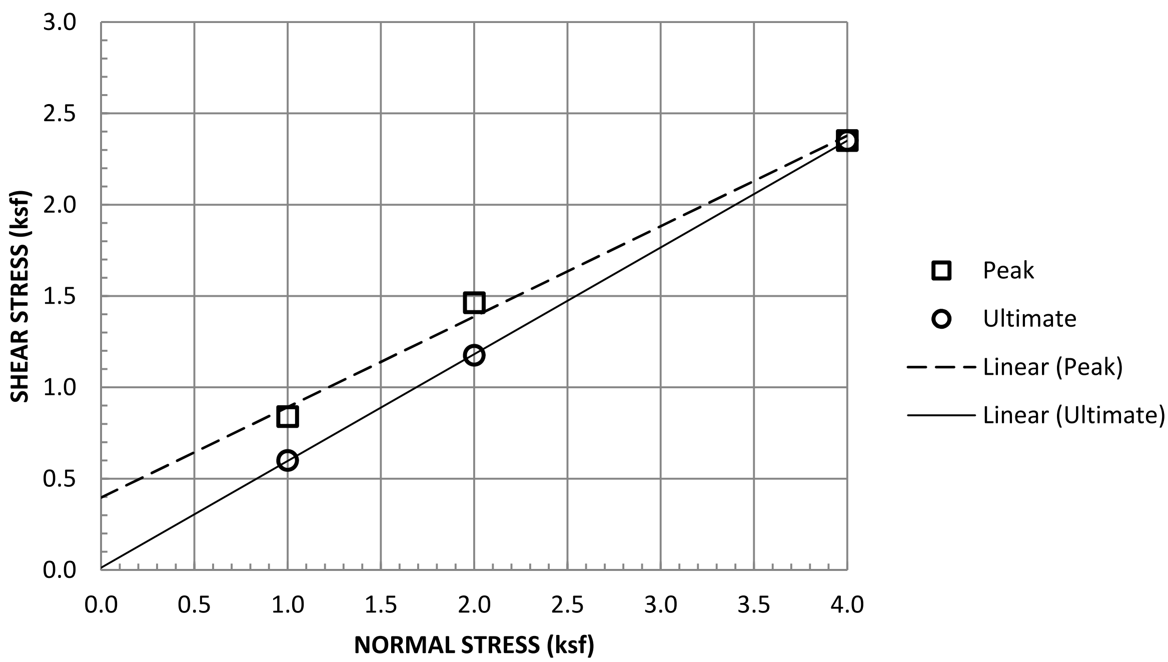
Job Number	Location	Depth	Description
2761.00	B-3	5	Silty Sand / Clayey Sand (SM/SC)

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
109.2	16.5	14.6

DIRECT SHEAR







Sample Type:	Remolded 90% of 122 @ 12%, Saturated		
Normal Stress (ksf)	1	2	4
Peak Shear Stress (ksf)	0.84	1.464	2.352
Peak Displacement (in)	0.003	0.004	0.005
Ultimate Shear Stress (ksf)	0.6	1.176	2.352
Ultimate Displacement (in)	0.25	0.25	0.25
Initial Dry Density (pcf)	109.8	109.8	109.8
Initial Moisture Content (%)	12	12	12
Final Moisture Content (%)	16.1	15.9	16.2
Strain Rate (in/min)	0.01		

Job Number	Location	Depth	Description
2761.00	B-1	0-5	Silty Sand (SM)

APPENDIX C

LIQUEFACTION ANALYSES

TABLE C-1

ANALYSIS OF LIQUEFACTION POTENTIAL
BORING: B-1 (2%PE in 50 yrs; FS=1.3)

Client: McEb
J.N. 2761.00
Site: Fullerton

Hammer Type (D,S,A)	**A**	[Ce= D 0.75, S 0.95, A Hammer Efficiency]		
Boring Diameter, ID (in)	**4**			
Site Acceleration (g)	**0.639**	PGAm w/o MSF		
for a Magnitude (Mw) of	**6.64**	Corresponding to 2%PE in 50 yrs		
and MSF of	**1.43**		Analysis Type:	General
Depth to High GW	**45.0**	ft.	FS for Liquefaction:	**1.3**
Depth to GW during invest.	**51.0**	ft.	FS for Liqu. Settlement:	**1.3**
Hammer Efficiency	**81.1**	%	PI Threshold for Liquefaction:	**12**
Sublayer Thickness	**1.0**	ft.	Min. Moisture Cnt for Liqu. (%LL)	**85**
Depth of Analysis	**50.0**	ft.	Max FS for Plotting:	**5.0**

Layer Label (Auto)	Depth Interval (ft)		Layer Mid-Depth (ft)	Soil Type (USCS)	Fines <#200 Sieve (%)	LL (%)	PI	M (%)	Field Nf (bls/ft)	Sample Type SPT/CA	Soil Wet Density (pcf)
	Top	Bottom									
1	0.0	**6.0**	3.0	**SM**	13			**8.6**	**25**	**CA**	**104**
2	6.0	**9.0**	7.5	**SC**	20	**28**	**9**	**13.3**	**19**	**CA**	**122**
3	9.0	**13.0**	11.0	**CL/SC**	40			**16.3**	**36**	**CA**	**127**
4	13.0	**18.0**	15.5	**SC**	20			**20**	**13**	**SPT**	127
5	18.0	**21.0**	19.5	**SP-SC**	10				**7**	**SPT**	127
6	21.0	**24.0**	22.5	**ML**	60				**7**	**SPT**	127
7	24.0	**26.0**	25.0	**SP**	1				**20**	**SPT**	127
8	26.0	**28.0**	27.0	**ML**	60				**20**	**SPT**	127
9	28.0	**34.0**	31.0	**ML**	60				**9**	**SPT**	127
10	34.0	**36.0**	35.0	**SC**	20				**13**	**SPT**	127
11	36.0	**39.0**	37.5	**SP**	1				**13**	**SPT**	127
12	39.0	**43.0**	41.0	**CL**	60				**15**	**SPT**	127
13	43.0	**46.0**	44.5	**SC**	**13.5**				**19**	**SPT**	127
14	46.0	**48.0**	47.0	**CL**	**60**	31	12		**19**	**SPT**	127
15	48.0	**50.0**	49.0	**CL**	60	**31**	**12**		**19**	**SPT**	127

TABLE C-2
ANALYSIS OF LIQUEFACTION POTENTIAL
BORING: B-1 (2%PE in 50 yrs; FS=1.3)

SM

Client: McEb
J.N. 2761.00
Site: Fullerton

Hammer Type (D,S,A)	A	
Boring Diameter, ID (in)	4	
Site Acceleration (g)	0.639	
for a Magnitude (Mw) of	6.64	
and MSF of	1.43	
Depth to High GW	45	ft.
Depth to GW during invest.	51	ft.
Hammer Efficiency	81.1	%
Sublayer Thickness	1	ft.
Depth of Boring	50	ft.

FS for Liquefaction:	1.3
FS for Liqu. Settlement:	1.3
PI Threshold for Liquefaction:	12
Moisture Cnt Threshold for Liqu. (%LL)	85

Notes: Underlined numbersare estimated values.
(1) Based on current groundwater conditions at the time of investigation.
(2) Based on assumed/proposed high groundwater conditions.
(3) Kα=1.0

(4)
A Layer is located above historically high groundwater
B Factor of Saftey is greater than the specified value of FS=1.3
C The $(N_1)_{60\text{-}cs}$ is greater than 30 blows per foot
D PI > 12 or the in situ moisture content (M%) < 85% LL

Reference: Youd, T.L., et.al., (2001), "Liquefaction Resistance of Soils: Summary Report From The 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils", ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol.127, No.10, pp.817-833, October, 2001.

Layer Label	Depth Interval (ft) Top	Depth Interval (ft) Bottom	Layer Mid-Depth (ft)	Soil Type (USCS)	Fines <#200 Sieve (%)	LL (%)	PI	M (%)	Field Nf (bls/ft)	Sample Type SPT/CA	Soil Wet Density (pcf)	Total Stress (psf)(1)	Effec. Stress (psf)(1)	C_n	C_e	C_b	C_r	C_L	$(N_1)_{60}$ (lbs/ft)	α	β	$(N_1)_{60\text{-}cs}$ (lbs/ft)	Effec. Stress (psf)(2)	R_d	CRR (M=7.5)	Kσ	CSR	FS (3)	To Liquefy Y/N?	Reason(4) not Liquifiable
																												1.30		
1	0.0	1.0	0.5	SM	13			8.6	25	CA	104	52	52	1.7	1.35	1.00	0.75	1.0	24.1	1.9	1.04	26.9	52	1.00	NA	1.00	0.42	NA	N	A
1	1.0	2.0	1.5	SM	13			8.6	25	CA	104	156	156	1.7	1.35	1.00	0.75	1.0	24.1	1.9	1.04	26.9	156	1.00	NA	1.00	0.42	NA	N	A
1	2.0	3.0	2.5	SM	13			8.6	25	CA	104	259	259	1.7	1.35	1.00	0.75	1.0	23.6	1.9	1.04	26.3	259	0.99	NA	1.00	0.42	NA	N	A
1	3.0	4.0	3.5	SM	13			8.6	25	CA	104	363	363	1.6	1.35	1.00	0.75	1.0	22.7	1.9	1.04	25.5	363	0.99	NA	1.00	0.42	NA	N	A
1	4.0	5.0	4.5	SM	13			8.6	25	CA	104	467	467	1.5	1.35	1.00	0.75	1.0	21.9	1.9	1.04	24.6	467	0.99	NA	1.00	0.42	NA	N	A
1	5.0	6.0	5.5	SM	13			8.6	25	CA	104	570	570	1.5	1.35	1.00	0.75	1.0	21.2	1.9	1.04	23.9	570	0.99	NA	1.00	0.42	NA	N	A
2	6.0	7.0	6.5	SC	20	28	9	13.3	19	CA	122	793	793	1.4	1.35	1.00	0.80	1.0	16.0	3.6	1.08	20.9	793	0.99	NA	1.00	0.40	NA	N	A
2	7.0	8.0	7.5	SC	20	28	9	13.3	19	CA	122	915	915	1.3	1.35	1.00	0.80	1.0	15.5	3.6	1.08	20.3	915	0.98	NA	1.00	0.40	NA	N	A
2	8.0	9.0	8.5	SC	20	28	9	13.3	19	CA	122	1037	1037	1.3	1.35	1.00	0.80	1.0	14.9	3.6	1.08	19.7	1037	0.98	NA	1.00	0.40	NA	N	A
3	9.0	10.0	9.5	CL/SC	40			16.3	36	CA	127	1205	1205	1.2	1.35	1.00	0.85	1.0	38.9	5.0	1.20	51.7	1205	0.98	NA	1.00	0.40	NA	N	A
3	10.0	11.0	10.5	CL/SC	40			16.3	36	CA	127	1332	1332	1.2	1.35	1.00	0.85	1.0	37.6	5.0	1.20	50.2	1332	0.98	NA	1.00	0.40	NA	N	A
3	11.0	12.0	11.5	CL/SC	40			16.3	36	CA	127	1459	1459	1.2	1.35	1.00	0.85	1.0	36.4	5.0	1.20	48.7	1459	0.97	NA	1.00	0.40	NA	N	A
3	12.0	13.0	12.5	CL/SC	40			16.3	36	CA	127	1586	1586	1.1	1.35	1.00	0.85	1.0	35.3	5.0	1.20	47.4	1586	0.97	NA	1.00	0.40	NA	N	A
4	13.0	14.0	13.5	SC	20			20	13	SPT	127	1715	1715	1.1	1.35	1.00	0.85	1.2	19.5	3.6	1.08	24.7	1715	0.97	NA	1.00	0.40	NA	N	A
4	14.0	15.0	14.5	SC	20			20	13	SPT	127	1842	1842	1.1	1.35	1.00	0.85	1.2	18.9	3.6	1.08	24.1	1842	0.97	NA	1.00	0.40	NA	N	A
4	15.0	16.0	15.5	SC	20			20	13	SPT	127	1969	1969	1.0	1.35	1.00	0.85	1.2	18.4	3.6	1.08	23.5	1969	0.96	NA	1.00	0.40	NA	N	A
4	16.0	17.0	16.5	SC	20			20	13	SPT	127	2096	2096	1.0	1.35	1.00	0.90	1.2	18.9	3.6	1.08	24.1	2096	0.96	NA	1.01	0.40	NA	N	A
4	17.0	18.0	17.5	SC	20			20	13	SPT	127	2223	2223	1.0	1.35	1.00	0.90	1.2	18.4	3.6	1.08	23.5	2223	0.96	NA	1.00	0.40	NA	N	A
5	18.0	19.0	18.5	SP-SC	10				7	SPT	127	2350	2350	0.9	1.35	1.00	0.90	1.2	9.7	0.9	1.02	10.7	2350	0.96	NA	0.99	0.40	NA	N	A
5	19.0	20.0	19.5	SP-SC	10				7	SPT	127	2477	2477	0.9	1.35	1.00	0.90	1.2	9.4	0.9	1.02	10.5	2477	0.96	NA	0.98	0.40	NA	N	A
5	20.0	21.0	20.5	SP-SC	10				7	SPT	127	2604	2604	0.9	1.35	1.00	0.90	1.2	9.2	0.9	1.02	10.3	2604	0.95	NA	0.97	0.40	NA	N	A
6	21.0	22.0	21.5	ML	60				7	SPT	127	2731	2731	0.9	1.35	1.00	0.90	1.2	9.0	5.0	1.20	15.8	2731	0.95	NA	0.96	0.40	NA	N	A
6	22.0	23.0	22.5	ML	60				7	SPT	127	2858	2858	0.9	1.35	1.00	0.95	1.2	9.2	5.0	1.20	16.1	2858	0.95	NA	0.95	0.40	NA	N	A
6	23.0	24.0	23.5	ML	60				7	SPT	127	2985	2985	0.8	1.35	1.00	0.95	1.2	9.0	5.0	1.20	15.8	2985	0.95	NA	0.94	0.40	NA	N	A
7	24.0	25.0	24.5	SP	1				20	SPT	127	3112	3112	0.8	1.35	1.00	0.95	1.2	25.2	0.0	1.00	25.2	3112	0.94	NA	0.93	0.40	NA	N	A
7	25.0	26.0	25.5	SP	1				20	SPT	127	3239	3239	0.8	1.35	1.00	0.95	1.2	24.6	0.0	1.00	24.6	3239	0.94	NA	0.93	0.40	NA	N	A
8	26.0	27.0	26.5	ML	60				20	SPT	127	3366	3366	0.8	1.35	1.00	0.95	1.2	24.1	5.0	1.20	33.9	3366	0.94	NA	0.92	0.40	NA	N	A
8	27.0	28.0	27.5	ML	60				20	SPT	127	3493	3493	0.8	1.35	1.00	0.95	1.2	23.6	5.0	1.20	33.3	3493	0.94	NA	0.91	0.38	NA	N	A
9	28.0	29.0	28.5	ML	60				9	SPT	127	3620	3620	0.8	1.35	1.00	0.95	1.2	10.4	5.0	1.20	17.5	3620	0.93	NA	0.90	0.38	NA	N	A
9	29.0	30.0	29.5	ML	60				9	SPT	127	3747	3747	0.7	1.35	1.00	1.00	1.2	10.7	5.0	1.20	17.9	3747	0.93	NA	0.90	0.38	NA	N	A
9	30.0	31.0	30.5	ML	60				9	SPT	127	3874	3874	0.7	1.35	1.00	1.00	1.2	10.5	5.0	1.20	17.6	3874	0.93	NA	0.89	0.38	NA	N	A
9	31.0	32.0	31.5	ML	60				9	SPT	127	4001	4001	0.7	1.35	1.00	1.00	1.2	10.3	5.0	1.20	17.4	4001	0.92	NA	0.88	0.38	NA	N	A
9	32.0	33.0	32.5	ML	60				9	SPT	127	4128	4128	0.7	1.35	1.00	1.00	1.2	10.1	5.0	1.20	17.1	4128	0.91	NA	0.88	0.38	NA	N	A
9	33.0	34.0	33.5	ML	60				9	SPT	127	4255	4255	0.7	1.35	1.00	1.00	1.2	9.9	5.0	1.20	16.9	4255	0.90	NA	0.87	0.38	NA	N	A
10	34.0	35.0	34.5	SC	20				13	SPT	127	4382	4382	0.7	1.35	1.00	1.00	1.2	14.1	3.6	1.08	18.8	4382	0.89	NA	0.86	0.38	NA	N	A
10	35.0	36.0	35.5	SC	20				13	SPT	127	4509	4509	0.7	1.35	1.00	1.00	1.2	13.8	3.6	1.08	18.5	4509	0.89	NA	0.86	0.36	NA	N	A
11	36.0	37.0	36.5	SP	1				13	SPT	127	4636	4636	0.6	1.35	1.00	1.00	1.2	13.6	0.0	1.00	13.6	4636	0.88	NA	0.85	0.36	NA	N	A
11	37.0	38.0	37.5	SP	1				13	SPT	127	4763	4763	0.6	1.35	1.00	1.00	1.2	13.3	0.0	1.00	13.3	4763	0.87	NA	0.84	0.36	NA	N	A
11	38.0	39.0	38.5	SP	1				13	SPT	127	4890	4890	0.6	1.35	1.00	1.00	1.2	13.1	0.0	1.00	13.1	4890	0.86	NA	0.84	0.36	NA	N	A
12	39.0	40.0	39.5	CL	60				15	SPT	127	5017	5017	0.6	1.35	1.00	1.00	1.2	14.9	5.0	1.20	22.8	5017	0.85	NA	0.83	0.36	NA	N	A
12	40.0	41.0	40.5	CL	60				15	SPT	127	5144	5144	0.6	1.35	1.00	1.00	1.2	14.6	5.0	1.20	22.5	5144	0.85	NA	0.83	0.36	NA	N	A
12	41.0	42.0	41.5	CL	60				15	SPT	127	5271	5271	0.6	1.35	1.00	1.00	1.2	14.4	5.0	1.20	22.2	5271	0.84	NA	0.82	0.34	NA	N	A
12	42.0	43.0	42.5	CL	60				15	SPT	127	5398	5398	0.6	1.35	1.00	1.00	1.2	14.1	5.0	1.20	22.0	5398	0.83	NA	0.82	0.34	NA	N	A
13	43.0	44.0	43.5	SC	13.5				19	SPT	127	5525	5525	0.6	1.35	1.00	1.00	1.2	17.6	2.0	1.04	20.4	5525	0.82	NA	0.81	0.34	NA	N	A
13	44.0	45.0	44.5	SC	13.5				19	SPT	127	5652	5652	0.6	1.35	1.00	1.00	1.2	17.4	2.0	1.04	20.1	5652	0.81	NA	0.81	0.34	NA	N	A
13	45.0	46.0	45.5	SC	13.5				19	SPT	127	5779	5779	0.6	1.35	1.00	1.00	1.2	17.1	2.0	1.04	19.8	5747	0.81	0.21	0.80	0.34	0.72	Y	
14	46.0	47.0	46.5	CL	60	31	12		19	SPT	127	5906	5906	0.5	1.35	1.00	1.00	1.2	16.8	5.0	1.20	25.2	5812	0.80	NA	0.80	0.34	NA	N	D
14	47.0	48.0	47.5	CL	60	31	12		19	SPT	127	6033	6033	0.5	1.35	1.00	1.00	1.2	16.6	5.0	1.20	24.9	5877	0.79	NA	0.79	0.34	NA	N	D
15	48.0	49.0	48.5	CL	60	31	12		19	SPT	127	6160	6160	0.5	1.35	1.00	1.00	1.2	16.3	5.0	1.20	24.6	5941	0.78	NA	0.79	0.34	NA	N	D
15	49.0	50.0	49.5	CL	60	31	12		19	SPT	127	6287	6287	0.5	1.35	1.00	1.00	1.2	16.1	5.0	1.20	24.3	6006	0.77	NA	0.78	0.34	NA	N	D

Client: McEb

J.N. 2761.00

Site: Fullerton

TABLE C-3
LIQUEFACTION INDUCED SETTLEMENT
BORING B-1 (2%PE in 50 yrs; FS=1.3)

Notes:

(1) Effective ER=55% normalized standard penetration resistance for clean sands, $(N_1)_{60\text{-}cs}$*1.1 (Seed, 1994).
(2) Volumetric strain (Ishihara and Yoshimine, 1992) using $(N_1)_{55\text{-}cs}$.
(3) Volumetric strain (Tokimatsu and Seed, 1987) using $(N_1)_{60\text{-}cs}$.

Depth Interval (ft)		Soil layer thickness (ft)	Fines <#200 Sieve (%)	$(N_1)_{60\text{-}cs}$	$(N_1)_{55\text{-}cs}$ (1)	FS	IY Percent ε_v (2)	CSR*	TS Percent ε_v (3)	IY δ (in.)	TS δ (in.)	Ave δ (in.)
									Total δ (in.)	0.23	0.20	0.22
Top	Bottom											
0.00	1.00	1.00	13	26.9	29.6	NA	0.00	0.42	NA	NA	NA	0
1.00	2.00	1.00	13	26.9	29.6	NA	0.00	0.42	NA	NA	NA	0
2.00	3.00	1.00	13	26.3	29.0	NA	0.00	0.42	NA	NA	NA	0
3.00	4.00	1.00	13	25.5	28.0	NA	0.00	0.42	NA	NA	NA	0
4.00	5.00	1.00	13	24.6	27.1	NA	0.00	0.42	NA	NA	NA	0
5.00	6.00	1.00	13	23.9	26.3	NA	0.00	0.42	NA	NA	NA	0
6.00	7.00	1.00	20	20.9	23.0	NA	0.00	0.40	NA	NA	NA	0
7.00	8.00	1.00	20	20.3	22.3	NA	0.00	0.40	NA	NA	NA	0
8.00	9.00	1.00	20	19.7	21.7	NA	0.00	0.40	NA	NA	NA	0
9.00	10.00	1.00	40	51.7	56.9	NA	0.00	0.40	NA	NA	NA	0
10.00	11.00	1.00	40	50.2	55.2	NA	0.00	0.40	NA	NA	NA	0
11.00	12.00	1.00	40	48.7	53.6	NA	0.00	0.40	NA	NA	NA	0
12.00	13.00	1.00	40	47.4	52.1	NA	0.00	0.40	NA	NA	NA	0
13.00	14.00	1.00	20	24.7	27.1	NA	0.00	0.40	NA	NA	NA	0
14.00	15.00	1.00	20	24.1	26.5	NA	0.00	0.40	NA	NA	NA	0
15.00	16.00	1.00	20	23.5	25.8	NA	0.00	0.40	NA	NA	NA	0
16.00	17.00	1.00	20	24.1	26.5	NA	0.00	0.40	NA	NA	NA	0
17.00	18.00	1.00	20	23.5	25.9	NA	0.00	0.40	NA	NA	NA	0
18.00	19.00	1.00	10	10.7	11.8	NA	0.00	0.40	NA	NA	NA	0
19.00	20.00	1.00	10	10.5	11.5	NA	0.00	0.40	NA	NA	NA	0
20.00	21.00	1.00	10	10.3	11.3	NA	0.00	0.40	NA	NA	NA	0
21.00	22.00	1.00	60	15.8	17.3	NA	0.00	0.40	NA	NA	NA	0
22.00	23.00	1.00	60	16.1	17.7	NA	0.00	0.40	NA	NA	NA	0
23.00	24.00	1.00	60	15.8	17.4	NA	0.00	0.40	NA	NA	NA	0
24.00	25.00	1.00	1	25.2	27.7	NA	0.00	0.40	NA	NA	NA	0
25.00	26.00	1.00	1	24.6	27.1	NA	0.00	0.40	NA	NA	NA	0
26.00	27.00	1.00	60	33.9	37.3	NA	0.00	0.40	NA	NA	NA	0
27.00	28.00	1.00	60	33.3	36.7	NA	0.00	0.38	NA	NA	NA	0
28.00	29.00	1.00	60	17.5	19.2	NA	0.00	0.38	NA	NA	NA	0
29.00	30.00	1.00	60	17.9	19.7	NA	0.00	0.38	NA	NA	NA	0
30.00	31.00	1.00	60	17.6	19.4	NA	0.00	0.38	NA	NA	NA	0
31.00	32.00	1.00	60	17.4	19.1	NA	0.00	0.38	NA	NA	NA	0
32.00	33.00	1.00	60	17.1	18.8	NA	0.00	0.38	NA	NA	NA	0
33.00	34.00	1.00	60	16.9	18.6	NA	0.00	0.38	NA	NA	NA	0
34.00	35.00	1.00	20	18.8	20.7	NA	0.00	0.38	NA	NA	NA	0
35.00	36.00	1.00	20	18.5	20.4	NA	0.00	0.36	NA	NA	NA	0
36.00	37.00	1.00	1	13.6	14.9	NA	0.00	0.36	NA	NA	NA	0
37.00	38.00	1.00	1	13.3	14.7	NA	0.00	0.36	NA	NA	NA	0
38.00	39.00	1.00	1	13.1	14.4	NA	0.00	0.36	NA	NA	NA	0
39.00	40.00	1.00	60	22.8	25.1	NA	0.00	0.36	NA	NA	NA	0
40.00	41.00	1.00	60	22.5	24.8	NA	0.00	0.36	NA	NA	NA	0
41.00	42.00	1.00	60	22.2	24.5	NA	0.00	0.34	NA	NA	NA	0
42.00	43.00	1.00	60	22.0	24.2	NA	0.00	0.34	NA	NA	NA	0
43.00	44.00	1.00	14	20.4	22.4	NA	0.00	0.34	NA	NA	NA	0
44.00	45.00	1.00	14	20.1	22.1	NA	0.00	0.34	NA	NA	NA	0
45.00	46.00	1.00	14	19.8	21.8	0.7	1.93	0.34	1.67	0.23	0.20	0.22
46.00	47.00	1.00	60	25.2	27.7	NA	0.00	0.34	NA	NA	NA	0
47.00	48.00	1.00	60	24.9	27.4	NA	0.00	0.34	NA	NA	NA	0
48.00	49.00	1.00	60	24.6	27.1	NA	0.00	0.34	NA	NA	NA	0
49.00	50.00	1.00	60	24.3	26.8	NA	0.00	0.34	NA	NA	NA	0

TABLE C-4
ANALYSIS OF DRY SEISMIC SETTLEMENT POTENTIAL
BORING B-1 (2%PE in 50 yrs; FS=1.3)

Client: McEb
J.N. 2761.00
Site: Fullerton

Total Seismic Settlement of Unsaturated Soil w/ FS=2.0 (in):	2.45
Subtotal Seismic Settlement of Unsaturated Soil (in)	1.22
Total Thickness of Unsaturated Soil (ft)	45.0

GW Depth: 45 feet
EQ Magnitude 6.64
MSF 1.43

Layer Mid-Depth (ft.)	Soil Type	Eff. Stress σ'_{vo}(tsf)	Clean Sand $(N_1)_{60}$	CSR	(psf) τ_{avg} Avg. Shear Stress	(tsf) σ_m' Mean Bulk Stress	(tsf) G_{max} Max. Dyn.Shr. Mod.	γ_{eff} (G_{eff}/G_{max})	Eff. Cyclic Shr.Strain γ_{eff} Fig.11	Eff. Cyclic Shr.Strain γ_{eff} (%)	Volume Strain (%) Fig.13	EQ Mag. Factor	Layer Thickness (ft.)	Estimated Dry Sand Seismic Settlement (in.)
0.5	SM	0.03	26.9	0.42	21.8	0.02	172.1	6.33E-05	1.04E-04	1.04E-02	6.80E-03	1.43	1.0	0.001
1.5	SM	0.08	26.9	0.42	65.3	0.05	298.0	1.10E-04	2.82E-04	2.82E-02	1.85E-02	1.43	1.0	0.002
2.5	SM	0.13	26.3	0.42	108.9	0.08	382.1	1.43E-04	5.23E-04	5.23E-02	3.54E-02	1.43	1.0	0.003
3.5	SM	0.18	25.5	0.42	152.5	0.12	447.0	1.71E-04	8.82E-04	8.82E-02	6.24E-02	1.43	1.0	0.005
4.5	SM	0.23	24.6	0.42	196.0	0.15	501.4	1.95E-04	1.88E-03	1.88E-01	1.39E-01	1.43	1.0	0.012
5.5	SM	0.29	23.9	0.42	239.6	0.19	548.6	2.18E-04	2.27E-03	2.27E-01	1.75E-01	1.43	1.0	0.015
6.5	SC	0.40	20.9	0.40	317.3	0.26	619.2	2.56E-04	1.52E-03	1.52E-01	1.36E-01	1.43	1.0	0.011
7.5	SC	0.46	20.3	0.40	366.1	0.30	658.6	2.78E-04	2.18E-03	2.18E-01	2.02E-01	1.43	1.0	0.017
8.5	SC	0.52	19.7	0.40	414.9	0.34	694.5	2.99E-04	3.04E-03	3.04E-01	2.93E-01	1.43	1.0	0.025
9.5	CL/SC	0.60	51.7	0.40	482.2	0.39	1029.1	2.34E-04	7.37E-04	7.37E-02	2.24E-02	1.43	1.0	0.002
10.5	CL/SC	0.67	50.2	0.40	532.9	0.43	1071.2	2.49E-04	7.80E-04	7.80E-02	2.37E-02	1.43	1.0	0.002
11.5	CL/SC	0.73	48.7	0.40	583.7	0.47	1110.2	2.63E-04	7.61E-04	7.61E-02	2.31E-02	1.43	1.0	0.002
12.5	CL/SC	0.79	47.4	0.40	634.4	0.52	1146.8	2.77E-04	7.14E-04	7.14E-02	2.17E-02	1.43	1.0	0.002
13.5	SC	0.86	24.7	0.40	685.8	0.56	961.6	3.57E-04	1.59E-03	1.59E-01	1.17E-01	1.43	1.0	0.010
14.5	SC	0.92	24.1	0.40	736.6	0.60	988.3	3.73E-04	1.83E-03	1.83E-01	1.39E-01	1.43	1.0	0.012
15.5	SC	0.98	23.5	0.40	787.4	0.64	1013.6	3.88E-04	2.09E-03	2.09E-01	1.64E-01	1.43	1.0	0.014
16.5	SC	1.05	24.1	0.40	838.2	0.68	1054.3	3.98E-04	2.19E-03	2.19E-01	1.66E-01	1.43	1.0	0.014
17.5	SC	1.11	23.5	0.40	889.0	0.72	1077.6	4.13E-04	2.43E-03	2.43E-01	1.91E-01	1.43	1.0	0.016
18.5	SP-SC	1.17	10.7	0.40	939.8	0.76	855.6	5.49E-04	8.03E-03	8.03E-01	1.37E+00	1.43	1.0	0.115
19.5	SP-SC	1.24	10.5	0.40	990.6	0.80	871.6	5.68E-04	8.10E-03	8.10E-01	1.41E+00	1.43	1.0	0.118
20.5	SP-SC	1.30	10.3	0.40	1041.4	0.85	886.9	5.87E-04	8.17E-03	8.17E-01	1.44E+00	1.43	1.0	0.121
21.5	ML	1.37	15.8	0.40	1092.2	0.89	1046.6	5.22E-04	4.78E-03	4.78E-01	6.36E-01	1.43	1.0	0.053
22.5	ML	1.43	16.1	0.40	1143.0	0.93	1077.9	5.30E-04	4.30E-03	4.30E-01	5.64E-01	1.43	1.0	0.047
23.5	ML	1.49	15.8	0.40	1193.8	0.97	1095.8	5.45E-04	3.97E-03	3.97E-01	5.32E-01	1.43	1.0	0.045
24.5	SP	1.56	25.2	0.40	1244.6	1.01	1304.5	4.77E-04	1.97E-03	1.97E-01	1.42E-01	1.43	1.0	0.012
25.5	SP	1.62	24.6	0.40	1295.4	1.05	1321.1	4.90E-04	2.15E-03	2.15E-01	1.59E-01	1.43	1.0	0.013
26.5	ML	1.68	33.9	0.40	1346.2	1.09	1496.6	4.50E-04	1.54E-03	1.54E-01	6.79E-02	1.43	1.0	0.006
27.5	ML	1.75	33.3	0.38	1327.2	1.14	1515.5	4.38E-04	1.39E-03	1.39E-01	6.34E-02	1.43	1.0	0.005
28.5	ML	1.81	17.5	0.38	1375.4	1.18	1246.9	5.52E-04	3.24E-03	3.24E-01	3.78E-01	1.43	1.0	0.032
29.5	ML	1.87	17.9	0.38	1423.7	1.22	1277.9	5.57E-04	3.29E-03	3.29E-01	3.71E-01	1.43	1.0	0.031
30.5	ML	1.94	17.6	0.38	1471.9	1.26	1293.2	5.69E-04	3.52E-03	3.52E-01	4.05E-01	1.43	1.0	0.034
31.5	ML	2.00	17.4	0.38	1520.2	1.30	1308.1	5.81E-04	3.74E-03	3.74E-01	4.41E-01	1.43	1.0	0.037
32.5	ML	2.06	17.1	0.38	1568.5	1.34	1322.7	5.93E-04	3.97E-03	3.97E-01	4.77E-01	1.43	1.0	0.040
33.5	ML	2.13	16.9	0.38	1616.7	1.38	1337.0	6.05E-04	4.20E-03	4.20E-01	5.14E-01	1.43	1.0	0.043
34.5	SC	2.19	18.8	0.38	1665.0	1.42	1405.2	5.92E-04	3.71E-03	3.71E-01	3.86E-01	1.43	1.0	0.032
35.5	SC	2.25	18.5	0.36	1623.1	1.47	1418.5	5.72E-04	3.08E-03	3.08E-01	3.29E-01	1.43	1.0	0.028
36.5	SP	2.32	13.6	0.36	1668.8	1.51	1297.8	6.43E-04	4.98E-03	4.98E-01	7.98E-01	1.43	1.0	0.067
37.5	SP	2.38	13.3	0.36	1714.5	1.55	1307.8	6.55E-04	5.23E-03	5.23E-01	8.42E-01	1.43	1.0	0.071
38.5	SP	2.44	13.1	0.36	1760.2	1.59	1317.6	6.68E-04	5.46E-03	5.46E-01	8.85E-01	1.43	1.0	0.074
39.5	CL	2.51	22.8	0.36	1805.9	1.63	1603.2	5.63E-04	2.51E-03	2.51E-01	2.04E-01	1.43	1.0	0.000
40.5	CL	2.57	22.5	0.36	1851.7	1.67	1616.4	5.73E-04	2.58E-03	2.58E-01	2.12E-01	1.43	1.0	0.000
41.5	CL	2.64	22.2	0.34	1792.0	1.71	1629.3	5.50E-04	2.13E-03	2.13E-01	1.78E-01	1.43	1.0	0.000
42.5	CL	2.70	22.0	0.34	1835.2	1.75	1642.0	5.59E-04	2.17E-03	2.17E-01	1.84E-01	1.43	1.0	0.000
43.5	SC	2.76	20.4	0.34	1878.3	1.80	1620.5	5.80E-04	2.36E-03	2.36E-01	2.18E-01	1.43	1.0	0.018
44.5	SC	2.83	20.1	0.34	1921.5	1.84	1631.4	5.89E-04	2.39E-03	2.39E-01	2.23E-01	1.43	1.0	0.019

D.2 - Paleontological Records Search Results

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Kenneth L. Finger, Ph.D.
Consulting Paleontologist

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March 11, 2021

Dana DePietro
FirstCarbon Solutions
1350 Treat Boulevard, Suite 380
Walnut Creek, CA 94597

Re: Paleontological Records Search: Highland and Valencia Mixed-Use Project (1412.0006), City of Fullerton, Orange County

Dear Dr. DePietro:

As per the request of Madelyn Dolan, I have performed a records search on the University of California Museum of Paleontology (UCMP) database for the proposed Highland and Valencia Mixed-Use Project in Fullerton. The project site is located at the northwest corner of the intersection of Highland Avenue and Valencia Drive. The applicant is proposing to demolish an existing self-serve car wash and to construct a three-story mixed-use building with 1,152 square feet of commercial use and 20 residential apartments. Its PRS location is NW¼, SE¼, Sec. 33, T3S, R10W, Anaheim quadrangle (USGS 7.5'-series topographic map).

Geologic Mapping

As shown here on part of the geologic map by Morton (2004), the surface of the entire project site (yellow rectangle at center) consists solely of young alluvial fan deposits (Qyf_{sa}). The differentiation of Qyf_{sa} and Qyf_{a} is not explained, as the suffices used in Morton's map legend are numbers 1 through 7. All are listed as Holocene except for Qyf_{1}, which is early Holocene–late Pleistocene. The surrounding half-mile search area (dashed outline) also includes middle to early Pleistocene alluvial fan deposits (Qvof) just north of the search area.

Paleontological Records Search

The paleontological records search on the UCMP database focused on the late Pleistocene deposits in the Anaheim quadrangle. The results were negative — no recorded vertebrate or plant localities were revealed.

Paleontological Assessment and Mitigation Recommendations

A preconstruction paleontological walkover survey of the proposed project site is not recommended because its natural surface is heavily disturbed by prior development. I also do not recommend paleontological monitoring of earth-disturbing construction activities because it is uncertain if any of the surficial deposits predate the Holocene. It would be prudent to obtain a records search report from the Natural History Museum of Los Angeles County, as that is where most significant paleontological resources from this part of Orange County would be housed. Without it, I recommend having a professional paleontologist provide the construction crew with a primer on recognizing the kinds of fossils that could be encountered and instructions on how to proceed should that occur.

Sincerely,

Ken Finger

Reference Cited

Morton, D.M., 2004, Preliminary digital geologic map of the Santa Ana 30'X60' quadrangle, southern California, version 2.0. U.S. Geological Survey, Open-File Report 99-172, scale 1:100,000.