APPENDIX B:

Geotechnical Investigations

ALBUS-KEEFE & ASSOCIATES, INC
GEOTECHNICAL CONSULTANTS

March 10, 2020
J.N.: 2871.00

Mr. Tom Lee
VenturePoint Inc.
4685 MacArthur Court, Suite 375
Newport Beach, California 92660

Subject: **Preliminary Geotechnical Investigation, Proposed Retail and Self-Storage Facility, 950 E. 33rd Street, City of Signal Hill, California.**

Dear Mr. Lee,

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

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

Sincerely,

ALBUS-KEEFE & ASSOCIATES, INC.

Michael O. Spira
Principal Engineering Geologist

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of our proposed geotechnical investigation is to evaluate the subsurface conditions beneath the site to assess the feasibility of site development and to develop preliminary geotechnical recommendations for the proposed development. The scope of our work included:

- Review of published geologic maps, reports, and seismic data of the general vicinity
- Review of the referenced Preliminary Grading and Utility Plan
- Exploratory drilling and soil sampling
- Laboratory testing of selected soil samples
- Engineering and geologic analyses
- Development of recommendations for site construction
- Preparation of this report

1.2 SITE LOCATION AND DESCRIPTION

The subject site is located southwest of the intersection of California Avenue and East 33nd Street, in the city of Signal Hill, California. The subject site encompasses both the Target property and the Fields North Trust Property to the south. The site is generally bounded by East 33nd Street to the north, California Avenue to the east, an offramp of the 405 Freeway to the south, and the Chick-fil-A restaurant property to the west. The location of the site and its relationship to the surrounding areas are shown on Figure 1, Site Location Map.

The Target property is relatively level property that is very gently inclined to the north and east. Surface elevations within the site vary from approximately 92 above Mean Sea Level (MSL) to roughly 84 feet MSL. The Fields North Trust Property is also a relatively level property that is very gently inclined to the south. Surface elevations within the site vary from approximately 94 above Mean Sea Level (MSL) to 92 feet MSL. Along the northerly margin of the Fields North Trust Property is a slope up to 6.5 feet in maximum height that descends to the Target property.

The Target property is currently being utilized for retail purposes and includes a large retail building and an associated asphalt covered parking lot with underground utility lines and overhead lighting. The Field's North Trust Property is currently being utilized for the extraction of oil and gas and is generally a dirt covered lot with existing oil wells (4 operating wells and 3 abandoned wells) and associated above and below ground oil field improvements, and various storm drain improvements. A large sign supported by a steel column is also located in the southeasterly corner of the Fields North Trust Property. A utility bridge, that crosses the 405 Freeway, also extends to the Fields

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SITE LOCATION MAP

Proposed Retail and Self-Storage Facility
VenturePoint Project
950 E. 33rd Street,
Signal Hill, California

NOT TO SCALE

FIGURE 1

Trust Property. South of the two properties, within the adjoining 405 San Diego Freeway easement, is a vegetated 10 foot to 20 foot wide level berm and a descending 2:1 (H:V) graded slope up to roughly 25 feet in height.

1.3 PROPOSED DEVELOPMENT

The Preliminary Grading and Utility Plan by Ware Malcomb indicates that the existing parking lot for the Target property and the Fields North Trust Property will be redeveloped to accommodate the construction of three one-story retail buildings and the construction of a three-story storage facility with an associated RV storage area within the southern portion of the site. Solar shade structures are proposed for the RV parking stalls. One of the retail buildings is to extend off the southwest corner of the existing retail building. Other site development will include a new parking lot, driveways, perimeter walls, underground utilities and a storm water infiltration system.

Based on our review of the referenced Preliminary Grading and Utility Plan, the proposed grading will generally involve minor cut-and-fill grading. Proposed maximum fills and cuts are approximately 3 feet and 7 feet, respectively. The cuts are generally limited to the Field North Trust Property. No fill or cut slopes are currently proposed.

No structural plans were available in preparing of this report. However, we anticipate that buildings will either use masonry block or concrete tilt-up walls and concrete slabs on grade. Foundations will likely utilize both continuous and spread footings. Structural loads on foundations are anticipated to be light to moderate.

1.4 PREVIOUS GEOTECHNICAL STUDIES

A previous geotechnical investigation report was prepared by this firm for the Fields North Trust Property on September 18, 2001. Our investigation involved the excavating of 3 exploratory borings up to 20 feet in depth and 10 exploratory trenches up to 4 feet in depth. The exploratory borings were excavated in August 2001 utilizing a 24-inch-diameter bucket auger drill rig. The exploratory trenches were also excavated in August 2001 utilizing a rubber-tire backhoe. Selected samples obtained from the boring excavations were also tested in our soil laboratory. Our testing consisted of soil classification, in-situ moisture and dry density, maximum dry density and optimum moisture, expansion index, sulfate content, consolidation, and direct shear. The approximate locations of the exploratory excavations completed by this firm are shown on the enclosed Geotechnical Map, Plate 1. Our exploration Logs are presented in Appendix C. A description of laboratory test criteria and test results are also presented in Appendix C.

2.0 INVESTIGATION

2.1 RESEARCH

We reviewed our previous geotechnical investigation for the Fields North Trust Property as well as previous geotechnical and environmental investigation reports prepared by this firm and others in the site vicinity. We also reviewed geologic publications, maps, and historic aerial photographs for the site and surrounding areas. See the reference section of the report for a complete listing of the referenced documents. Data from these sources were utilized to develop some of the findings and conclusions presented herein.

2.2 SUBSURFACE EXPLORATION

Subsurface exploration for this investigation was completed on January 22, 2020. Our subsurface investigation involved the excavation and logging of 4 exploratory borings. The borings were drilled to depths of approximately 26.5 to 51.5 feet below the existing ground surface utilizing a truck-mounted, hollow-stem-auger drill rig. A representative of *Albus-Keefe & Associates, Inc.* logged the exploratory excavation. Visual and tactile identifications were made of the materials encountered, and their descriptions are presented on the Exploration Log in Appendix A. The approximate locations of the exploratory excavations completed by this firm are shown on the enclosed Geotechnical Map, Plate 1.

Bulk and relatively undisturbed 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. During each sampling interval, the sampler was driven 12 inches with successive drops of a 140-pound automatic hammer free falling approximately 30 inches. The number of blows required to advance the split-spoon sampler was recorded for each six inches of advancement. A representative “blow count” for each sample is recorded on the boring logs. 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.

Upon completion of drilling, three additional borings were drilled adjacent to borings B-4 (P-3), B-6 (P-1), and B-7 (P-2). 3-inch-diameter casings were installed for subsequent percolation testing as part of our infiltration study for proposed water quality improvements. The location of the percolation wells is also depicted the enclosed Geotechnical Map, Plate 1. The findings from our percolation testing are presented under separate cover in our referenced Infiltration Study for the site.

2.3 LABORATORY TESTING

Selected soil samples of representative earth materials were tested to assist in the formulation of conclusions and recommendations presented in this report. Tests consisted of in-situ moisture content and dry density, expansion potential, soluble sulfate analysis, direct shear, maximum dry density, and grain-size analysis. Descriptions of laboratory testing and a summary of the test results are presented in Appendix B.

3.0 GEOLOGIC CONDITIONS

3.1 SETTING

The project area is located approximately 1.5 miles northwest of elevated ridge known as Signal Hill, which is one of a series of uplifted anticlinal hills and mesas positioned within and/or adjacent the Newport-Inglewood structural zone. Bedrock encountered beneath the site to the depths explored consists of non-marine and marine sediments of the upper Pleistocene-age Lakewood Formation (Qlw). Marine sediments of the lower Pleistocene-age San Pedro Formation (Qsp) underlie the Lakewood Formation at depth. The bedrock is generally covered by artificial fill (Qaf) associated with existing and previous site uses. Detailed descriptions of each of the units are provided in the following section.

3.2 GEOLOGIC UNITS

3.2.1 Artificial Fill (Qaf)

Artificial fill materials were generally encountered in the Fields North Trust portion of the site. Where encountered during this investigation and as previously reported, these fills generally consist of fine-grained silty sands that are various shades of gray and brown. The fill materials are generally dry to moist with some localized wet areas and medium dense. The artificial fills generally ranged in thickness from approximately 1 to 2 feet. However, deeper fills associated with previous development of the Target property, previous oil field operations, previous demolition operations, and previous environmental operation are likely present within portions the subject site. These fills can easily extend up to 10 feet or more in depths.

3.2.2 Lakewood Formation (Qlw)

Upper Pleistocene-age non-marine and marine sedimentary bedrock materials of the Lakewood Formation underlie the entire site. The upper portion of the Lakewood Formation encountered consists primarily of fine-grained, brown to red-brown, silty/clayey sandstone that is damp, moderately hard, massive, and locally porous (pinhole pores) particularly within the weathered upper 1 to 2 feet. At depth, this unit grades to a fine-grained, tan to light gray sandstone, that is slightly micaceous and friable.

3.2.3 San Pedro Formation (Qsp)

Marine sediments of the lower Pleistocene-age San Pedro Formation (Qsp) underlie the Lakewood Formation at depth. Where encountered in our exploratory boring B-6 at a depth of approximately 32 feet, respectively, this unit is generally comprised of fine to coarse grained sandstone. This unit is typically mottled with reddish brown oxidation staining, damp, and very dense.

3.3 GROUNDWATER

Groundwater was not encountered within our exploratory excavations to the maximum depths explored (51.5 feet below the existing ground surface). A review of the referenced Seismic Hazard Zone Report 28 indicates that historical high groundwater levels are not available for this area. Our review of the referenced literature for the site and surrounding area do not indicate the presence of shallow ground water (less than 50 feet) beneath the site.

3.4 FAULTING

Geologic literature and field exploration do not indicate the presence of active faulting within the site. The site does not lie within an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act. Table 3.1 presents a summary of all the known seismically active faults within 10 miles of the site based on the 2008 National Seismic Hazards Maps.

TABLE 3.1
SUMMARY OF SEISMICALLY ACTIVE FAULTS

Name	Distance (miles)	Slip Rate (mm/yr.)	Preferred Dip (degrees)	Slip Sense	Rupture Top (km)	Fault Length (km)
Newport Inglewood Connected alt 2	0.24	1.3	90	strike slip	0	208
Newport Inglewood Connected alt 1	0.3	1.3	89	strike slip	0	208
Newport-Inglewood, alt 1	0.3	1	88	strike slip	0	65
Palos Verdes Connected	6.3	3	90	strike slip	0	285
Palos Verdes	6.3	3	90	strike slip	0	99
Puente Hills (Santa Fe Springs)	6.77	0.7	29	thrust	2.8	11
Puente Hills (Coyote Hills)	9.55	0.7	26	thrust	2.8	17
Puente Hills (LA)	9.86	0.7	27	thrust	2.1	22

4.0 ANALYSES

4.1 SEISMICITY AND SEISMIC DESIGN PARAMETERS

2019 CBC requires seismic parameters in accordance with ASCE7-16. As such, we have performed probabilistic seismic analyses per ASCE7-16 utilizing the U.S. Seismic Design Maps accessed through the Applied Technical Council (ATC) web application. From our analyses, we obtain a PGA of 0.725g. The site coefficient, F_{PGA}, for this range of PGA and site class C is 1.2. Therefore, the site modified peak ground acceleration is $PGA_M = 1.2 \times 0.725 = 0.87g$. The mean event associated with a probability of exceedance equal to 2% over 50 years has a moment magnitude of 6.97 with a mean distance to the seismic source of 4.1 miles.

4.2 STATIC SETTLEMENT

Based on our review of laboratory test data, blow count data obtained during this investigation, and from the referenced geotechnical reports, the weathered bedrock are prone to a slight collapse upon wetting (hydrocollapse). Settlement from these materials would likely exceed 1 inch, of which significant portions of settlement could occur after construction of proposed structures.

The unweathered bedrock materials beneath the site have a low compressibility and are generally not subject to hydro-collapse. Settlement of these materials due to the weight of the proposed structures or introduction of water is not anticipated to exceed approximately 1-inch.

5.0 CONCLUSIONS

5.1 FEASIBILITY OF PROPOSED DEVELOPMENT

From a geotechnical point of view, the proposed site development is considered feasible. Furthermore, it is also our opinion that the proposed development will not adversely impact the stability of adjoining properties. Key geotechnical issues that could have significant fiscal impacts on development of proposed site improvements are discussed further in later sections of this report.

5.2 GEOLOGIC HAZARDS

5.2.1 Ground Rupture

No active faults are known to project through the site nor does the site lie within the bounds of an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act. The closest known active fault is the Newport-Inglewood fault located about 0.24 mile from the site. As such, the potential for ground rupture due to a fault displacement beneath the site is considered very low.

5.2.2 Ground Shaking

The site is 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 structures, the property will probably experience similar moderate to occasionally high ground shaking from these fault zones, as well as some background shaking from other seismically active areas of the Southern California region. Design and construction in accordance with the current California Building Code (CBC) requirements is anticipated to address the issues related to potential ground shaking.

5.2.3 Landsliding

The site is not located within an area identified by the California Geologic Survey (CGS) as having potential for seismic slope instability. In addition, review of referenced publications and geotechnical reports have indicated that geologic hazards associated with landsliding are not anticipated at the sites.

5.2.4 Liquefaction

Engineering research of soil liquefaction potential (Youd, et al., 2001) indicates that generally three basic factors must exist concurrently in order for liquefaction to occur. These factors include:

- A source of ground shaking, such as an earthquake, capable of generating soil mass distortions.

- A relatively loose silty and/or sandy soil.
- A relative shallow groundwater table (within approximately 50 feet below ground surface) or completely saturated soil conditions that will allow positive pore pressure generation.

The liquefaction susceptibility of the onsite soils was evaluated by analyzing the potential concurrent occurrence of the above-mentioned three basic factors. The liquefaction evaluation for the site was completed under the guidance of Special Publication 117A: Guidelines for Evaluating and Mitigating Seismic Hazards in California (CDMG, 2008).

The site is underlain by very dense bedrock materials of the Lakewood and San Pedro Formations and the depth to historic high and current groundwater is not present within the upper 50 feet of the site. As such the potential for liquefaction at the site is considered to be low. Furthermore, the site is not located within a mapped California Geologic Survey liquefaction hazard zone.

5.3 GROUNDWATER

As previously discussed in Section 3.3, groundwater is likely well below the existing ground surface. However, groundwater conditions in the future may vary substantially as a result of seasonal variations of rainfall and future site development. Provided the recommendations concerning surface and subsurface drainage improvements, and other pertinent recommendations contained in this report are incorporated into the construction of the project, adverse effects from future groundwater conditions are not anticipated at the site.

5.4 STATIC SETTLEMENT

The information obtained from our exploration and laboratory testing indicate that the weathered bedrock is porous and prone to hydrocollapse. In its current state, this material is likely to cause settlements beyond the tolerances of proposed site development. Provided the grading is performed in accordance with the recommendations provided herein, total and differential settlement is not anticipated to exceed 1 inch and 1/2 inch over 30 feet, respectively. The estimated magnitudes of settlement are considered within tolerable limits for the proposed structures.

5.5 EXCAVATION AND MATERIAL CHARACTERISTICS

The onsite earth materials are anticipated to be easily excavated with conventional heavy earthmoving equipment. The site earth materials are generally considered suitable for reuse as fill materials provided they are cleared of deleterious debris and satisfy the requirements of the environmental consultant. Most of the onsite earth materials are at or below the optimum moisture content. As such, some water will generally be required to prepare these materials for use as compacted fill.

Cuts in the fills and the bedrock are not anticipated to generate oversized materials.

Demolition of the existing site improvements will generate a considerable amount of concrete and asphaltic concrete debris. Significant portions of concrete and asphaltic concrete debris can likely be reduced in size to less than 4 inches and incorporated within fill soils during earthwork operations.

5.6 SHRINKAGE AND SUBSIDENCE

Volumetric changes in earth quantities will occur when excavated onsite soil materials are replaced as engineered compacted fill. Our estimates of shrinkage and bulking, based on laboratory test data from nearby sites and our experience with similar projects, are summarized in Table 5.1 below.

TABLE 5.1
Estimates of Shrinkage and Bulking

MATERIAL	VOLUME CHANGE	SHRINKAGE/BULKING
Artificial Fill (Qaf)	5% to 15%	Shrinkage
Bedrock upper 5 ft	3% to 7%	Shrinkage
Bedrock below 5 ft	2% to 4%	Bulking

Subsidence as a result of scarification and re-compaction of exposed surfaces is expected to be negligible.

The above 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.

5.7 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 to Low** expansion potential. Additional testing for soil expansion may 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 earth earthwork and grading should be performed in accordance with applicable requirements of Cal/OSHA, applicable specifications of the Grading Codes of the City of Signal Hill, 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 that 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 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, oversized materials, vegetation and other deleterious materials should be removed from the areas to be developed. Existing underground improvements such as onsite disposal systems, utility lines, abandoned pipelines, previous oil field improvements, etc. are also anticipated at the site. If encountered during site development, these improvements will require proper abandonment or be completely removed from the site.

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.

6.1.4 Oil Well Abandonment

Any oil wells to be abandoned should be brought to the attention of the project geotechnical consultant for specific geotechnical layback and backfill recommendations.

6.1.5 Ground Preparation

All existing artificial fill (Qaf) and the weathered portion (upper 1 to 2 feet) of the bedrock are considered unsuitable for support of proposed fills and site development. These materials should be removed to expose competent bedrock (Qlw) and replaced as engineered compacted fill. In addition, there are numerous exploratory trenches excavated by this firm as well as other various excavations that were backfilled without compaction. These backfills will also require removal and replacement with compacted fill. Removal depths are anticipated to vary from 2 to 4 feet below existing ground surface and are shown on the Geotechnical Map, Plate 1.

In areas of the proposed screen walls and pavement, the removals may be limited to within the existing artificial soils with an estimated thickness of approximately 1 to 2.5 feet and with a moisture content at or above the optimum moisture content and a dry density of at least 90 percent of the laboratory determined maximum dry density. Moisture and density testing should be performed to confirm the competency of the fill materials to be left in-place.

Removals should extend laterally beyond the limits of the proposed buildings a distance equal to the depth of removal (i.e. 1:1 projection) but not less than 5 feet. For screen walls and roadways, the removals should be extend laterally to at least the edge of the structure or pavement.

The grading contractor should take appropriate measures when excavating adjacent any existing onsite and offsite improvements to remain in-place to avoid disturbing or compromising support of existing structures. Where removals are limited by the existing onsite and offsite improvements and/or property lines, specific recommendations should be provided by this firm. Special considerations may also be required in the construction of proposed improvements affected by the limited removals.

Building Pad Overexcavations: In addition to general removal of unsuitable soils, the building pads should be over-excavated at least 3 feet below bottom of footings. This overexcavation should extend at least 5 feet beyond the outer edge/foundations of the buildings. If the depth of fill will vary by more than 12 feet below a building due to well abandonment and/or removal of existing loose backfills, backcuts within the building limits should be laid back to 2:1 (H:V) or flatter and evaluated by the geotechnical consultant. Prior to fill placement, the geotechnical consultant should determine if deeper overexcavation is also necessary around the deepened well excavation.

Parking Areas and Driveways: Existing soils within driveway and parking areas should be removed to at least 12 inches below the proposed pavement subgrade and replaced with engineered compacted fill.

All removal and overexcavation bottoms 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.

6.1.6 Scarification

Following removals, the exposed grade should first be scarified to a depth of 6 inches; moisture conditioned to above the optimum moisture content, and then compacted to at least 90 percent of the laboratory determined maximum dry density.

6.1.7 Temporary Excavations

Temporary construction slopes and trench excavations in the surficial units may be cut vertically up to a height of 5 feet provided that no surcharging of the excavations is present. Temporary excavations greater than 5 feet in height but no more than 10 feet should be laid back to a 1:1 (H:V) or flatter or shored to mitigate the potential for instability. Where temporary excavations expose granular soils, laybacks should be limited gradients of 1½ :1 (H:V) or flatter with no allowances of a vertical height.

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.

The grading contractor should take appropriate measures when excavating adjacent existing improvements to avoid disturbing or compromising support of existing structures.

6.1.8 Fill Placement

In general, materials excavated from the site may be used as fill provided they are free of deleterious materials, do not contain rocks greater than 6 inches in maximum dimension within 3 feet of finished pad grade and do not contain rocks greater than 12 inches in maximum dimension below 3 feet from finish pad grade. Rocks greater than 12 inches in diameter that cannot be reduced in size should be removed from the site. Asphaltic concrete debris generated by site demolition can be reduced to no more than 4 inches in maximum dimension and incorporated with fill soils during earthwork operations. All fills should be sufficiently well graded to prevent nesting of larger particles. Fill should be placed in lifts no greater than 8 inches in loose thickness, moisture-conditioned to above the optimum moisture content, and then compacted in place to at least 90 percent of the maximum dry density determined in accordance with ASTM D 1557. Each lift should be treated in a similar manner. Subsequent lifts should not be placed until the project geotechnical consultants have approved the preceding lift.

To mitigate the potential for creating building pads with variable expansion potentials, we recommend that fill materials in the upper 3 feet of the building pads be placed in lifts across the entire building pad with soils that possess uniform expansion potentials.

6.1.9 Import Materials

If import materials are required to achieve the proposed finish grades, the import soils should have an Expansion Index (EI) less than 25 (ASTM D 4829) and negligible soluble sulfate content. Import sources should be indicated to the geotechnical consultant at least 3 days prior to hauling the materials to the site so that appropriate testing and evaluation of the fill materials can be performed in advance.

6.2 SEISMIC DESIGN PARAMETERS

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

TABLE 6.1
2019 CBC Seismic Design Parameters

Parameter	Value
Site Class	C
Mapped MCE_R Spectral Response Acceleration, short periods, S_S	1.658
Mapped MCE_R Spectral Response Acceleration, at 1-sec. period, S_1	0.597
Site Coefficient, Fa	1.0
Site Coefficient, Fv	1.403
Adjusted MCE_R Spectral Response Acceleration, short periods, S_{MS}	1.990
Adjusted MCE_R Spectral Response Acceleration, at 1-sec. period, S_{M1}	0.838
Design Spectral Response Acceleration, short periods, S_{DS}	1.326
Design Spectral Response Acceleration, at 1-sec. period, S_{D1}	0.558
Seismic Design Category (SDC)	D

MCE = Maximum Considered Earthquake

6.3 CONVENTIONAL FOUNDATION DESIGN

6.3.1 General

The following design parameters are provided to assist the project structural engineer to design foundation systems to support the proposed structures at the site. Recommendations for design of other foundation systems will be provided upon request. These design parameters are based on typical site materials encountered during subsurface exploration and are provided for preliminary design and estimating purposes. Depending on actual materials encountered during site grading and actual foundation loads, the design parameters presented herein may require modification.

6.3.2 Soil Expansion

The recommendations presented herein are based on soils with a **Low** expansion potential (EI<51). 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 Settlement

Foundations should be designed for total and differential settlement up to 1 inch and ½-inch over 30 feet, respectively. These estimated magnitudes of settlement should be considered by the structural engineer in design of the proposed structures at the site.

6.3.4 Allowable Bearing Value

Provided site grading is performed as recommended herein, a bearing value of 2,000 pounds per square foot (psf) may be used for continuous beams or isolated spread footings. The bearing value is based on beams having a minimum width of 12 inches and founded at a minimum of 12 inches below the lowest adjacent grade. The bearing value for isolated footings is based on a minimum width of 24 inches and founded a minimum of 12 inches. The above value may be increased by 250 psf and 700 psf for each additional foot in width and depth, respectively, up to a maximum value of 4,000 psf. Recommended allowable bearing values include both dead and live loads and may be increased by one-third for wind and seismic forces.

6.3.5 Lateral Resistance

Provided site grading is performed in accordance with the recommendations provided by the project geotechnical consultant, a passive earth pressure of 300 pounds per square foot per foot of depth up to a maximum value of 1,500 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. For footings facing descending slopes, 50% of this capacity should be used. A coefficient of friction of 0.33 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 cannot be performed, the above-noted values should be decreased by 50%.

The above values are based on foundations placed directly against compacted fill. In the case where footing sides are formed, all backfill against the foundations should be compacted to at least 90 percent of the laboratory standard.

6.3.6 Footings and Slabs on Grade

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

All spread footings used to support columns should have a minimum width of 24 inches and minimum embedment of 12 inches below lowest adjacent grade. All spread footings should be tied in both directions with a grade beam having a minimum depth and width of 12 inches. The grade beams should be reinforced with a minimum of one No. 4 bar on top and one No. 4 bar on the bottom. Reinforcing of the grade beams should hook into the footings.

Slabs on grade should have a minimum thickness of 4 inches and be reinforced with a minimum of No. 3 bars spaced at 18 inches center to center. Slabs on grade in habitable structures should be hooked to the underlying grade beams on a minimum spacing of 24 inches or poured monolithically with the grade beams.

Interior grade beams as required by the WRI method should be provided in both directions at a maximum spacing of 22 feet. Design of the slab in accordance with the WRI method may use an effective PI of 20. This value already accounts for the factors for ground slope and over-consolidation.

All slabs on grade that may have moisture sensitive coverings should be underlain with a minimum of 10-mil moisture vapor retarder conforming to ASTM E 1745, Class A. A minimum of two (2) inches of clean sand having a sand equivalent (SE) of at least 30 should be placed under the membrane. One inch of this sand may be placed over the vapor barrier to aid in the uniform curing of the slab if preferred. This vapor barrier system is anticipated to be suitable for most flooring finishes that can accommodate some vapor emissions. However, this system may emit more than 4 pounds of water per 1000 sq. ft. and therefore, may not be suitable for all flooring finishes. Additional steps should be taken if such vapor emission levels are too high for anticipated flooring finishes.

Prior to placing concrete, the subgrade below all floor slab areas should be moisture-conditioned to achieve a moisture content that is at least 110 percent of the optimum moisture content. This moisture content should be maintained a minimum depth of 12 inches below the bottoms of the slabs.

6.3.7 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. Final wall designs specific to the site development should be provided to project geotechnical consultant 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

Provided site grading is performed as recommended herein, the values for bearing and lateral resistance provided in Sections 6.3.4 and 6.3.5 may be utilized in design of retaining and screen walls. The coefficient of friction should not be applied to portions of the footing in front of keyways used for passive resistance.

The above values are based on footings placed directly against properly compacted fill. 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. The passive pressure used for lateral bearing should be reduced by 50% for walls along the property lines where lateral removals cannot be performed.

6.4.3 Earth Pressures

Static and seismic earth pressures for level and 2:1 (H:V) backfill conditions are provided in Table 6.2. Seismic earth pressures provided herein are based on the method provided by Seed & Whitman (1970) using a peak ground acceleration (PGA) of 0.43g for 10% probability of exceedance in 50 years. As indicated in Section 1803.5.12 of the 2019 CBC, retaining walls supporting 6 feet of backfill or less are not required to be designed for seismic earth pressures. The values provided in the following table do not consider hydrostatic pressure. Retaining walls should also be designed to support adjacent surcharge loads imposed by other nearby footings or traffic loads in addition to the earth pressure.

6.4.4 Drainage and Moisture-Proofing

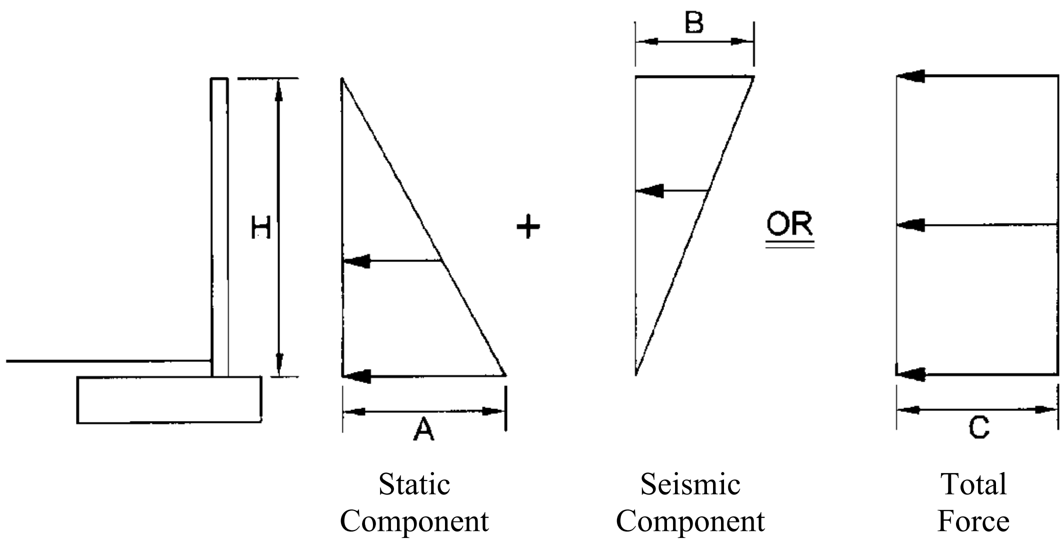
Retaining walls should be constructed with a perforated pipe and gravel subdrain to prevent entrapment of water in the backfill. The perforated pipe should consist of 4-inch-diameter, ABS SDR-35 or PVC Schedule 40 with the perforations laid down. The pipe should be embedded in ¾- to 1½-inch open-graded gravel wrapped in filter fabric. The gravel should be at least one foot wide and extend at least one foot up the wall above the footing and drainage outlet. Drainage gravel and piping should not be placed below outlets and weepholes. Filter fabric should consist of Mirafi 140N, or equal. Outlet pipes should be directed to positive drainage devices.

The use of weepholes may be considered in locations where aesthetic issues from potential nuisance water are not a concern. Weepholes should be 2 inches in diameter and provided at least every 6 feet on center. Where weepholes are used, perforated pipe may be omitted from the gravel subdrain.

Retaining walls supporting backfill should also be coated with a moisture-proofing compound or covered with such material to inhibit infiltration of moisture through the walls. Moisture-proofing material should cover any portion of the back of wall that will be in contact with soil and should lap over and onto the top of footing. A drainage panel should be provided between the soil backfill and water proofing. The panel should extend from the top of the backdrain gravel up to within 12 inches of finish grade. The top of footing should be finished smooth with a trowel to inhibit the infiltration of water through the wall. The project structural engineer should provide specific recommendations for moisture-proofing, water stops, and joint details.

TABLE 6.2

SEISMIC EARTH PRESSURES
Pressure Diagram



Earth Pressure Values
Walls Up to 10 Feet in Height

Value	Backfill Condition	
	Level	2H:1V Slope
A	42H	68H
B	13H	13H
C	28H	41H

Note:

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

6.4.5 Footing Reinforcement and Wall Jointing

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

All free-standing, exterior site walls should be provided with cold joints through the masonry block section at horizontal spacing generally not exceeding 20 feet. The joints should not extend through the footing. Retaining walls that are integral to the building should be provided joints based on recommendations by the structural engineer.

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 Wall Backfill

Onsite soils may be used for backfill behind retaining walls. The project geotechnical consultant should approve the backfill used for retaining walls. Wall backfill should be thoroughly moistened to provide moisture contents slightly over 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.

6.5 EXTERIOR FLATWORK

Concrete sidewalks, patios, and similar flatwork should be a nominal 4 inches thick and provided with saw cuts or expansion joints at spacing no greater than 10 feet in each direction. Special jointing details should be provided in areas of block-outs, notches, or other irregularities to avoid cracking at points of high stress.

Drainage from flatwork areas should be directed to local area drains and/or other appropriate collection devices designed to carry runoff water to the street or other approved drainage structures. The concrete flatwork should also be sloped at a minimum gradient of 2% away from building foundations and masonry walls.

Subgrade soils below flatwork areas should be thoroughly moistened prior to placing concrete. The moisture content of the soils should be at least 110 percent of the optimum moisture content and penetrate to a depth of approximately 12 inches into the subgrade. Flooding or ponding of the subgrade is not recommended. Moisture conditioning should be achieved by a light application of water to the subgrade just prior to pouring concrete.

6.6 CONCRETE MIX DESIGN AND CORROSION

Laboratory testing of existing near-surface soils for soluble sulfate content indicates soluble sulfate concentration at 0.03% or smaller. We recommend following the procedures provided in ACI 318, Section 4.3, Table 4.3.1 for Class S0 sulfate exposure. Upon completion of rough grading, an evaluation of as-graded conditions and further laboratory testing should be completed for the site to confirm or modify the recommendations provided in this section.

6.7 CORROSION

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

6.8 POST GRADING CONSIDERATIONS

6.8.1 Site Drainage and Irrigation

Positive drainage devices, such as sloping concrete flatwork, graded swales or area drains, should be provided around the new construction to collect and direct all surface water to suitable discharge areas. In general, the site should be graded to conform to the requirements of Section 1804.3 of the 2019 California Building Code. No rain or excess water should be directed toward or allowed to pond against structures such as walls, foundations, flatwork, etc.

Excessive irrigation water can be detrimental to the performance of the proposed site development. Water applied in excess of the needs of vegetation will tend to percolate into the ground. Such percolation can lead to nuisance seepage and shallow perched groundwater. Seepage can form on slope faces, on the faces of retaining walls, in streets, or other low-lying areas. These conditions could lead to adverse effects such as the formation of stagnant water that breeds insects, distress or damage of trees, surface erosion, slope instability, discoloration and salt buildup on wall faces, and premature failure of pavement. Excessive watering can also lead to elevated vapor emissions within buildings that can damage flooring finishes or lead to mold growth inside the home.

Key factors that can help mitigate the potential for adverse effects of overwatering include the judicious use of water for irrigation, use of irrigation systems that are appropriate for the type of vegetation and geometric configuration of the planted area, the use of soil amendments to enhance moisture retention, use of low-water demand vegetation, regular use of appropriate fertilizers, and seasonal adjustments of irrigation systems to match the water requirements of vegetation. Specific recommendations should be provided by a landscape architect or other knowledgeable professional.

6.8.2 Utility Trenches

Trench excavations should be constructed in accordance with the recommendations contained in Section 6.1.7 of this report. Trench excavations must also conform to the requirements of Cal/OSHA.

Trench backfill materials and compaction criteria should conform to the requirements of the local municipalities. As a minimum, utility trench backfill should be compacted to at least 90 percent of the laboratory standard. 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. 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.

6.9 PRELIMINARY PAVEMENT DESIGN

6.9.1 Preliminary Structural Sections

Based on the soil conditions present at the site and estimated traffic indices, preliminary pavement sections are provided in Table 6.3 below. An assumed “R-value” of 25 was used for the near-surface soil in this preliminary pavement design. The sections provided below are for planning purposes only and should be re-evaluated subsequent to site grading. Final pavement sections should be based on actual R-value testing of in-place soils and analysis of anticipated traffic.

TABLE 6.3
PRELIMINARY PAVEMENT STRUCTURAL SECTIONS

Location	Traffic Index	AC (inches)	Paver Thickness (mm)	Portland Cement Concrete (inches)	AB (inches)
Drive Aisles	6.5	3.0 4.0	--	--	11 9.0
		--	80	--	14.0
		---	--	8.0	---
Parking Stalls	--	3.0		---	5.0

6.9.2 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.

6.9.3 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.

6.9.4 Asphaltic Concrete

Paving asphalt should be PG 64-10. Asphaltic concrete materials should conform to Section 203-6 of the Greenbook and construction should conform to Section 302 of the Greenbook. Where traffic will traverse over cold joints in asphaltic concrete such as against concrete ribbon gutters and concrete paver sections, the asphaltic concrete section should be thickened by 1 additional inch from the values indicated in the above Table 6.3 within 2 feet of cold joints.

6.9.5 Concrete Pavers

Concrete pavers should conform to the requirements of ASTM C 936. Construction of the pavers, including bedding sand, should follow manufacturer's specifications. Typical thickness of bedding sand is about 1 inch. The gradation of bedding sand should meet the requirement in Table 6.4.

TABLE 6.4
Gradation for Sand Bedding

Sieve Size	Percent Passing
⅜”	100
No. 4	95 - 100
No. 8	80 - 100
No. 16	50 - 85
No. 30	25 - 60
No. 50	5 - 30
No. 100	0 - 10
No. 200	0 - 1

Construction of edge restraints should also follow manufacturer’s specifications. As a minimum, restraints should be provided along the perimeter of concrete pavers and where there is a change in the paving materials. The proposed concrete bands should extend to the bottom of the base course underlying the concrete pavers. Portland cement concrete used to construct concrete bands should conform to Section 201 of the Greenbook and should have a minimum compressive strength of 2500 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. However, cold joints should be provided with dowels or keyways are recommended by PCA.

6.9.6 Portland Cement Concrete (PCC)

Portland cement concrete used to construct concrete paving should conform to Section 201 of the Greenbook and should have a minimum compressive strength of 3,500 pounds per square inch (psi) at 28 days. Reinforcement and jointing of concrete pavement sections should be designed according to the minimum recommendations provided by the Portland Cement Association (PCA). For rigid pavement, transverse and longitudinal contraction joints should be provided at spacing no greater than 15 feet. Score joints may be constructed by saw cutting to a depth of ¼ of the slab thickness. Expansion/cold joints may be used in lieu of score joints. Such joints should be properly sealed. Where traffic will traverse over cold joints or edges of concrete paving, the edges should be thickened by 20% of the design thickness toward the edge over a horizontal distance of 5 feet.

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 8 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.

6.10 PLAN REVIEW AND CONSTRUCTION SERVICES

We recommend *Albus-Keefe & Associates, Inc*. be engaged to review any future development plans, including revisions to the 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 described herein and in other literature are believed representative of the total project area, and the conclusions contained in this report are presented on that basis. However, soil 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 prior to and 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 **VenturePoint Inc. as well as Signal Hill Petroleum** to assist the project consultants in the 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.

Respectfully submitted,

ALBUS-KEEFE & ASSOCIATES, INC

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REFERENCES

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California Department of Conservation, Division of Mines and Geology, Seismic Hazard Report 028, "Seismic Hazard Zone Report for the Long Beach 7.5-Minute Quadrangle, Orange County, California", 1998.

California Department of Conservation, Division of Mines and Geology, Special Publication 117A "Guidelines for Evaluating and Mitigating Seismic Hazards in California", 2008.

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

Reports

Albus-Keefe & Associates, Inc., Geotechnical Investigation, Proposed Dog Park and parking Lot Improvements, 3100 California Avenue, City of Signal Hill, California, November 8, 2016, (J.N. 2541.00)

Albus-Keefe & Associates, Inc., Preliminary Geotechnical Investigation for Proposed Retail/Restaurant Development, Pad A Parcel, Signal Hill Gateway Project, Northwest Corner of East Spring Street and California Avenue, Signal Hill, California, November 4, 2013, (J.N. 2218.00)

Albus-Keefe & Associates, Inc., Preliminary Geotechnical Investigation, Proposed Applebee's Restaurant, Northwest of Intersection of East Spring Street and California Avenue, Signal Hill, California, December 13, 2012, (J.N. 2127.00)

Albus-Keefe & Associates, Inc., Preliminary Geotechnical Investigation, Proposed Self Storage Facility, Field's North Trust Property, Northwest Corner of 32nd Street and California Avenue, City of Signal Hill, California, September 18, 2001, (J.N. 1137.00)

Giles Engineering Associates, Inc., Updated Geotechnical Engineering Exploration and Analysis, Proposed Chick-fil-A Restaurant #3371, 405 @ Atlantic FSU, 3290 Atlantic Boulevard, Long Beach, California, prepared for Chick-fil-A, Inc., October 18, 2016, (Project No, 2G-1308002-1)

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Plans

ALTA/NSPS Land Title Survey, 860 E. 33rd Street, In the City of Signal Hill, County of Los Angeles, State of California, prepared by Ware Malcomb Sheets 1 and 2, Job No.: WMA19076

Conceptual Design, Signal Hill Retail Development, 950 E 33rd Street, Signal Hill, CA 90755, prepared for VenturePoint, prepared by Ware Malcomb, Sheet 2, dated December 16, 2016, Job No.: IRV19-0211

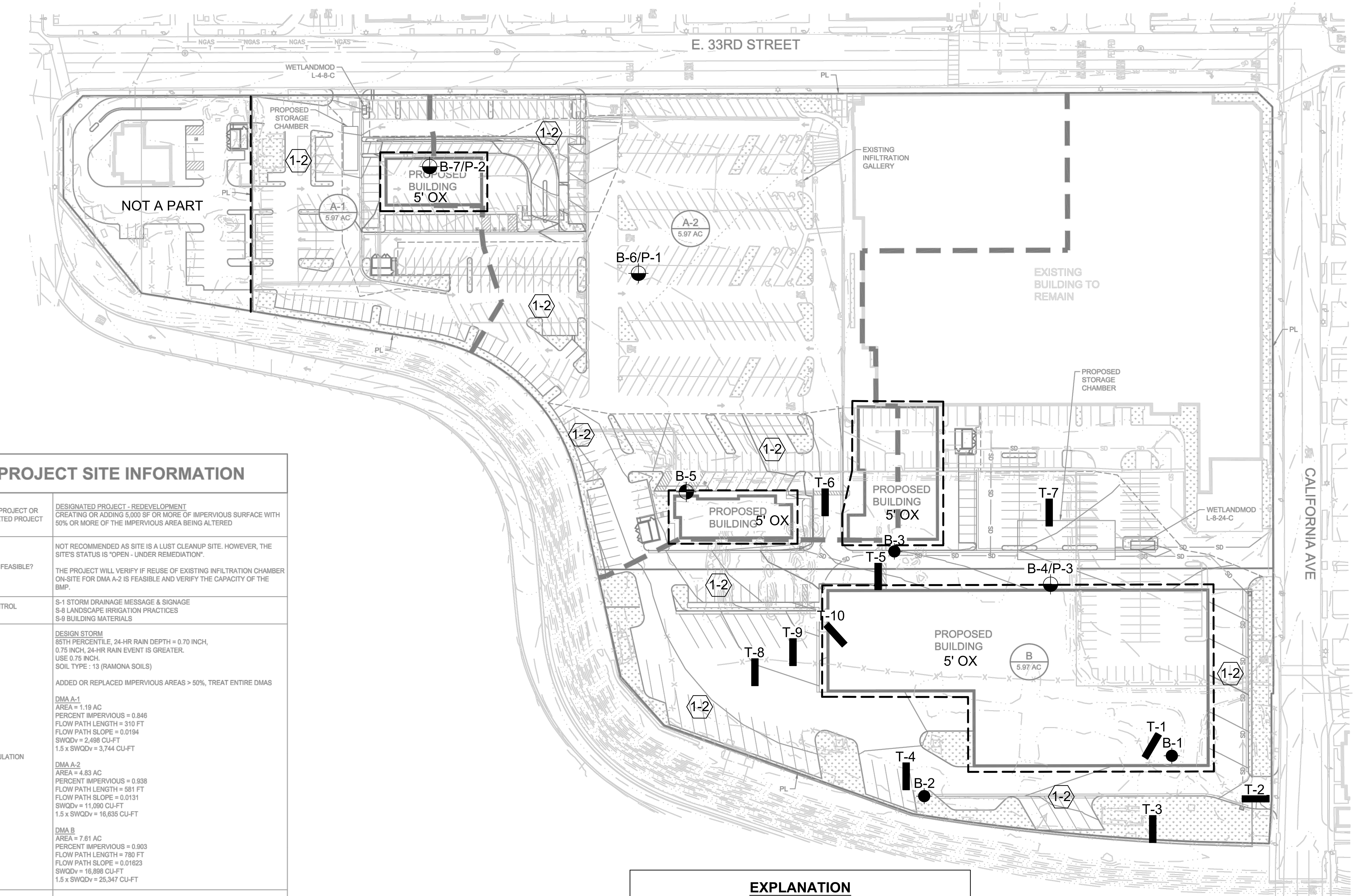
Preliminary Grading Plan and Utility Plan, Venture Point, 950 E. 33rd Street, Signal Hill, CA, prepared for VenturePoint, prepared by Ware Malcomb, Sheets 1 through 8, dated December 19, 2019, Job No.: IRV19-0211

Preliminary LID Plan, prepared for VenturePoint, prepared by Ware Malcomb, Sheet 7, dated December 19, 2019, Job No. IRV19-0211

Aerial Photographs

Source	Date Flown	Flight No.	Photo No.
Continental	11/19/53	14K	94 & 95
Continental	1/31/70	61-7	177 &178
Continental	5/12/79	FC-LA	11-126, -127, & -28
Continental	4/1981	ORA	4-1 & -2
Continental	1/27/86	F	351, 352, & 353
Continental	7/7/88		19205 & 10206
Continental	7/7/88		19210 & 19211
Continental	6/12/90	C84-15	-7 & -8
Continental	1/29/92	C85-7	-29, -30, & 31
Continental	6/9/93	C93-13	-164, 165, & -166
Continental	1/29/95	C103-35	-126, 127, & 128
Continental	10/15/97	C117-35	-239 & -240
Continental	2/24/99	C134-35	-132 & -133
UCSB	1928	C-300	M-156
UCSB	1947	C-11351	7-67
UCSB	1947	C-11351	7-68
UCSB	1956	C-22555	26-34
UCSB	1956	C-22555	26-35
UCSB	1960	C-23870	2131
UCSB	1960	C-23870	2130
UCSB	1968	TG-2400	2-10

CAUTION: IF THIS SHEET IS NOT 24"x36" IT IS A REDUCED PRINT



PROJECT SITE INFORMATION

DESIGNATED PROJECT OR NON-DESIGNATED PROJECT	DESIGNATED PROJECT - REDEVELOPMENT CREATING OR ADDING 5,000 SF OR MORE OF IMPERVIOUS SURFACE WITH 50% OR MORE OF THE IMPERVIOUS AREA BEING ALTERED
INFILTRATION FEASIBLE?	NOT RECOMMENDED AS SITE IS A LUST CLEANUP SITE. HOWEVER, THE SITE'S STATUS IS "OPEN - UNDER REMEDIATION". THE PROJECT WILL VERIFY IF REUSE OF EXISTING INFILTRATION CHAMBER ON-SITE FOR DMA A-2 IS FEASIBLE AND VERIFY THE CAPACITY OF THE BMP.
SOURCE CONTROL MEASURES	S-1 STORM DRAINAGE MESSAGE & SIGNAGE S-8 LANDSCAPE IRRIGATION PRACTICES S-9 BUILDING MATERIALS
SWQDv CALCULATION	DESIGN STORM 85TH PERCENTILE, 24-HR RAIN DEPTH = 0.70 INCH, 0.75 INCH, 24-HR RAIN EVENT IS GREATER. USE 0.75 INCH. SOIL TYPE : 13 (RAMONA SOILS) ADDED OR REPLACED IMPERVIOUS AREAS > 50%, TREAT ENTIRE DMAS DMA A-1 AREA = 1.19 AC PERCENT IMPERVIOUS = 0.846 FLOW PATH LENGTH = 310 FT FLOW PATH SLOPE = 0.0194 SWQDv = 2,498 CU-FT 1.5 x SWQDv = 3,744 CU-FT DMA A-2 AREA = 4.83 AC PERCENT IMPERVIOUS = 0.938 FLOW PATH LENGTH = 581 FT FLOW PATH SLOPE = 0.0131 SWQDv = 11,090 CU-FT 1.5 x SWQDv = 16,635 CU-FT DMA B AREA = 7.61 AC PERCENT IMPERVIOUS = 0.903 FLOW PATH LENGTH = 780 FT FLOW PATH SLOPE = 0.01623 SWQDv = 16,898 CU-FT 1.5 x SWQDv = 25,347 CU-FT
HARVEST AND USE FEASIBLE?	NO
STORMWATER QUALITY CONTROL MEAUSRES	PROPRIETARY TREATMENT CONTROL MEASURES DMA A-1 : WETLANDMOD L-4-8-C TREATMENT CAPACITY @48-HR DRAWDOWN = 30,216 CU-FT DMA B : WETLAND MOD L-8-24-C TREATMENT CAPACITY @ 48-HR DRAWDOWN = 5,036 CU-FT RETENTION-BASED STORMWATER QUALITY CONTROL MEASURES DMA A-2: REUSE OF EXISTING INFILTRATION GALLERY INFILTRATION CAPACITY : TO BE VERIFIED
HYDROMODIFICATION REQUIRED?	STORMWATER FROM PROJECT SITE DISCHARGES INTO IMPROVED DRAINAGE SYSTEMS. HYDROMODIFICATION CONTROL BMPS ARE NOT REQUIRED.

LEGEND

DMA BOUNDARY

FLOW PATH LENGTH

EXPLANATION
(Locations Approximate)

- Exploratory Boring (Albus-Keefe, this report)
- Exploratory Boring and Percolation Test (Albus-Keefe, this report)
- Exploratory Boring (Albus-Keefe, 09/18/2001, J.N. 1137.00)
- Exploratory Trench (Albus-Keefe, 09/18/2001, J.N. 1137.00)
- 1-2 - Anticipated Depth of Removals in pavement Areas (in feet)
- 5' OX - Recommended Over-Excavation of Building Pad, Extending 5' Beyond Footprint (to be based on final plans)

ALBUS-KEEFE & ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

GEOTECHNICAL MAP

Job No.: 2871.00 | Date: 02/25/20 | Plate: 1

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REGISTERED PROFESSIONAL ENGINEER
LUCAS A. CORSBIE
No. 72588
CIVIL
STATE OF CALIFORNIA
Jan 10, 2020

FOR AND ON BEHALF OF WARE MALCOMB

VENTURE POINT

PRELIMINARY LID PLAN

NO.	DATE	REMARKS

JOB NO.:	IRV19-0211
PA / PM:	BS
DRAWN BY:	JP
DATE:	12/19/2019
PLOT DATE:	

SHEET
LID-1
Sheet 7 of 8

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			Laboratory Tests		
				Blows Per Foot	Core	Bulk	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests

EXPLANATION

Solid lines separate geologic units and/or material types.

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.

Vertical Lines in core column represents Shelby sampler.

Solid black rectangle in Bulk column respresents large bag sample.

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

Depth scale: 5, 10, 15, 20

Albus-Keefe & Associates, Inc.

EXPLORATION LOG

Project: Field's North Trust Property	Location: B-4
Address: 950 E 33rd St, Signal Hill, CA	Elevation: 94
Job Number: 2870.00 · Client: Signal Hill Petroleum	Date: 1/22/2020
Drill Method: Hollow-Stem Auger · Driving Weight: 140 lbs / 30 in	Logged By: SD

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
		BEDROCK - LAKEWOOD FORMATION (QIw) Silty Sandstone : Mottled grayish brown, reddish brown, and brown, damp, very dense, fine grained sand, rootlets, black stainings							Max EI SO4 ATT pH Resist Ch
				94			6.1	126.9	
		Clayey Sandstone : Mottled reddish brown and grayish brown, damp, very dense, fine grained sand, trace coarse grained sand, iron oxide stainings, increased sand content toward sampler tip		75			6.7	119.2	
5									
		@ 6 ft, dense, mica present, caliche		54			10.2	122.1	
		Sandstone/ Silty Sandstone : Light brown, damp, dense, fine grained sand, rootlets, pin-hole pores							
10				40			12.8	109.9	
		Sandstone : Light gray, moist, dense, fine grained sand, iron oxide stainings							
15				23					SA

Albus-Keefe & Associates, Inc.

Plate A-2

EXPLORATION LOG

Project: Field's North Trust Property		Location: B-4
Address: 950 E 33rd St, Signal Hill, CA		Elevation: 94
Job Number: 2870.00	Client: Signal Hill Petroleum	Date: 1/22/2020
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: SD

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
				30					
25		@ 25 ft, increased iron oxide stainings		33					
		End of boring at depth of 26.5 feet. No groundwater encountered. Backfilled with soil cuttings.							

EXPLORATION LOG

Project: California Retail Center		Location: B-5
Address: 950 E 33rd St, Signal Hill, CA		Elevation: 94
Job Number: 2871.00	Client: Venture Point	Date: 1/22/2020
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: SD

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
		BEDROCK - LAKEWOOD FORMATION (QIw)							
		Silty Sandstone : Mottled brown and reddish brown, damp, very dense, fine grained sand, trace fine to medium gravel, iron oxide stainings		50			5.1	120.6	
5		@ 4 ft, dense, some clay nodules, some fine to coarse gravel, some red stainings		44			6.6	117.5	Consol
		Clayey Sandstone : Light brown, damp, dense, fine grained sand, trace pores, rootlet, increased medium sand toward sampler tip, some red spots		39			6.2	113.5	
10		@ 10 ft, increased clay content		49			16.5	111.8	
		Sandstone : Mottled gray and light brown, moist, dense, fine grained sand, some iron oxide stainings							
15				25					
20		@ 20 ft, very dense, no iron oxide stainings		39					

Albus-Keefe & Associates, Inc.

EXPLORATION LOG

Project: California Retail Center	Location: B-5
Address: 950 E 33rd St, Signal Hill, CA	Elevation: 94

Job Number: 2871.00	Client: Venture Point	Date: 1/22/2020
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: SD

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
				34					
		End of boring at depth of 26.5 feet. No Groundwater encountered. Backfilled with soil cuttings.							

EXPLORATION LOG

Project: California Retail Center	Location: B-6
Address: 950 E 33rd St, Signal Hill, CA	Elevation: 90
Job Number: 2871.00 / Client: Venture Point	Date: 1/22/2020
Drill Method: Hollow-Stem Auger / Driving Weight: 140 lbs / 30 in	Logged By: SD

Depth (feet)	Material Description	Water	Samples: Blows Per Foot	Moisture Content (%)	Dry Density (pcf)	Other Lab Tests
0	Asphalt (AC): 5.5" Asphalt					Max EI SO4 DS ATT pH Resist Ch
	ARTIFICIAL FILL (Af) Clayey Sand (SC): Reddish brown, moist, medium dense, fine grained sand, some dark brown spots, increased fine content in the tip of sampler		29	9.8	117.6	
	BEDROCK - LAKEWOOD FORMATION (QIw) Clayey Sandstone : Light brown, moist, dense, fine grained sand, trace pin-hole porous, red and brown spots		61	10.2	118.5	
5						
	Sandstone : light brown, damp, medium dense, fine grained sand, rootlet, some red and brown spots		14	4.3	97.1	Consol
10	@ 10 ft, mottled gray, white, and black, dry, very dense		59	1.1	100.1	SA
15	@ 15 ft, mottled light gray and light brown, dense, increased medium grained sand in the tip of sampler		33			

EXPLORATION LOG

Project: California Retail Center	Location: B-6
Address: 950 E 33rd St, Signal Hill, CA	Elevation: 90
Job Number: 2871.00 / Client: Venture Point	Date: 1/22/2020
Drill Method: Hollow-Stem Auger / Driving Weight: 140 lbs / 30 in	Logged By: SD

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		@ 20 ft, very dense		49					
25				44					
30		@ 30 ft, light brown, dense, fine to coarse grained sand		48					
		BEDROCK - San Pedro Formation (Qsp) ? Sandstone : Mottled light brown, gray, and reddish brown, damp, very dense, fine to coarse grained sand							
35				49					

EXPLORATION LOG

Project: California Retail Center	Location: B-6
Address: 950 E 33rd St, Signal Hill, CA	Elevation: 90
Job Number: 2871.00 / Client: Venture Point	Date: 1/22/2020
Drill Method: Hollow-Stem Auger / Driving Weight: 140 lbs / 30 in	Logged By: SD

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		@ 40 ft, some iron oxide stainings		53					
45		@ 45 ft, light brown, iron oxide layers, layer of reddish brown		56					
50		@ 50 ft, fine to medium grained sand		82					
		End of Boring at depth of 51.5 feet. No Groundwater encountered. Backfilled with soil cuttings and patched with cold asphalt.							

Albus-Keefe & Associates, Inc.

EXPLORATION LOG

Project: California Retail Center		Location: B-7
Address: 950 E 33rd St, Signal Hill, CA		Elevation: 89
Job Number: 2871.00	Client: Venture Point	Date: 1/22/2020
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: SD

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): 5" Asphalt							
		Crushed Aggregate Base (CAB): 4"							
		ARTIFICIAL FILL (Af) Clayey Sand (SC): Reddish brown, moist, medium dense, fine grained sand, iron oxide stainings		29			13.6	117.3	
		BEDROCK - LAKEWOOD FORMATION (Qlw) Clayey Sandstone : Mottled light brown and reddish brown, moist, dense, fine to medium grained sand, rootlet, black spots, mica present		41			16.1	113.5	Consol
5									
		@ 6 ft, light brown to brown, medium dense, fine grained sand, some dark brown spots		13			11.5	95.2	Consol
		Sandstone : Light gray, moist, dense, fine grained sand, mica present							
10				43			13	86.4	SA
15		@ 15 ft, some black spots		33					

EXPLORATION LOG

Project: California Retail Center		Location: B-7
Address: 950 E 33rd St, Signal Hill, CA		Elevation: 89
Job Number: 2871.00	Client: Venture Point	Date: 1/22/2020
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: SD

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		@ 20 ft, very dense, no black spots		37					
25		@ 25 ft, dense		28					
		End of boring at depth of 26.5 feet. No groundwater encountered. Backfilled with soil cuttings and patched with cold asphalt.							

APPENDIX B

LABORATORY TEST PROGRAM

LABORATORY TESTING PROGRAM

Soil Classification

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

In-Situ Moisture and Density

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

Laboratory Maximum Dry Density

Maximum dry density and optimum moisture content of onsite soils were determined for selected samples in general accordance with ASTM D1557. Pertinent test values are given on Table B.

Expansion Potential

Expansion index testing was performed on a selected sample. The test was performed in accordance with ASTM D4829. The test result is presented on Table B.

Consolidation

Consolidation tests were performed for selected soil samples in general conformance with ASTM D2435. Axial loads were applied in several increments to a laterally restrained 1-inch-high sample. Loads were applied in geometric progression by doubling the previous load, and the resulting deformations were recorded at selected time intervals. The test sample was inundated at a selected load to evaluate the effect of a sudden increase in moisture content (hydro-consolidation potential). Results of the tests are graphically presented on Plates B-4 through B-8.

Direct Shear

The Coulomb shear strength parameters, angle of internal friction and cohesion, were determined for a bulk sample obtained from one our borings. The tests were performed in general conformance with Test Method ASTM D 3080. The sample was remolded to 90 percent of maximum dry density and at the optimum moisture content. Three specimens were prepared for each test, artificially saturated, and then sheared under varied loads at an appropriate constant rate of strain. Results are graphically presented on Plate B-9.

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.

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.

Soluble Sulfate Analysis

A chemical analysis was performed on a selected sample to determine soluble sulfate content. Soil and Plant Laboratories, Inc. of Orange, California in accordance with Test Method No performed this test. California 417. Their test result is included on Table B-1.

Particle-Size Analyses

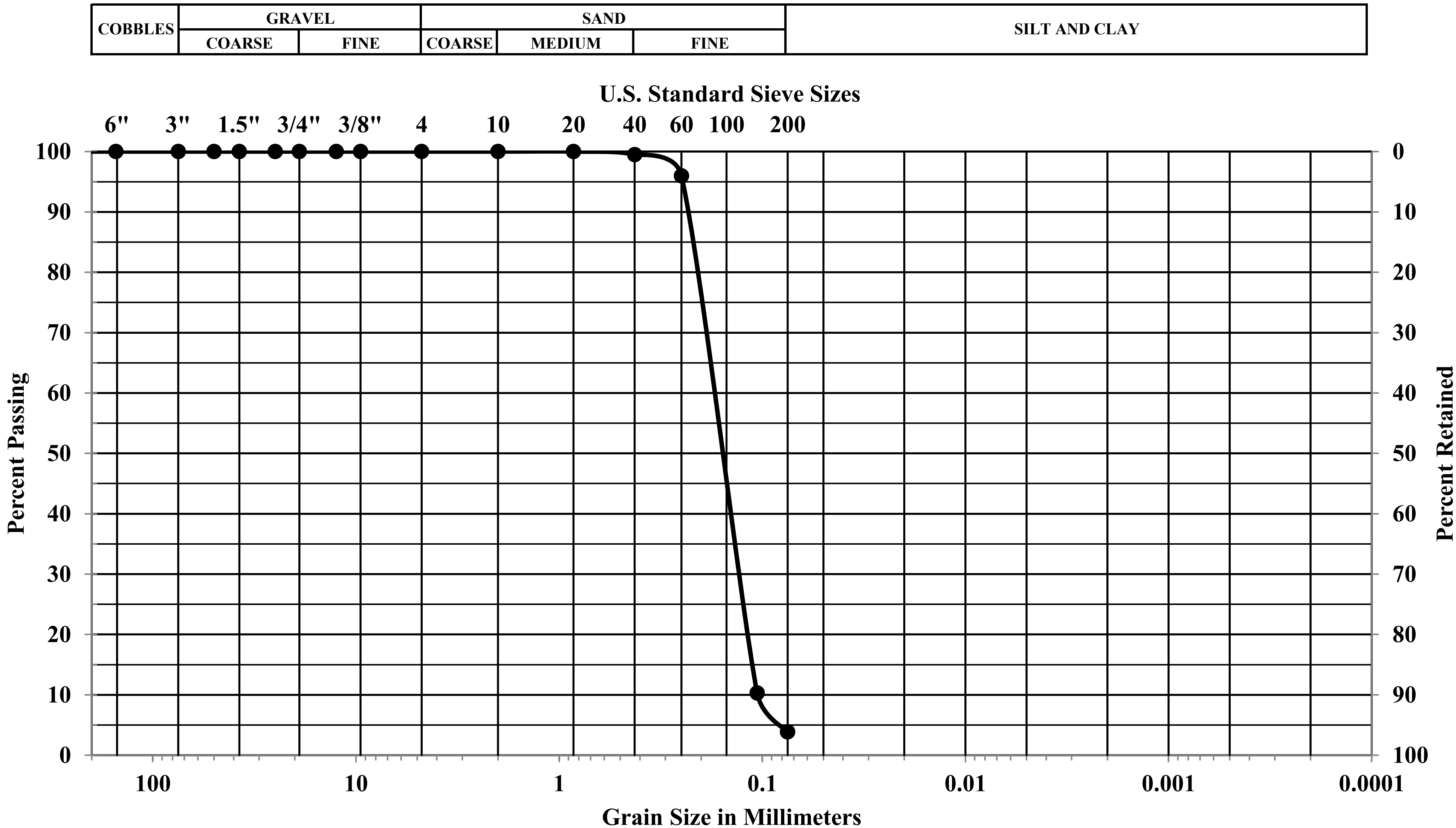
Particle-size analyses were performed on selected samples in accordance with ASTM D 422-63. The results are presented graphically on the attached Plates B-1 through B-3.

TABLE B-1
SUMMARY OF LABORATORY TEST RESULTS

Boring No.	Sample Depth (ft.)	Soil Description	Test Results	
B-2	3 - 5	Silty Sandstone	Maximum Dry Density: Optimum Moisture Content: Expansion Index: Expansion Potential: Soluble Sulfate Content:	125.0 pcf 10.5 % 28 Low 0.030 %
B-3	0 - 5	Silty Sandstone	Maximum Dry Density: Optimum Moisture Content: Expansion Index: Expansion Potential: Soluble Sulfate Content: Minimum Resistivity: pH: Chloride: Liquid Limit: Plastic Index:	130 pcf 8.5 % 2 Very Low 0.005 % 101,000 ohm-cm 7.55 24.5 ppm 19 4
B-6	0 - 5	Clayey Sandstone	Maximum Dry Density: Optimum Moisture Content: Expansion Index: Expansion Potential: Soluble Sulfate Content: Minimum Resistivity: pH: Chloride: Liquid Limit: Plastic Index:	127.0 pcf 10.5 % 42 Low 0% 1,400 ohm-cm 7.49 19.2 ppm 29 16

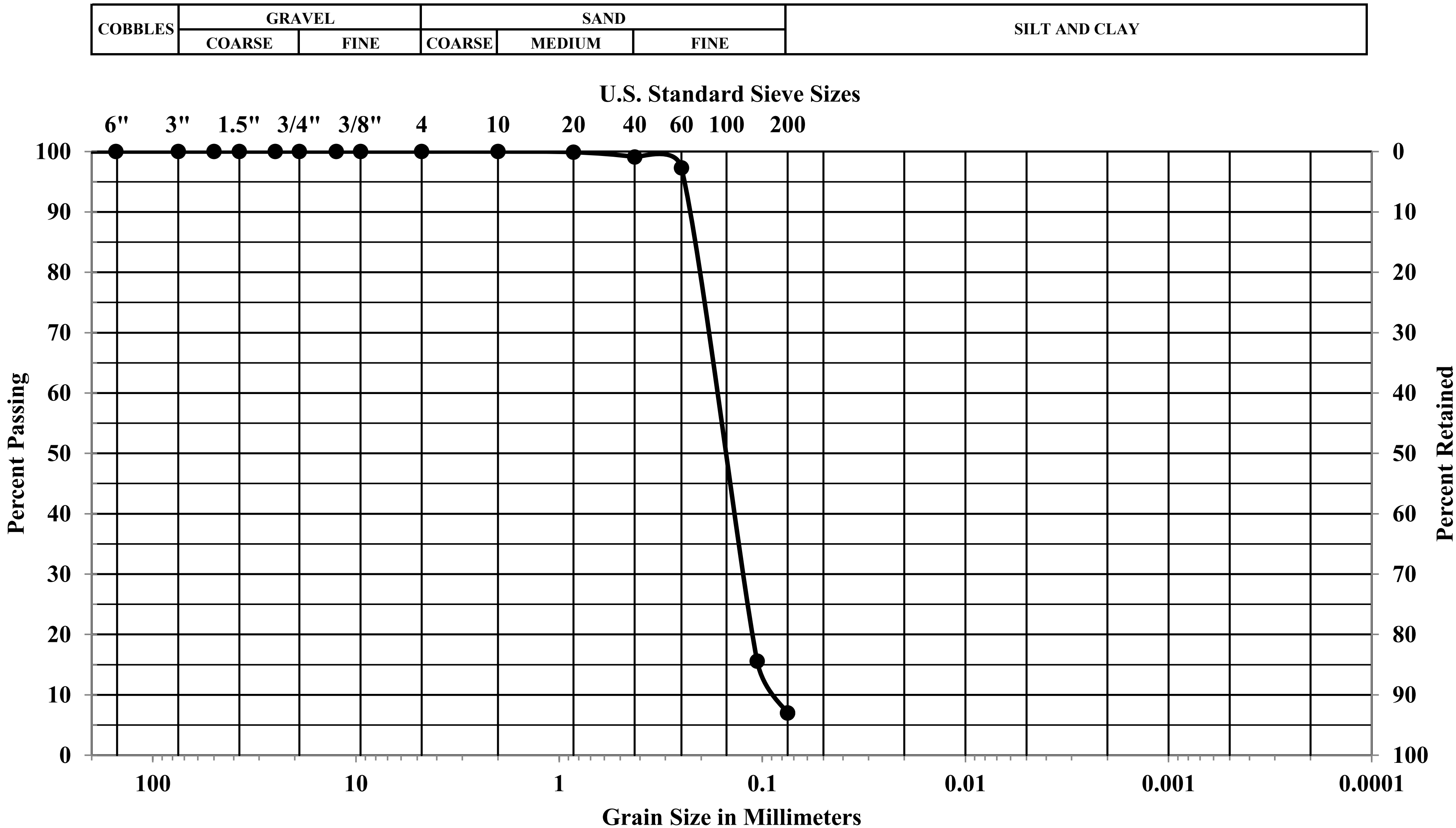
Note: Additional laboratory test results are provided on the boring logs provided in Appendix A.

GRAIN SIZE DISTRIBUTION



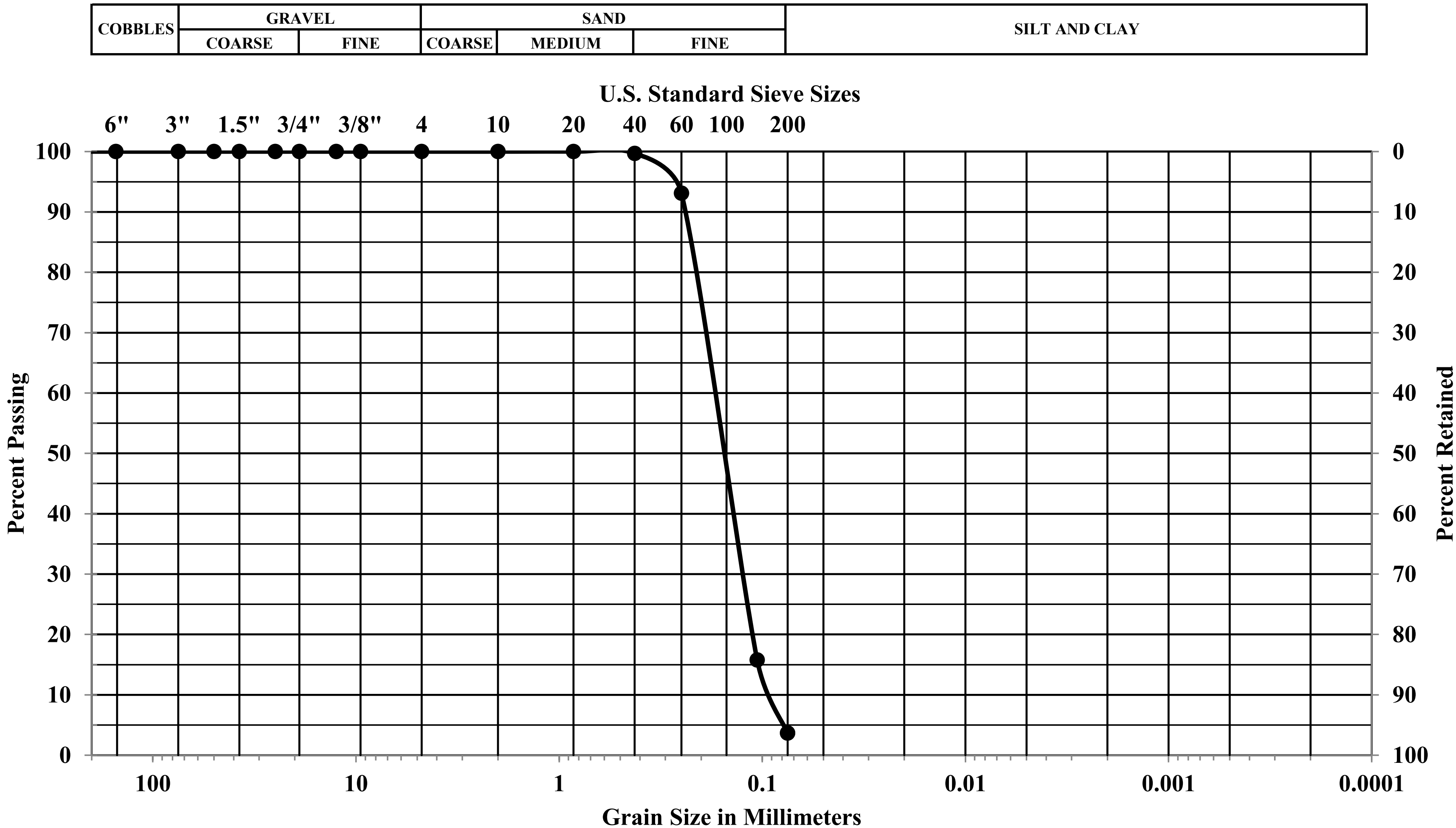
Job Number	Location	Depth	Description
2870.00	B-4	15	Sand (SP)

GRAIN SIZE DISTRIBUTION



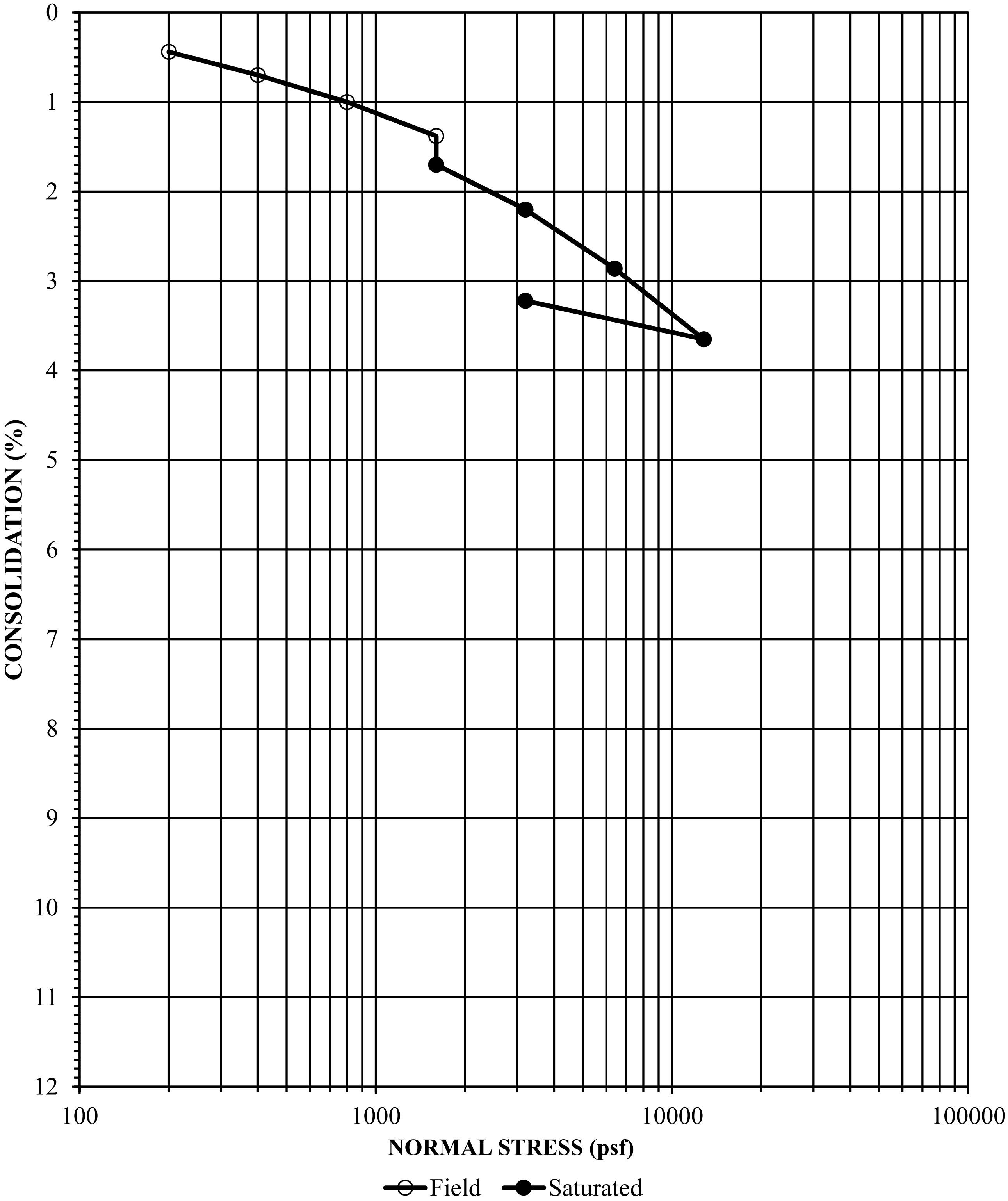
Job Number	Location	Depth	Description
2871.00	B-6	10	Poorly graded Sand with Silt (SP-SM)

GRAIN SIZE DISTRIBUTION



Job Number	Location	Depth	Description
2871.00	B-7	10	Sand (SP)

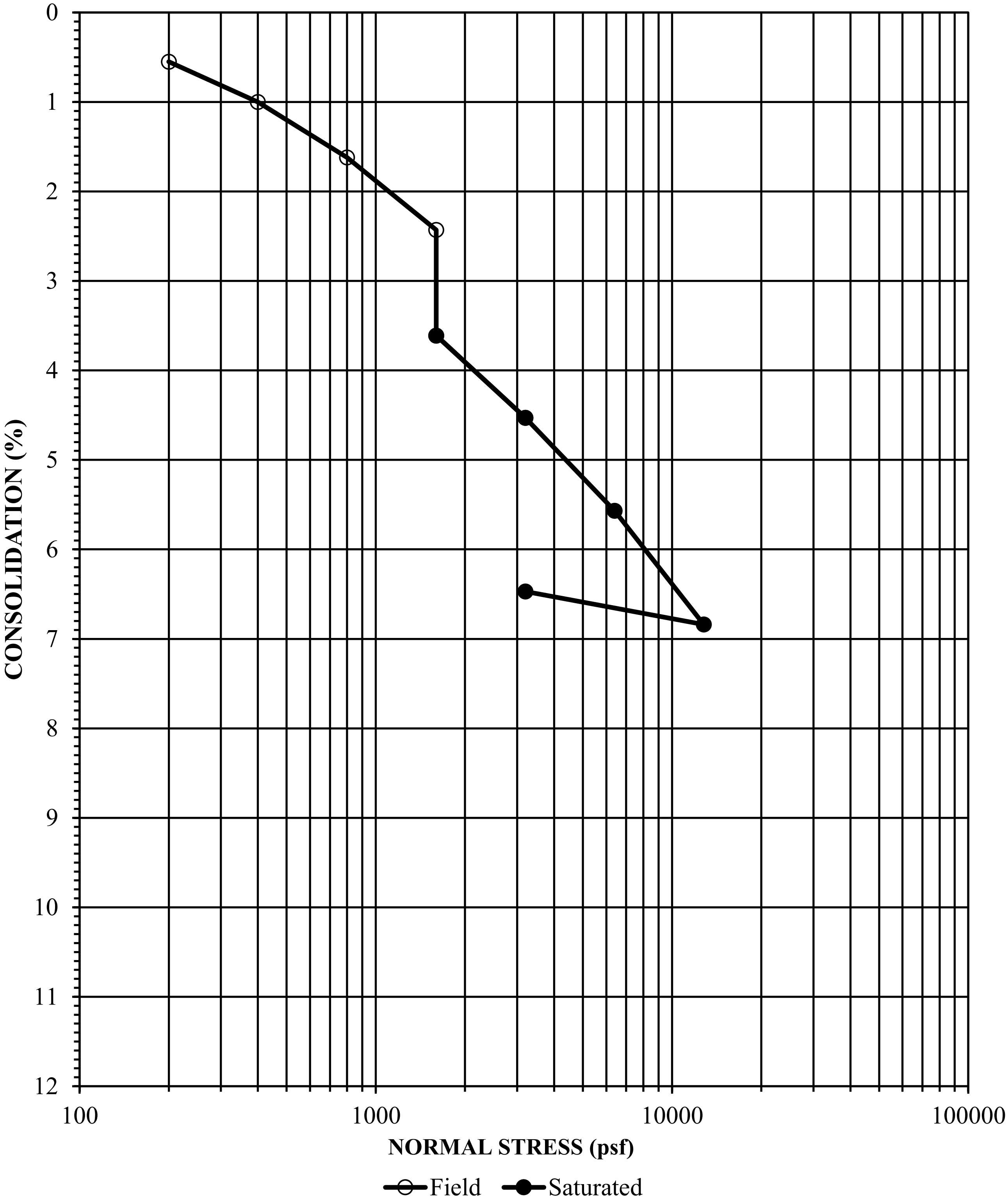
CONSOLIDATION



Job Number	Location	Depth	Description
2870.00	B-4	4	Clayey Sandstone ()

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
121.7	7	10.4

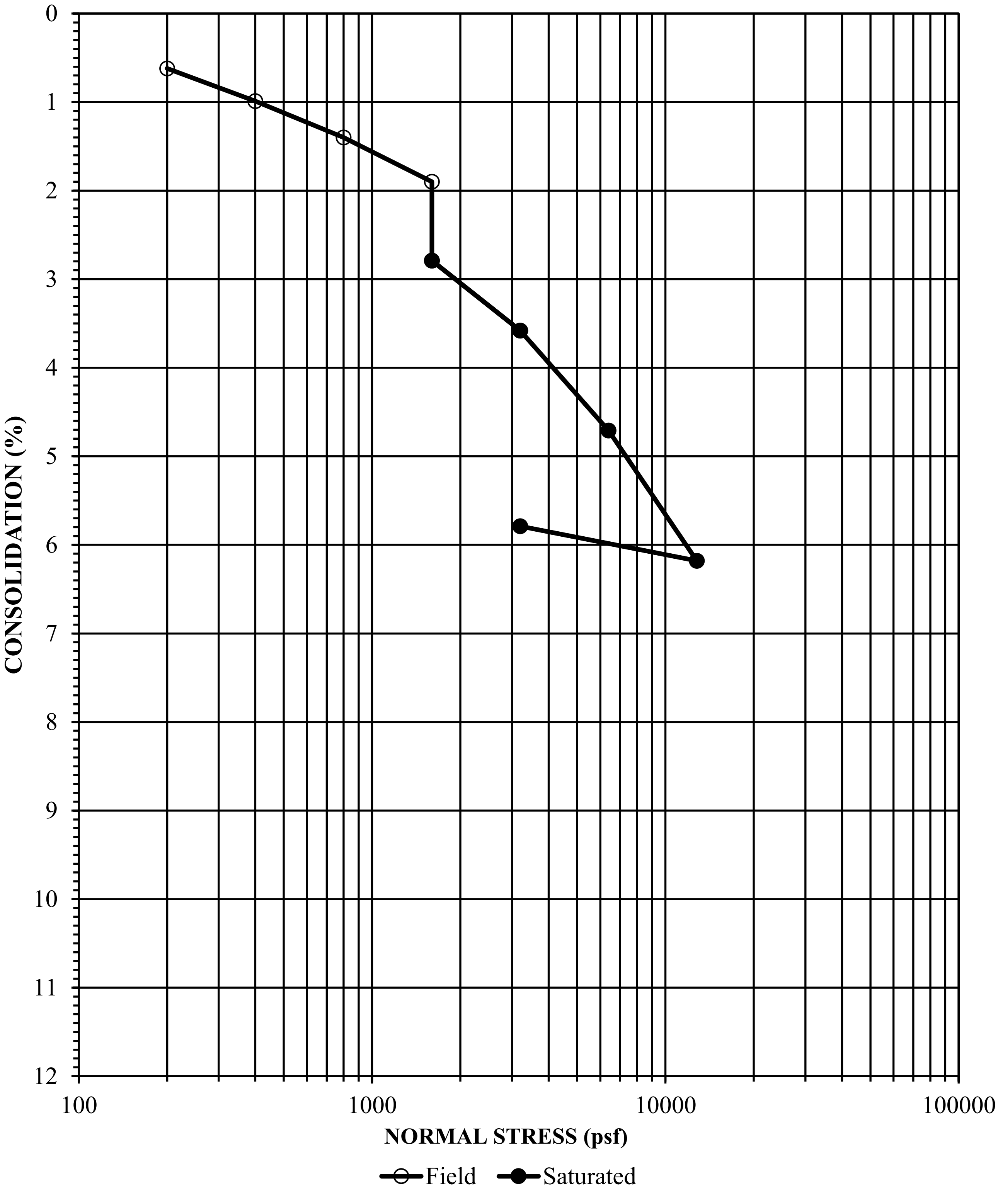
CONSOLIDATION



Job Number	Location	Depth	Description
2871.00	B-5	4	Silty Sandstone

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
110.4	21	12.1

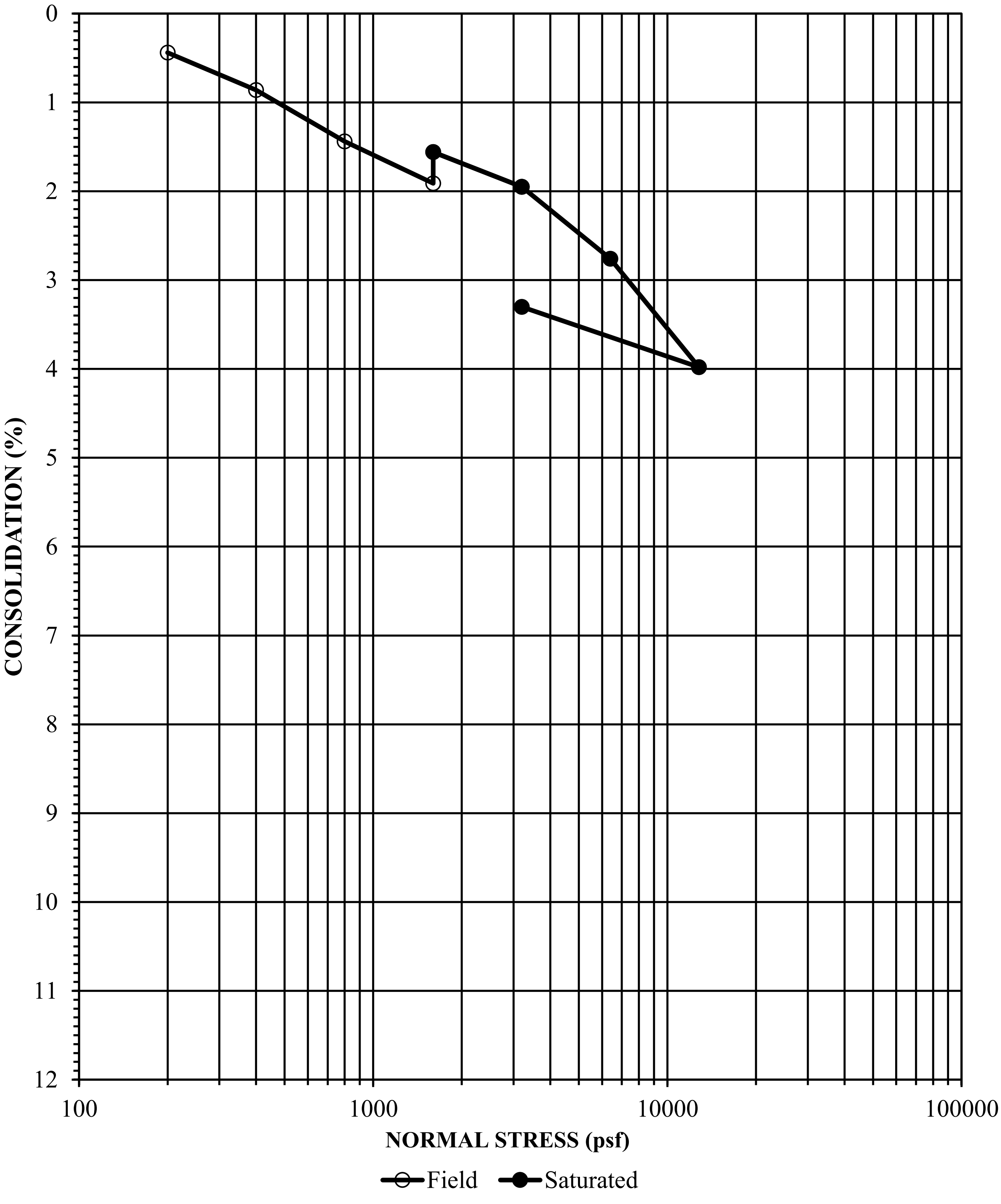
CONSOLIDATION



Job Number	Location	Depth	Description
2871.00	B-6	6	Sandstone

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
96.5	6.2	21.4

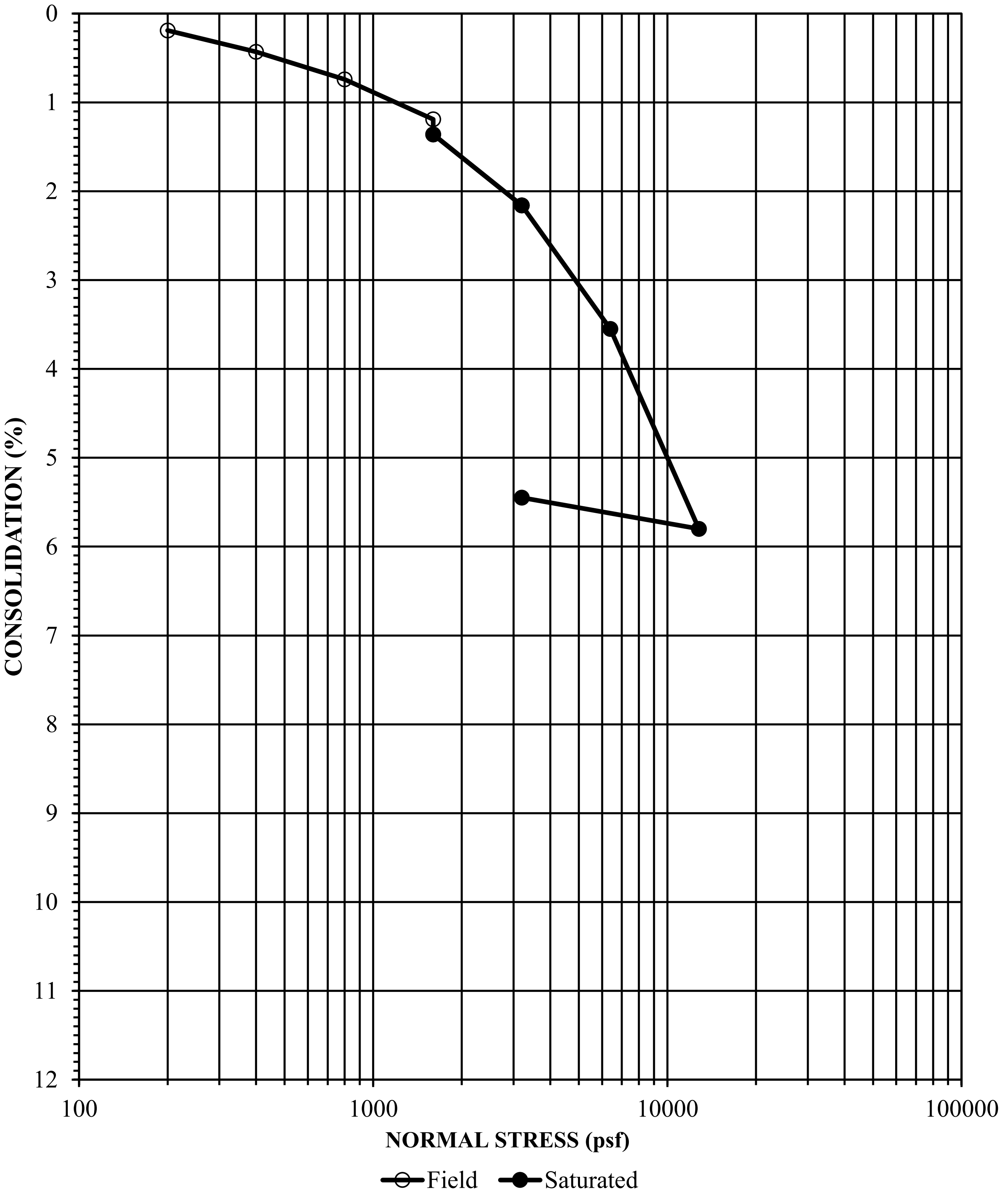
CONSOLIDATION



Job Number	Location	Depth	Description
2871.00	B-7	4	Clayey Sandstone

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
115.1	20.7	17.2

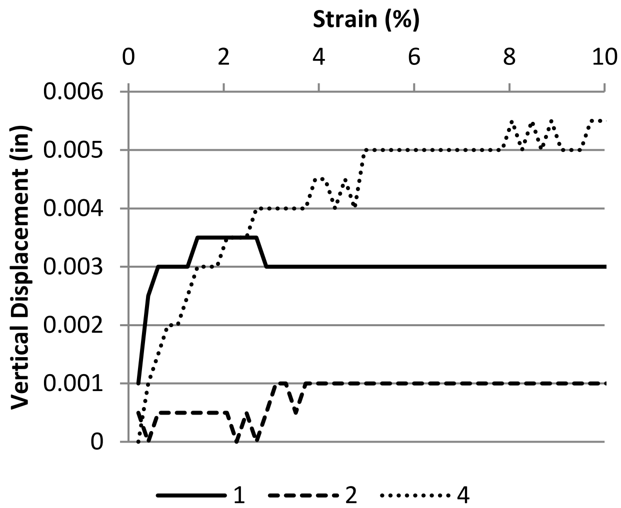
CONSOLIDATION

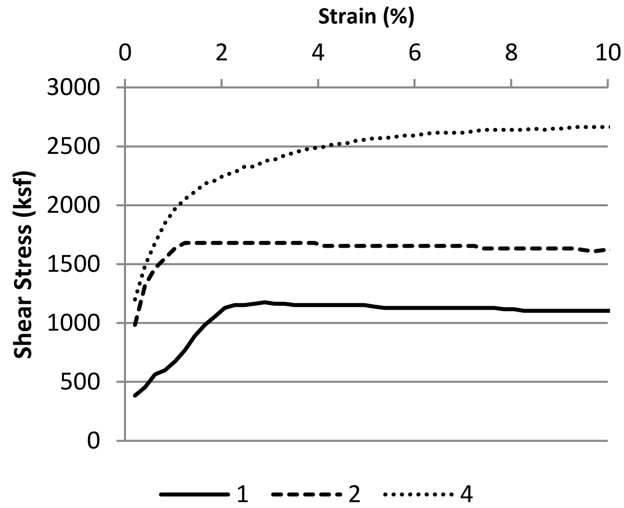


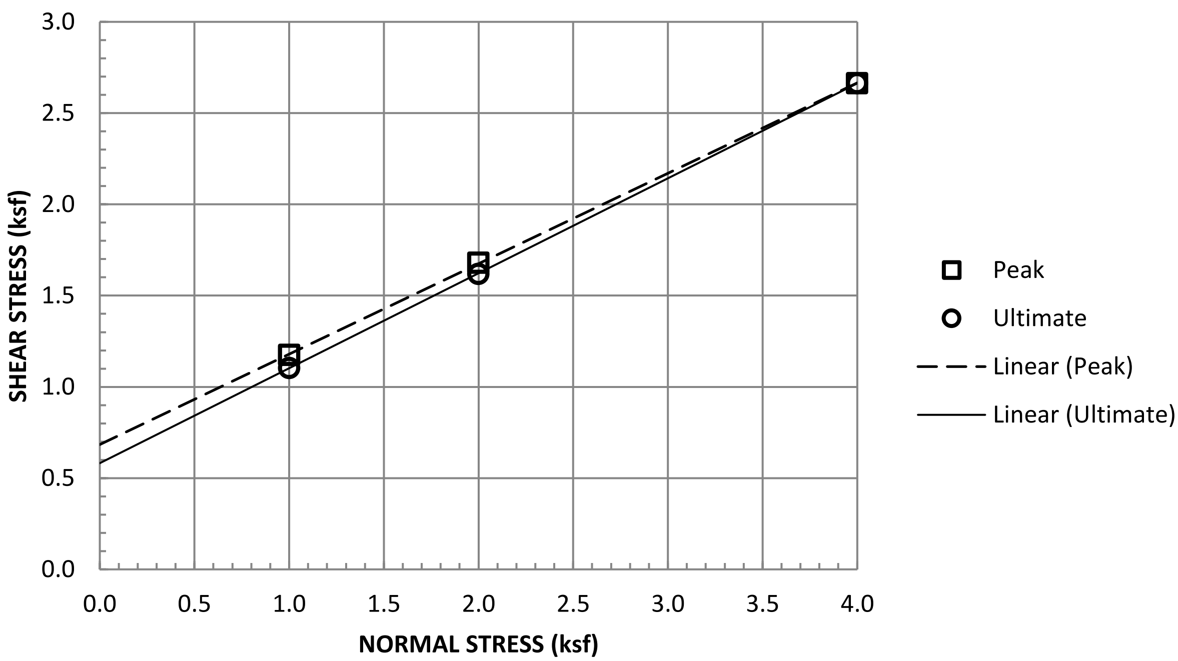
Job Number	Location	Depth	Description
2871.00	B-7	6	Clayey Sandstone

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
97.4	11.4	21.3

DIRECT SHEAR







Sample Type:	Remolded 90% of 127 @ 10.5%, Saturated		
Normal Stress (ksf)	1	2	4
Peak Shear Stress (ksf)	1.176	1.68	2.664
Peak Displacement (in)	0.004	0.001	0.006
Ultimate Shear Stress (ksf)	1.104	1.62	2.664
Ultimate Displacement (in)	0.25	0.25	0.25
Initial Dry Density (pcf)	114.3	114.3	114.3
Initial Moisture Content (%)	10.5	10.5	10.5
Final Moisture Content (%)	16.3	15.5	14.2
Strain Rate (in/min)	0.01		

Job Number	Location	Depth	Description
2871.00	B-6	0-5	Clayey Sand (SC)

APPENDIX C

PREVIOUS EXPLORATION LOGS AND LABORATORY TEST DATA
(AKA 9-18-2001)

BORING LOG NO. B-1

Client:	**Signal Hill Petroleum**	**Job No.: 1137.00**	
Project:	**Field's North Trust Property**	**Elevation:**	**Date: 8/31/01**
Location:	**Signal Hill, California**	**Total Depth: 20.0'**	**Log by: RSR**
Drill Rig Type:	**24" Bucket Auger** — **Driving Weights: 0-25 feet: 2500 lbs.**	**Sheet: 1 of 1**	Start: Stop:

Depth (Feet)	USCS Symbol	Material Description	Water	Samples: Blow Count	Samples: Ring	Samples: Bulk	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Other Lab Tests
	SM	**ARTIFICIAL FILL (Qaf):** Silty sand, fine grained, brown, dry to damp, medium dense, gravelly road base in upper 2 inches.		PUSH	■		107.5	8.9	
5		**BEDROCK: LAKEWOOD FORMATION (Qlw):** Silty sandstone, fine grained, brown to red-brown, damp, soft, soft to moderately hard, weathered, locally porous, some rootlets.		4	■		120.3	11.2	
				4	■		107.0	17.1	
10		@ 5.0 feet: Becomimg less porous, moderately hard, with some weathered siltstone clods.		4	■		97.1	3.6	
		@ 9.0 feeet: Sandstone with some traces of silt, fine grained, tan to light gray, damp, moderately hard, slightly micaceous.							
15				5	■		97.8	4.2	
		@ 18.0 feet: Heavy caving.							
20									
25		Total Depth: 20.0 feet Caving from 18.0 feet to 20.0 feet No Ground Water							

BORING LOG NO. B-2

Client:	**Signal Hill Petroleum**	**Job No.: 1137.00**	
Project:	**Field's North Trust Property**	**Elevation:**	**Date: 8/31/01**
Location:	**Signal Hill, California**	**Total Depth: 12.0'**	**Log by: RSR**
Drill Rig Type:	**24" Bucket Auger** — **Driving Weights: 0-25 feet: 2500 lbs.**	**Sheet: 1 of 1**	Start: / Stop:

Depth (Feet)	USCS Symbol	Material Description	Water	Samples: Blow Count	Samples: Ring	Samples: Bulk	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Other Lab Tests
	SM	**ARTIFICIAL FILL (Qaf):**							(MAX./
		Silty sand, fine grained, brown, dry to damp, medium dense,							E.I./SO_4/
		scattered gravel and debris.							& D.S.)
				6			118.4	9.4	(Consol.)
5									
		BEDROCK: LAKEWOOD FORMATION (Qlw):							
		Silty sandstone, fine grained, brown to red-brown, damp, moderately		5			108.9	7.6	
		hard, weathered, locally porous, scattered rootlets.							
				3			97.0	3.0	
10		@ 3.0 feet: Becoming harder and less porous.							
		@ 6.0 feet: Becoming locally more porous, increase in rootlets.							
		@ 8.0 feet: Becoming less weathered, less porous, decrease in rootlets.							
15									
		@ 9.0 feet: Sandstone with traces of silt, fine grained, tan to light gray, dry to damp, moderately hard, micaceous.							
20		@ 12.0 feet: Heavy caving.							
		Total Depth: 12.0 feet							
25		Caving @ 12.0 feet							
		No Ground Water							

BORING LOG NO. B-3

Client:	**Signal Hill Petroleum**	**Job No.: 1137.00**	
Project:	**Field's North Trust Property**	**Elevation:**	**Date: 8/31/01**
Location:	**Signal Hill, California**	**Total Depth: 10.0'**	**Log by: RSR**
Drill Rig Type:	**24" Bucket Auger** / **Driving Weights: 0-25 feet: 2500 lbs.**	**Sheet: 1 of 1**	Start: / Stop:

Depth (Feet)	USCS Symbol	Material Description	Water	Samples: Blow Count	Samples: Ring	Samples: Bulk	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Other Lab Tests
	SM	**ARTIFICIAL FILL (Qaf):**							
		Silty sand, fine grained, brown, dry to damp, medium dense,		2			115.2	7.3	
		scattered gravel and debris.							
5				2			103.3	23.8	
		BEDROCK: LAKEWOOD FORMATION (Qlw):							
		Silty sandstone, fine grained, brown to red-brown, damp, soft to		PUSH					
		moderately hard, weathered, locally porous.		for 6"/			93.7	2.8	
				1 for 6"					
10		@ 3.0 feet: Increase in moisture and silt content.							
				4			96.2	2.6	
		@ 4.0 feet: Clayey sandstone with some silt, fine grained, red-brown, damp to slightly moist, soft to moderately hard, weathered, mottled, locally porous.							
15									
		@ 6.0 feet: Sandstone with traces of silt, fine grained, tan to light gray, damp, soft to moderately hard, micaceous.							
		@ 10.0 feet: Some rust-brown oxidation staining present. Hole caving heavily.							
20									
		Total Depth: 10.0 feet							
		Caving @ 10.0 feet							
25		No Ground Water							

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
T-1	0-1.0	SM	ARTIFICIAL FILL (Qaf): Silty sand, fine grained, brown, dry to damp, medium dense to dense, gravelly road base in upper 2 inches.
	1.0-3.0		BEDROCK: LAKEWOOD FORMATION (Qlw): Silty sandstone, fine grained, brown to red-brown, damp, moderately hard, locally porous (porosity decreases with depth).
			Total Depth: 3.0 feet No Caving No Ground Water Backfill Not Compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
T-2	0-2.5	SM	ARTIFICIAL FILL (Qaf): Silty sand, fine grained, light tan to gray, dry to damp, loose to medium dense, rootlets in upper 6 inches, gravelly road base in upper 12 inches.
	2.5-3.5		BEDROCK: LAKEWOOD FORMATION (Qlw): Silty sandstone, fine grained, brown to red-brown, damp, moderately hard to hard, weathered, rootlets, pinhole pores.
			Total Depth: 3.5 feet No Caving No Ground Water Backfill Not Compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
T-3	0-1.5	SM	ARTIFICIAL FILL (Qaf): Silty sand, fine grained, light brown to tan, dry, medium dense to dense, rootlets, locally porous.
	1.5-4.0		BEDROCK: LAKEWOOD FORMAION (Qlw): Silty sandstone, fine grained, brown to red-brown, dry to damp, moderately hard, weathered, pinhole pores, scattered rootlets.
			@ 3.5: Becoming less weathered, damp to moist.
			Total Depth: 4.0 feet No Caving No Ground Water Backfill Not Compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
T-4	0-1.0	SM	ARTIFICIAL FILL (Qaf): Silty sand, fine grained with some gravel, brown, dry to damp, loose to medium dense, rootlets, pinhole pores, scattered debris.
	1.0-2.5		BEDROCK: LAKEWOOD FORMATION (Qlw): Silty sandstone, fine grained, brown to red-brown, damp, moderately hard, weathered, locally porous, scattered rootlets, some ped development.
			Total Depth: 2.5 feet No Caving No Ground Water Backfill Not Compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
T-5	0-1.0	SM	ARTIFICIAL FILL (Qaf): Silty sand, fine grained, light brown to tan, dry, medium dense to dense, rootlets, locally porous.
	1.0-2.0		BEDROCK: LAKEWOOD FORMATION (Qlw): Silty sandstone, fine grained, brown to red-brown, damp, moderately hard to hard, weathered, rootlets, pinhole pores.
			Total Depth: 2.0 feet No Caving No Ground Water Backfill Not Compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
T-6	0-1.0	SM	ARTIFICIAL FILL (Qaf): Silty sand, fine grained with some gravel, brown, dry to damp, loose to medium dense, rootlets, pinhole pores, scattered debris.
	1.0-2.5		BEDROCK: LAKEWOOD FORMATION (Qlw): Silty sandstone, fine grained, brown to red-brown, damp, moderately hard, weathered, locally porous, scattered rootlets.
			Total Depth: 2.5 feet No Caving No Ground Water Backfill Not Compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
T-7	0-3.0		BEDROCK: LAKEWOOD FORMATION (Qlw): Silty sandstone, brown to red-brown, damp, moderately hard, moderately to very weathered, locally very porous, some ped development, some roots. @ 3.0 feet: Less weathered and harder. Total Depth: 3.0 feet No Caving No Ground Water Backfill Not Compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
T-8	0-2.0	SM	ARTIFICIAL FILL (Qaf): Silty sand, fine grained, gray-brown, very moist to saturated, medium dense.
	2.0-4.0		BEDROCK: LAKEWOOD FORMATION (Qlw): Silty sandstone, fine grained, brown to red-brown, very moist, soft to moderately hard.
			@ 3.0 feet: Becoming damp and moderately hard.
			Note: Trench located approximately 7.0 feet below adjacent grade within excavated tank site.
			Total Depth: 4.0 feet No Caving No Ground Water Backfill Not Compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
T-9	0-3.0		BEDROCK: LAKEWOOD FORMATION (Qlw): Silty sandstone/Sandy siltstone, red-brown, damp to moist, soft to moderately hard, desiccated with clayey film and caliche. @ 2.0 feet: Becoming moderately hard. Note: Trench located 5.0 feet below adjacent grade within excavated tank site. Total Depth: 3.0 feet No Caving No Ground Water Backfill Not Compacted

DESCRIPTIVE LOG OF TRENCH EXCAVATION

Trench Number	Depth (Feet)	U.S.C.S. Symbol	Field Description
T-10	0-1.0	SM	ARTIFICIAL FILL (Qaf): Silty sand, fine grained, brown and dark gray-brown, very moist to saturated, loose, roots from nearby palm tree.
	1.0-4.0		BEDROCK: LAKEWOOD FORMATION (Qlw): Silty sandstone/Sandy siltstone, red-brown, damp to moist, soft to moderately hard, desiccated with clayey film and caliche. @ 2.0 feet: Becoming moderately hard. Note: Trench located approximately 2.0 feet below adjacent grade within excavated tank site. Total Depth: 4.0 feet No Caving No Ground Water Backfill Not Compacted

sudden increase in moisture content (hydro-consolidation potential). Results of this test are graphically presented on Plate B-1.

Direct Shear

The Coulomb shear strength parameters, angle of internal friction and cohesion, were determined for a sample remolded to 90 percent of maximum dry density. AMEC Earth & Environmental of Anaheim, California performed the test in general conformance with Test Method ASTM D 3080-80. 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-2.

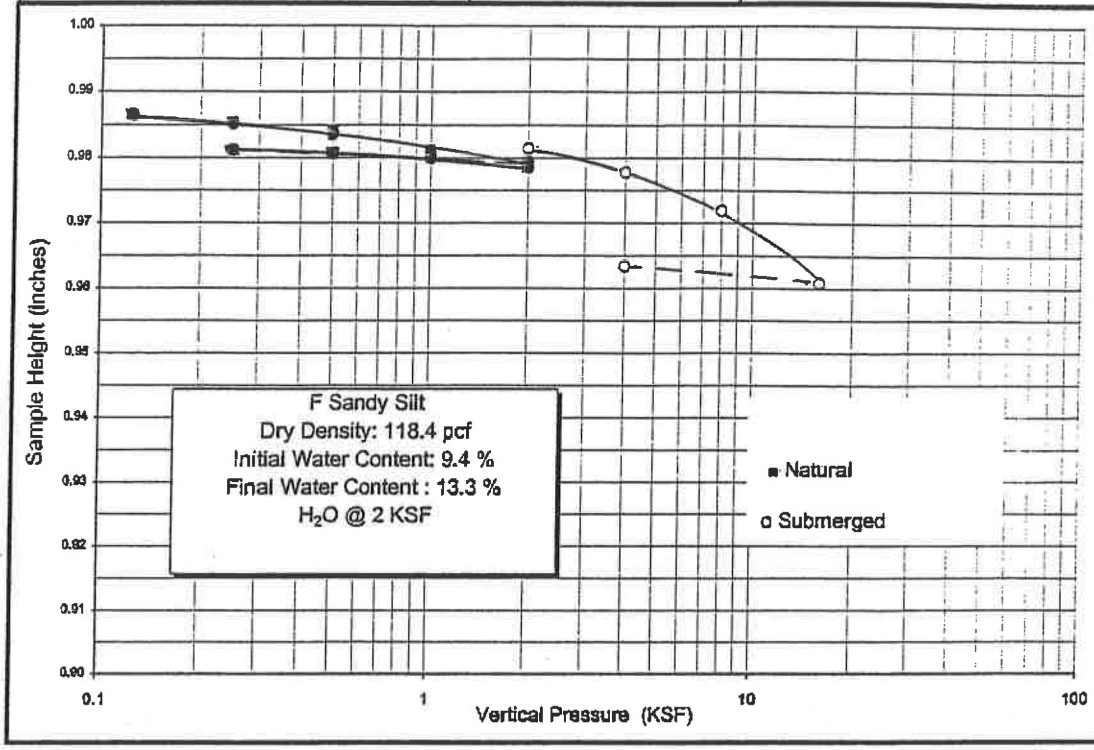
TABLE B-1
Summary of Laboratory Test Data

Number	Depth (feet)	Material Type	Test Results	
B-2	3.0-5.0	Silty Sandstone	Maximum Dry Density: Optimum Moisture Content: Expansion Index: Expansion Potential: Soluble Sulfate Content:	125.0 10.5 % 28 Low 0.0297

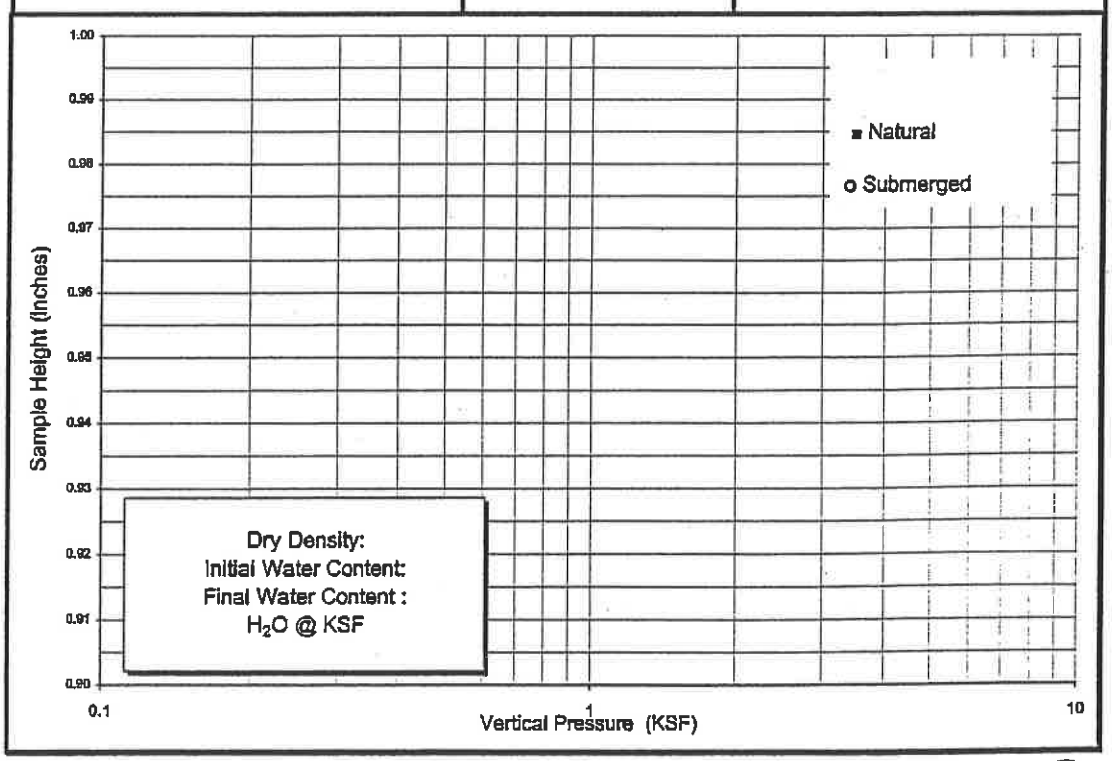
CONSOLIDATION TESTS

Job No. 0-212-1022_Albus Keefe # 1115.00

Boring / Sample No.	B2	Depth:	3'	Date	09-06-01



Boring / Sample No.	Depth:	Date



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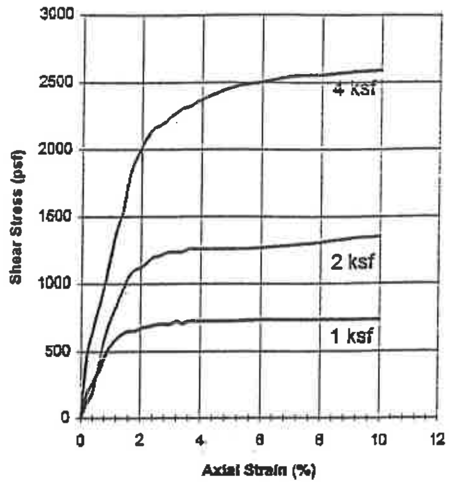
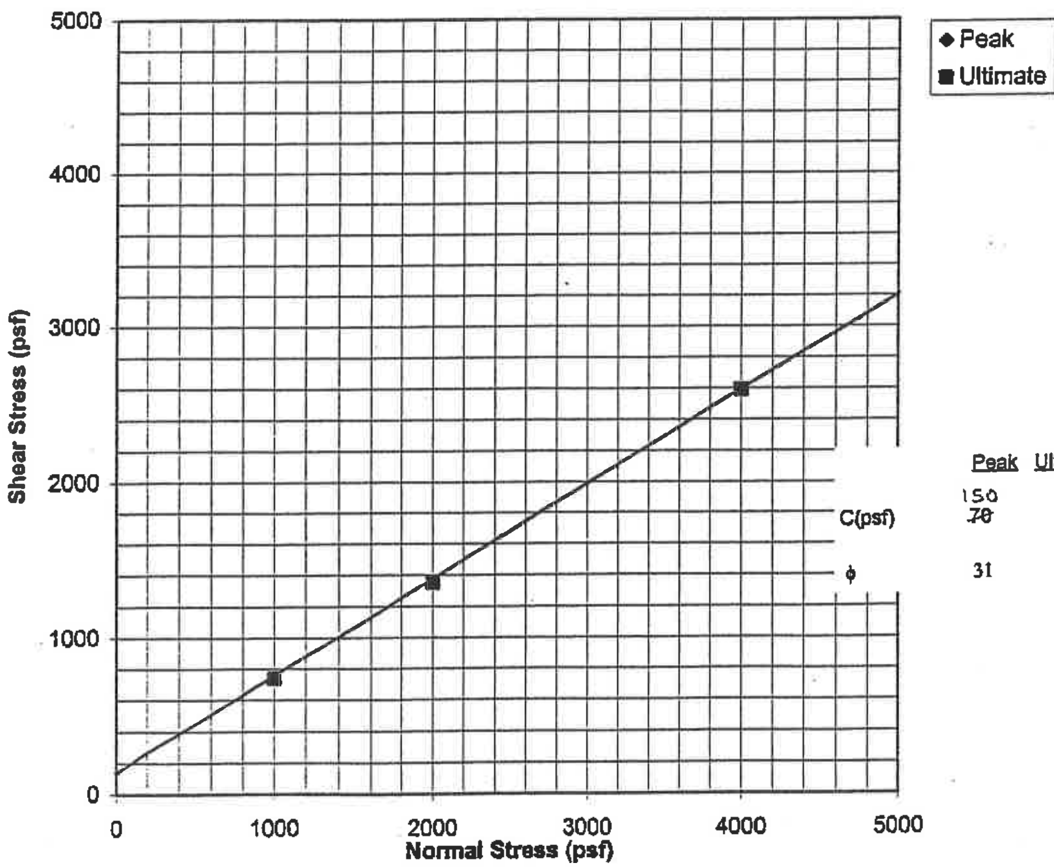
DIRECT SHEAR TEST

PROJECT:	Albus-Keefe #1137.00	Date:	09/07/2001
JOB NO.:	0-212-1022		
SAMPLE LOCATION:	B2 @ 3-5'		
SAMPLE TYPE:	Remolded –Saturated		
DESCRIPTION:	Silty F Sand		

Specimen No.	1	2	3
Normal Stress, psf	1000	2000	4000
Peak Stress, psf	740	1350	2590
Displacement, in.	0.100	0.249	0.227
Ultimate Stress, psf	740	1350	2590
Displacement, in.	0.250	0.250	0.250
Initial Dry Density, pcf	112.5	112.5	112.5
Initial Water Content, %	10.5	10.5	10.5
Strain Rate, in/min.	0.0083	0.0083	0.0083





	Peak	Ultimate
C(psf)	150 ~~70~~	150 ~~70~~
ϕ	31	31

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