March 4, 2021
J.N.: 2738.01

Mr. Steve Armanino
The Olson Company
3010 Old Ranch Parkway, Suite 100
Seal Beach, California 90740

Subject: **Geotechnical Grading Plan Review Report, Proposed Residential Development, 50 Tetley St, Hacienda, Heights, California.**

Dear Mr. Armanino,

Albus & Associates, Inc. is pleased to present to you our geotechnical grading plan review 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 & ASSOCIATES, INC.

Paul Hyun Jin Kim
Associate Engineer

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

The purpose of our geotechnical design report is to review the proposed site development shown on the referenced site improvement plan with respect to the geotechnical conditions in order to provide recommendations for site development. The scope of our work included:

- Review of published geologic reports, maps, historic photos and seismic data for the site and surrounding area
- Review of the referenced improvement plan
- Exploratory drilling and soil sampling
- Laboratory testing of selected soil samples
- Engineering and geologic analyses of data obtained from our review, subsurface exploration and laboratory testing
- Evaluation of site seismicity, liquefaction potential, and settlement potential
- Development of recommendations for site construction
- Preparation of this report

1.2 SITE LOCATION AND DESCRIPTION

The site is located at 50 Tetley St in the city of Hacienda Heights, California. The site is bordered by Tetley Street to the North and residential developments to the West, East and South. The location of the site and its relationship to the surrounding areas is shown on the Site Location Map, Figure 1.

The site is rectangular in shape and comprises approximately 2.1 acres of land. Current site developments include two 2-story buildings in the Northeast corner of the lot. The buildings appear to be used for worship and education. The northern portion of the site is covered in grass with light hardscape features and a children's playground area. The center of the site is covered in asphalt and used for parking. Light utilities are expected in this area in and around existing lighting features. The southern portion of the site is an undeveloped dirt lot with light hardscaped features and patio furniture. Relatively short masonry block walls approximately 1-3 feet in height separate the asphalt from the dirt sections. The site has a few moderate sized trees and shrubs, with a majority of the vegetation at the northern portion of the site. Two existing driveways run along the western and eastern portion of the site to gain access to the parking areas. Chain link fencing is located in the northern section around the children's play area. The site is relatively flat and drainage appears to flow north towards Tetley St.

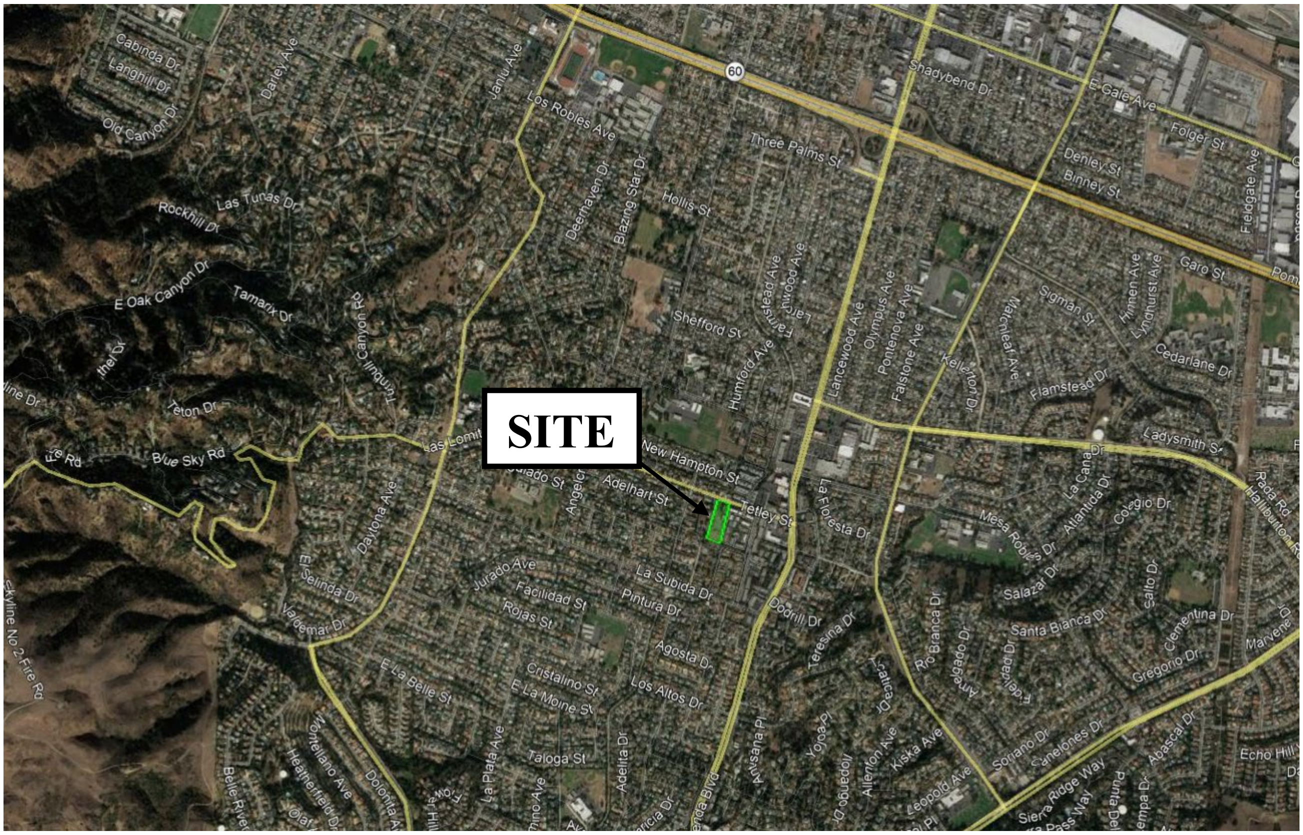

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FIGURE 1 - SITE LOCATION MAP
The Olson Company
Proposed Residential Development
50 Tetley St
Hacienda Heights, California

NOT TO SCALE

1.3 PROPOSED DEVELOPMENT

Upon review of the referenced plans, we understand the site will be developed for residential use consisting of 11 buildings with a total of 33 units of attached three-story townhomes. It is anticipated that all proposed structures will be constructed on grade (i.e. no subterranean elements). Associated interior driveways, perimeter/retaining walls, and underground utilities are also planned.

Based on the conceptual grading plans, we anticipate that minor cuts and fills of less than 3 feet of the site will be required to achieve future surface configuration. We expect the proposed residential dwellings will be wood-framed structures with concrete slabs on grade yielding relatively light foundation loads.

2.0 INVESTIGATION

2.1 RESEARCH

We have reviewed the referenced geologic publications, maps, and historical aerial photos of the vicinity. Data from these sources were utilized to the development of some of our findings and conclusions presented in this report.

As early as 1953, the site was used primarily for agricultural purposes and the vicinity of the site appears to as have been generally utilized for agricultural purposes. It appears that an old drainage ran along the southern portion of the east property line and then flows northeast away from the property into the adjoining property. By 1970, the property appears to have been developed with essentially the current configurations present today. The buildings preset today have also been constructed and the drainage is still preset. By 1976, the site appears relatively unchanged. By 1983, the drainage has disappeared and the residential property to the east has been constructed. Since 1983, the site appears relatively unchanged.

2.2 SUBSURFACE EXPLORATION

Subsurface exploration for this investigation was conducted at the site on July 19th, 2018, and consisted of drilling 4 exploratory borings. The borings were drilled to maximum depths of approximately 51.5 feet below the existing ground surface utilizing a track rig-mounted, hollow-stem-auger drill rig. Representatives of *Albus & Associates, Inc*. logged the exploratory excavations. Visual and tactile identifications were made of the materials encountered, and their descriptions are presented on the Exploration Logs in Appendix A. The approximate locations of the exploratory excavations completed by this firm are shown on the enclosed Geotechnical Map, Plate 1.

Bulk, relatively undisturbed and Standard Penetration Test (SPT) samples were obtained at selected depths within the exploratory boring for subsequent laboratory testing. Relatively undisturbed samples were obtained using a 3-inch O.D., 2.5-inch I.D., California split-spoon soil sampler lined with brass rings. SPT samples were obtained from the borings using a standard, unlined SPT soil sampler. During each sampling interval, the sampler was driven 18 inches with successive drops of a 140-pound automatic hammer falling 30 inches. The number of blows required to advance the sampler was recorded for each six inches of advancement. The total blow count for the lower 12 inches of advancement per soil sample is recorded on the exploration 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.

In addition, two percolation test borings, P-1 and P-2, were also excavated to an approximate depth of 10 and 15 feet in the vicinity of exploratory boring B-1 for subsequent percolation testing. The percolation test wells were later backfilled with auger cuttings upon completion of testing. Results of our percolation testing are discussed later in a separate report.

2.3 LABORATORY TESTING

Selected samples of representative earth materials from the borings were tested in our laboratory. Tests consisted of in-situ moisture and dry density, maximum dry density and optimum moisture content, expansion index, soluble sulfate content, consolidation/collapse potential, direct shear, grain size analysis, Atterberg limits. Descriptions of laboratory testing and a summary of the test results are presented in Appendix B and on the exploration log in Appendix A.

3.0 SUBSURFACE CONDITIONS

3.1 SOIL CONDITIONS

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

Review of the Diblee Map for the Whittier and La Habra Quadrangles shows that the site sits within the wash zone of the nearby Puente Hills. As such, the site has probably seen seasonal flooding and deposits from the nearby hills. The site is classified as Quaternary elevated alluvium (Qae), slightly elevated and locally dissected alluvial gravel and sand, on north side of Puente Hills.

Soils encountered at the site consisted of artificial fills and Qae deposits to the maximum depth explored of 51.5 feet. Artificial Fills were encountered in all of our exploratory borings to an approximate depth of 5 to 6 feet below ground surface. The fills encountered were typically silty clays, brown to grayish brown, moist and stiff to very stiff.

Qae deposits encountered in our exploratory borings were typically encountered at a depth of 6 feet to the maximum depth explored. Soils encountered between 6-20 feet were generally coarser grained materials consisting of silty sand with clay and gravelly sand with silt and clay, yellowish brown, damp to moist and medium dense to dense. Materials encountered below 20 feet were typically fine-grained materials consisting of Silty Clay, grayish brown to reddish brown, very moist and very stiff.

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

3.2 GROUNDWATER

A review of the CDMG Seismic Hazard Zone Report 09 La Habra indicates that historical high groundwater levels for the general site area is as shallow as 25 feet below the existing ground surface. Groundwater was encountered during this firm's subsurface exploration at approximately 34 feet.

3.3 FAULTING

Geologic literature and field exploration do not indicate the presence of active faulting within the site. The site does not lie within an "Earthquake Fault Zone" as defined by the State of California in the 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 Faults

Name	Distance (miles)	Slip Rate (mm/yr.)	Preferred Dip (degrees)	Slip Sense	Rupture Top (km)	Fault Length (km)
Elsinore;W+GI	1.82	n/a	81	strike slip	0	83
Elsinore;W	1.82	2.5	75	strike slip	0	46
Elsinore;W+GI+T+J +CM	1.82	n/a	84	strike slip	0	241
Elsinore;W+GI+T+J	1.82	n/a	84	strike slip	0	199
Elsinore;W+GI+T	1.82	n/a	84	strike slip	0	124
Puente Hills (Santa Fe Springs)	5.05	0.7	29	thrust	2.8	11
San Jose	6.17	0.5	74	strike slip	0	20
Puente Hills (Coyote Hills)	7.8	0.7	26	thrust	2.8	17

4.0 ANALYSES

4.1 SEISMICITY AND SEISMIC DESIGN PARAMETERS

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

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

However, "A ground motion hazard analysis is not required for structures where: Structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient Cs is determined by Eq. (12.8-2) for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for $T > T_L$." Assuming this exception is met for this project, a ground motion hazard analysis is not required and mapped seismic values can be used. Should this exception not be met, a ground motion hazard analysis is required to determine the Design response spectra for the proposed structures at this site. Both mapped and site specific seismic design parameters are provided in this report as presented in Section 6.2. Details of a ground motion hazard analysis are explained below.

According to Section 21.2.3 (Supplement 1), the site-specific Risk Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration at any period is the lesser of the probabilistic and the deterministic response accelerations, subject to the exception specified in the same section. The probabilistic response spectrum was developed using the computer program OpenSHA (Field et al., 2013), which implements Method 1 as described on Section 21.2.1.1. Fault Models 3.1 and 3.2 from the Third Uniform California Earthquake Rupture Forecast (UCERF3) were used as the earthquake rupture forecast models for the PSHA. In addition to known fault sources, background seismicity was also included in the PSHA. The ground motion Prediction Equations (GMPEs) selected for use in this analysis are those developed for the Pacific Earthquake Engineering Research Center (PEER) Next Generation Attenuation (NGA) West 2 project. Four GMPEs - Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), and Chiou and Youngs (2014) were used to perform the analysis.

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

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

Per Section 11.2 (definitions on Page 79 of ASCE7-16) for evaluation of liquefaction, lateral spreading, seismic settlements, and other soil-related issues, Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration PGA_M shall be used. The site-specific PGA_M is calculated per Section 21.5.3, as the lesser of the probabilistic PGA_M (Section 21.5.1) and deterministic PGA_M (Section 21.5.2), but no less than 80% site modified peak ground acceleration, PGA_M, obtained from OSHPD seismic hazard tool. From our analyses, we obtain a PGA_M of 0.862g.

4.2 STATIC SETTLEMENT

Analyses were performed to evaluate potential for static settlement. Our analyses were based on the results of consolidation tests performed on selected samples from our borings. The artificial fill is not considered suitable for support of engineered fill or foundations in its current condition. Results of our testing indicate the alluvial soils are prone to slight consolidation upon loading. If the existing 5 to 6 feet of earth materials are removed and recompacted, we estimate the total settlement will be less than 1 inch.

4.3 LIQUEFACTION

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

- A source of ground shaking, such as an earthquake, capable of generating soil mass distortions.
- A relatively loose silty and/or sandy soil.
- A relative shallow groundwater table (within approximately 50 feet below ground surface) or completely saturated soil conditions that will allow positive pore pressure generation.

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

A tabulated summary of our liquefaction analyses is provided on Table C-1 in Appendix C. Blow counts obtained from the larger California ring-lined sampler were corrected in the analyses (SCEC, 1999). Historic high groundwater was assumed at a depth of 25 feet below the existing ground surface based on the CDMG Seismic Hazard Report. Fine-grained soils that do not have a Plasticity Index (PI) less than 12 and field moisture contents greater than 85% of liquid limit (LL) or soils with corrected blow counts greater than 30 blows per foot were assumed to be not susceptible to liquefaction. Based on our analyses, the soils below the historic high groundwater levels are clayey in nature and exhibit a Plasticity Index (PI) greater than 12 and not considered susceptible to liquefaction per Youd, et al. (2001).

Seismic-induced settlement can also occur both above the groundwater table during a strong seismic event. We have estimated the dry soil settlement using the Tokumatsu and Seed (1987) Method. The analysis indicates a total dry seismic settlement 1.2 inches. However, Martin and Lew (1999) recommend that the dry seismic settlement estimate be multiplied by two to account for multi-directional shaking. Therefore, the total estimated dry seismic settlement is 2.4 inches.

5.0 CONCLUSIONS

5.1 FEASIBILITY OF PROPOSED DEVELOPMENT

From a geotechnical point of view, the proposed site development is considered feasible provided the recommendations presented in this report are incorporated into the design and construction of the project. Furthermore, it is also our opinion that the proposed development will not adversely impact the stability of adjoining properties if grading and construction is performed in accordance with the recommendations presented in this report. Key issues that could have significant impacts on the geotechnical aspects of the proposed site development are discussed in the following sections of this report.

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

5.2 GEOLOGIC HAZARDS

5.2.1 Ground Rupture

No known active faults are known to project through the site nor does the site lie within the boundaries of an "Earthquake Fault Zone" as defined by the State of California in the Alquist-Priolo Earthquake Fault Zoning Act. The closest known active fault is the Elsinore (Whittier) fault located about 1.82 miles from the site. Therefore, the potential for ground rupture due to an earthquake beneath the site is considered low.

5.2.2 Ground Shaking

The site is situated in a seismically active area that has historically been affected by generally moderate to occasionally high levels of ground motion. The site lies in relative close proximity to several 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. Potential ground accelerations have been estimated for the site and are presented in Section 4.1 of this report. Design and construction in accordance with the current California Building Code (C.B.C.) requirements is anticipated to adequately address 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. Geologic hazards associated with landsliding are not anticipated at the sites.

5.2.4 Liquefaction

Based on our analyses, the liquefaction potential of the underlying soils is considered to be very low. This is due to the presence of clayey cohesive soils.

However, the analyses does indicate a total dry sand settlement of 2.4 inches. Seismic-induced differential settlement is not expected to exceed one half the total settlement according to Martin and Lew (1999). The differential settlement can be less than one half the total settlement at sites with relatively uniform soil conditions and deep sediments. We estimate that differential settlement of the proposed structure will not exceed 1.2 inches during the design event.

While this magnitude is relatively high, past performance of one- to three-story wood-frame structures has shown that such movement may cause significant cosmetic damage but does not typically result in compromising the overall structural integrity of the building. Based on the State of California Special Publication 117A, hazards from liquefaction should be mitigated to the extent required to reduce seismic risk to "acceptable levels". The acceptable level of risk means, "that level that provides reasonable protection of the public safety" [California Code of Regulations Title 14, Section 3721 (a)]. Therefore, no special ground improvement measures are anticipated to mitigate adverse effects from seismic settlement caused by liquefaction.

If strong ground shaking occurs, seismic-induced settlement in excess of 1 inch could occur. This condition could cause a loss of bearing support for foundations leading to significant tilting or collapse of buildings not properly designed for this condition. This condition can be mitigated provided the buildings are no more than three stories in height and supported by well-reinforced foundations such as post-tensioned mats.

5.3 STATIC SETTLEMENT

The upper 5 to 6 feet of site soils (the existing artificial fill) are considered unsuitable for support of proposed site development in the current conditions. Provided the upper 5 to 6 feet of existing soils are removed and recompacted, total and differential static settlement can likely be limited to a maximum of 1 inch and ½-inch over 30 feet, respectively. These estimated magnitudes of static settlements are considered within tolerable limits for the proposed residential structures.

5.4 EXCAVATION AND MATERIAL CHARACTERISTICS

In general, the existing near-surface soils are considered unsuitable in their existing condition to support proposed structural fills and site development. This condition can be mitigated by removal and recompaction of unsuitable soils. The anticipated depth of removal to mitigate structural load-induced settlement below the proposed residential buildings, retaining walls, and pavement is on the order of 5 to 6 feet below existing ground surface.

Temporary construction slopes and trench excavations can likely be cut vertically up to a height of 5 feet within the onsite materials provided that no surcharging of the excavations is present. Temporary excavations greater than 5 feet in height will likely require side laybacks to 1:1 (H:V) or flatter to mitigate the potential for sloughing.

An existing sewer line is present approximately 3 feet from the west property line and approximately 6 feet below existing grades. If the existing sewer line is to be removed, removals along the property line will likely require slot cutting techniques. If the sewer line is to be abandoned in place, a slurry cap will need to be provided at the property line.

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.

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

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

Subsurface soils are anticipated to be relatively easy to excavate with conventional heavy earthmoving equipment. Removal and recompaction of the site materials will result in some moderate shrinkage and subsidence. Design of site grading will require consideration of this loss when evaluating earthwork balance issues.

The existing near surface soils are typically above and below optimum moisture content. The dry soils are anticipated to require water to achieve proper compaction while the wet soils will require some drying. Due to the cohesive nature of the soils, disking may be required.

5.5 SHRINKAGE AND SUBSIDENCE

Volumetric changes in earth quantities will occur when excavated onsite soil materials are replaced as properly compacted fill. We estimate the existing upper 6 feet of earth materials will shrink up to approximately 5 to 10 percent. Subsidence of removal bottoms is estimated to be on the order of 0.1 feet. The estimates of shrinkage and bulkage 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 swelling and bulkage that occurs during the grading process.

5.6 SOIL EXPANSION

Based on our laboratory test results and the USCS visual manual classification, the near-surface soils within the site are generally anticipated to possess a **Medium to High** 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 earthwork and grading should be performed in accordance with applicable requirements of Cal/OSHA, applicable specifications of the Grading Codes of the City of Hacienda Heights, 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 utility lines, septic tanks, seepage pits, etc. are also anticipated at the site. If encountered during site development, these improvements should also be completely removed from the site and seepage pits should be properly abandoned in accordance with the requirements established by the governing agencies as well as recommendations made in the field by the project geotechnical consultant.

In general, seepage pits that are open should be cleared of any fluids and then filled with 2-sack cement slurry up to within 5 feet of proposed grades. Any brick lining that remains in the upper 5 feet should be removed and the remainder of the pit filled with engineered fill in accordance with Section 6.1.7. Seepage pits that are presently backfilled with soil should be removed to a depth of 10 feet below pad grade and be capped with 2-sack cement slurry. The slurry cap should be at least 5 feet thick and should extend at least 12 inches outside the perimeter of the seepage pit. The remaining 5 feet should be filled with engineered fill in accordance with Section 6.1.7.

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.

Asphaltic concrete debris generated by site demolition can be reduced to no more than 4 inches in maximum dimension and uniformly incorporated with fill soils during earthwork operations.

6.1.4 Ground Preparation

To provide a uniform bearing material, the upper 5 to 6 feet of the existing earth materials should be removed and replaced as engineered compacted fills. These removals will be required in proposed building pads, retaining walls, and any other “structural” areas, and replaced as engineered compacted fill. Due to the undocumented nature of the artificial fills, areas of hardscape and pavement will also require removals to a depth of 5 to 6 feet below the existing ground surface. The actual depth of removal should be determined by the geotechnical consultant during grading.

In addition to general removal of unsuitable soils, the existing soils should be over-excavated to a minimum depth of 2 foot below the bottom of footings for residential structures supported by conventional spread footings. Existing soils within driveways, parking areas and retaining walls less than 3 feet, should be removed to at least 12 inches below the proposed pavement subgrade and replaced with engineered compacted fill.

Removals should extend laterally beyond the limits of the proposed buildings and retaining walls over 3 feet in height a distance equal to the depth of removal (i.e. 1:1 projection) but not less than 5 feet. Existing soils below proposed retaining walls less than 3 feet in height, screen walls, hardscapes and roadways ways should be removed laterally to at least the edge of the structure or pavement. Where removals are limited by existing structures, protected trees or property lines, special considerations may be required in the construction of affected improvements. Under such conditions, specific recommendations should be provided by this firm.

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

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

6.1.5 Scarification

Following removals, the exposed grade should first be scarified to a depth of 6 inches; moisture conditioned to at least 120 percent of the optimum moisture content, and then compacted to at least 90 percent of the laboratory determined maximum dry density.

6.1.6 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, the vertical cut may be decreased to as much as zero (0) and lay backs will likely be flatter to a gradient of 2:1 (H:V).

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.

6.1.7 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.

6.1.8 Import Materials

If import materials are required to achieve the proposed finish grades, the import soils should have an Expansion Index (EI) less than 75 (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

6.2.1 Mapped Seismic Design Parameters

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

TABLE 6.1
2019 CBC Mapped Seismic Design Parameters

Parameter	Value
Site Class	D
Mapped MCE_R Spectral Response Acceleration, short periods, S_S	1.851
Mapped MCE_R Spectral Response Acceleration, at 1-sec. period, S_1	0.655
Site Coefficient, F_a	1
Site Coefficient, F_v	1.7*
Adjusted MCE_R Spectral Response Acceleration, short periods, S_{MS}	1.851
Adjusted MCE_R Spectral Response Acceleration, at 1-sec. period, S_{M1}	1.114
Design Spectral Response Acceleration, short periods, S_{DS}	1.234
Design Spectral Response Acceleration, at 1-sec. period, S_{D1}	0.742
Long-Period Transition Period, T_L (sec.)	8
Seismic Design Category for Risk Categories I-IV	II

MCE_R = Risk-Targeted Maximum Considered Earthquake

*According to Section 11.4.8 in ASCE 7-16, "a ground motion hazard analysis shall be performed in accordance with Section 21.2 for the following structures on Site Class D and E sites with S_1 greater than or equal to 0.2." However, "A ground motion hazard analysis is not required for structures where: Structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient C_s is determined by Eq. (12.8-2) for values of $T \leq 1.5T_s$ and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for $T_L \geq T > 1.5T_s$ or Eq. (12.8-4) for

$T > T_L$." The F_v value of 1.7 above from Table 11.4-2 assumes that this exception is met and that a ground motion hazard analysis is not required. Should this exception not be met, the site-specific seismic design parameters provided in the next section should be used.

6.2.2 Site-Specific Seismic Design Parameters

In addition to the Code Spectra parameters presented in Table 6.1, we have performed a site-specific ground motion hazard analysis in accordance with Chapter 21 of ASCE 7-16 to obtain site-specific seismic design acceleration parameters, the risk-targeted maximum considered earthquake response spectrum, and the design earthquake response spectrum. The site-specific seismic design parameters are presented below.

TABLE 6.2
2019 CBC Site Specific Seismic Design Parameters

Parameter	Value
Site Class	D
Site Coefficient, F_a	1.0
Site Coefficient, F_v	2.5
Adjusted MCE Spectral Response Acceleration, short periods, S_{MS}	2.008
Adjusted MCE Spectral Response Acceleration, at 1-sec. period, S_{M1}	1.310
Design Spectral Response Acceleration, short periods, S_{DS}	1.339
Design Spectral Response Acceleration, at 1-sec. period, S_{D1}	0.873

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 **Medium to High** 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

Under normal static conditions, the foundation system should be designed to tolerate a total settlement of 1 inches and a differential settlement of ½-inch over 30 feet. The foundations should also be designed for total and differential seismic settlement of 2.4 inches and 1.2 inches over 30 feet, respectively. The PTI design parameters presented below incorporate the estimated seismic settlements.

6.3.4 Allowable Bearing Value

Provided site grading is performed as recommended herein, a bearing value of 2,500 pounds per square foot (psf) may be used for continuous beams or isolated pad 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 pad footings is based on a minimum width of 24 inches and founded a minimum of 12 inches. The above value may be increased by 200 psf and 500 psf for each additional foot in width and depth, respectively, up to a maximum value of 3,500 psf. Recommended allowable bearing values include both dead and live loads and may be increased by one-third for wind and seismic forces.

6.3.5 Lateral Resistance

Provided site grading is performed in accordance with the recommendations provided by the project geotechnical consultant, a passive earth pressure of 240 pounds per square foot per foot of depth up to a maximum value of 1,200 pounds per square foot may be used to determine lateral bearing for beams. This value may be increased by one-third when designing for wind and seismic forces. A coefficient of friction of 0.41 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 passive resistance values should be decreased by 50% such as property line walls.

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 Post-Tensioned Slab/Mat on Grade

Due to expansion potential, the proposed structures may be supported by a post-tension slab. Perimeter edge beams for the post-tensioned slabs should have a minimum effective width of 12 inches and be founded at a minimum depth of 18 inches below the lowest adjacent final ground surface. Interior beams may be founded at a minimum depth of 12 inches below the tops of the finish floor slabs. Where a post-tensioned mat is utilized, the exterior edge of the mat should be embedded at least 8 inches below the lowest adjacent grade. The thickness of the floor slab/mat should be determined by the project structural engineer; however, we recommend a minimum slab thickness of 5.0 inches.

Design of the mat may be based on a modulus of subgrade reaction (Kv1) of 57 pounds per cubic inch (pci). The modulus is based on an effective loading area of 1 foot by 1 foot. The modulus may be adjusted for other effective loading areas using the equation provided below.

$$k_b(pci) = 57\left\{\frac{b+1}{2b}\right\}^2$$

where “b” is the effective width of loading (minimum dimension) in feet.

Concrete floor slabs in areas to receive carpet, tile, or other moisture sensitive coverings should be underlain with a minimum of 10-mil moisture vapor retarder conforming to ASTM E 1745, Class A. The membrane should be properly lapped, sealed, and underlain within a layer of sand at least 2 inches thick. One inch of sand may be placed over the membrane to aid in the curing of the concrete. The sand should have a SE no less than 30. This vapor retarder system is anticipated to be suitable for most flooring finishes that can accommodate some vapor emissions. However, this system may emit more than 4 pounds of water per 1000 sq. ft. and therefore, may not be suitable for all flooring finishes. Additional steps should be taken if such vapor emission levels are too high for anticipated flooring finishes.

Prior to placing concrete, subgrade soils below slab-on-grade/mat areas should be thoroughly moistened to provide moisture contents at least 120 percent of the optimum moisture content to a depth of 12 inches.

Based on the guidelines provided in the "Design of Post-Tensioned Slabs-on-Ground" 3rd Edition by Post-Tensioning Institute, the em and ym values are summarized below:

TABLE 6.3
PTI Design Parameters

Parameter	Value
Edge Lift Moisture Variation Distance, e_m	3.9 feet
Edge Lift, y_m	2.128 inches
Center Lift Moisture Variation Distance, e_m	7.0 feet
Center Lift, y_m	1.492 inches

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 passive resistance values should be reduced by 50% for walls along property lines.

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.

6.4.3 Earth Pressures

Static and seismic earth pressures for level and 2:1 (H:V) backfill conditions are provided in Table 6.3. Seismic earth pressures provided herein are based on the method provided by Seed & Whitman (1970) using a peak ground acceleration (PGA) of 0.475 g for 10% probability of exceedance in 50 years. As indicated in Section 1803.5.12 of the 2019 CBC, retaining walls supporting 6 feet of backfill or less are not required to be designed for seismic earth pressures. The values provided in the following table do not consider hydrostatic pressure. Retaining walls should also be designed to support adjacent surcharge loads imposed by other nearby footings or traffic loads in addition to the earth pressure.

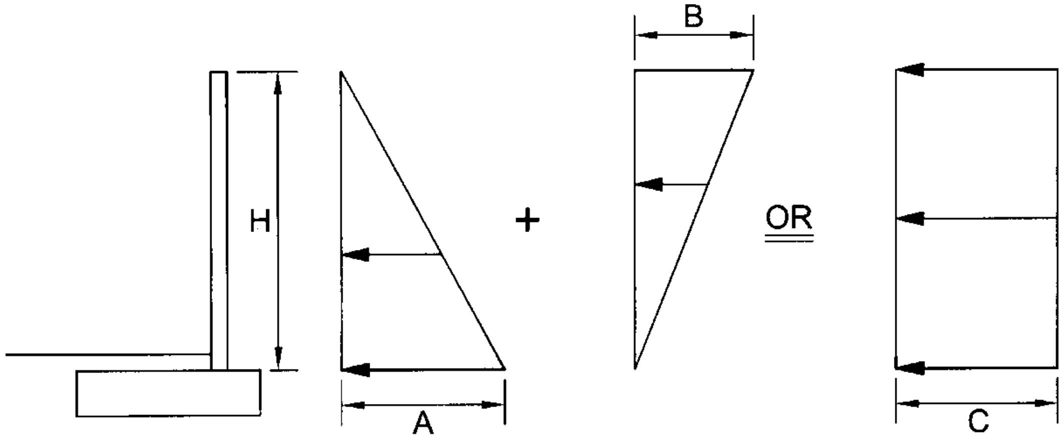
6.4.4 Drainage and Moisture-Proofing

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

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

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

TABLE 6.4
SEISMIC EARTH PRESSURES
Pressure Diagram



Static Component | Seismic Component | Total Force

Earth Pressure Values
Walls Up to 10 Feet in Height

Value	Backfill Condition	
	Level	2H:1V Slope
A	45H	65H
B	12H	12H
C	28.5H	38.5H

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

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

6.4.6 Wall Jointing

All free-standing, exterior site walls should be provided with cold joints through the masonry block section at horizontal spacing generally not exceeding 15 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.

6.4.7 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.

6.4.8 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 4.5 inches thick and provided with saw cuts or expansion joints at spacing no greater than 6 feet in each direction. Flatwork more than 6 feet in width across the minimum dimension should be reinforced with 4" by 4", W2.9 by W2.9 welded wire mesh or No 3 bars spaced 12 inches center to center in both directions. Cold joints should be keyed or provided with dowels spaced 24 inches on center. Flatwork that meets the structure at points of entry should be doweled into the footing or grade beam of the structure. Consideration should also be given to doweling flatwork into curbs where they meet. Special jointing detail should be provided in areas of block-outs, notches, or other irregularities to avoid cracking at points of high stress. Subgrade soils below flatwork should be thoroughly moistened to a moisture content of at least 120 percent of optimum to a depth of 12 inches. Moistening should be accomplished by lightly spraying the area over a period of a few days just prior to pouring concrete.

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.

6.6 CONCRETE MIX DESIGN AND CORROSION

Laboratory testing of existing near-surface soils for soluble sulfate content indicates soluble sulfate concentration less than 0.10%. We recommend following the procedures provided in ACI 318, Section 4.3, Table 4.3.1 for **negligible** sulfate exposure. Upon completion of rough grading, an evaluation of as-graded conditions and further laboratory testing should be completed for the site to confirm or modify the recommendations provided in this section.

6.7 POST GRADING CONSIDERATIONS

6.7.1 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 1804.4 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.

6.7.2 Utility Trenches

Trench excavations should be constructed in accordance with the recommendations contained in Section 6.1.6 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.

6.8 PRELIMINARY PAVEMENT DESIGN

6.8.1 Preliminary Structural Sections

Based on the soil conditions present at the site and estimated traffic indices, preliminary pavement sections are provided in Table 6.4 below. A preliminary "R-value" of 5 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 6.5
PRELIMINARY PAVEMENT STRUCTURAL SECTIONS
FOR RESIDENTIAL DEVELOPMENT

Location	Traffic Index	AC (inches)	Paver Thickness (mm)	Portland Cement Concrete (inches)	AB (inches)
Main Street	5.0	3.0 4.0	--	--	11.0 8.0
		--	80	--	12.0
		--	--	7.5	--
Parking Stalls	--	3.0	--	---	6.0

6.8.2 Subgrade Preparation

Prior to placement of pavement elements, subgrade soils should be moisture-conditioned to at least 120 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.

6.8.3 Aggregate Base

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

6.8.4 Asphaltic Concrete

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 6.5 within 2 feet of cold joints.

6.8.5 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 6.6.

TABLE 6.6
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 2,500 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. However, cold joints should be provided with dowels or keyways are recommended by PCA.

6.8.6 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.

6.9 PLAN REVIEW AND CONSTRUCTION SERVICES

We recommend *Albus & 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 summarizes several geotechnical topics that should be beneficial for project planning and budgetary evaluations. The information presented herein is intended only for a preliminary feasibility evaluation and is not intended to satisfy the requirements of a site specific and detailed geotechnical investigation required for further planning and permitting.

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 **The Olson Company** to assist the project consultants in determining the feasibility 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,

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Plans

Advanced Civil Group, Inc., Vesting Tentative Trace No. 82498 For Condominium Purposes, Tetley Avenue, Hacienda Heights, CA, Sheet 1, Dated 2/17/2021

Aerial Photographs

Photo Source	Date Flown	Flight No.	Photo No.
Continental Aerial Photo, Inc.	23-Feb-99	C133-30	21, 22
Continental Aerial Photo, Inc.	16-Oct-97	C119-30	185
Continental Aerial Photo, Inc.	11-Jul-95	C114-30	57, 58
Continental Aerial Photo, Inc.	19-May-93	C92-18	121
Continental Aerial Photo, Inc.	23-Jan-92	C85-1	11
Continental Aerial Photo, Inc.	12-Jun-90	C83-12	39, 40
Continental Aerial Photo, Inc.	07-Jul-88	19148	
Continental Aerial Photo, Inc.	17-May-83	218-4	5
Continental Aerial Photo, Inc.	28-Dec-76	181-4	7
Continental Aerial Photo, Inc.	30-Jan-70	60-4	108, 109
Continental Aerial Photo, Inc.	30-Jan-70	60-3	80
Continental Aerial Photo, Inc.	30-May-53	AXK-6K	33

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0 20 40 80

APPROX. SCALE : 1" = 40'

EXPLANATION

(Locations Approximate)

- Exploratory Boring
- Percolation Test Boring
- Limit of Removal
- Anticipated Depth of Removal Below Existing Grades (feet)

ALBUS & ASSOCIATES

GEOTECHNICAL MAP

Job No.: 2738.01	Date: 03/04/21	Plate: 1

APPENDIX A

EXPLORATION LOGS

Field Identification Sheet

- Light gray
- Gray
- Dark gray
- Gray Brown
- Light brown
- Brown
- Dark Brown
- Olive brown
- Olive
- Yellow
- Yellowish brown
- Yellowish red
- Red
- Reddish Brown
- Tan

Description Order:
Description, Color, Moisture, Density, Grain Size, Additional Description

Description	%	Example
	0-5	Sand
trace	5-15	Sand trace Silt
with	15-30	Sand with Silt
	30+	Silty Sand

More Examples
Sand with Silt trace Clay
Sand trace Silt and Clay
Sand with Silt and Clay
Gravelly Sand with Silt trace Clay
Silty Clay with Sand trace Gravel

Moisture

Dry	absence of water
Damp	below optimum
Moist	near optimum
Very Moist	above optimum
Wet	free water visible

Density (Navfac)

Coarse grained soils	SPT	CA
Very Loose	0-3	0-5
Loose	3-8	5-13
Medium Dense	8-14	13-22
Dense	14-25	22-40
Very Dense	25>	40>

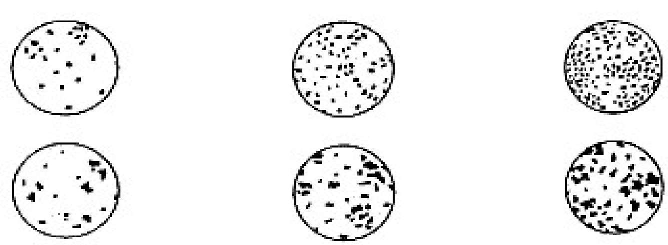
Fine grained soils		
Very Soft	2<	0-3
Soft	2-4	3-6
Medium Stiff	4-8	6-13
Stiff	8-15	13-24
Very Stiff	15-30	24-48
Hard	30>	48>

Grain Size

Description		Sieve Size	Approx. Size
Boulders		>12"	Larger than basketball
Cobbles		3-12"	Fist to basketball
Gravel	coarse	3/4-3"	Thumb to Fist
	fine	#4-3/4"	Pea to Thumb
Sand	coarse	#10-4	Rock Salt to Pea
	medium	#40-10	Sugar to Rock Salt
	fine	#200-40	Flour to Sugar
Fines		Pass #200	Smaller than Flour

Additional Description (ie. roots, pinhole pores, debris, etc.)

Trace 5% Moderate 15% Abundant 30%



EXPLORATION LOG

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

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

EXPLORATION LOG

Project: Hacienda Heights (Tetley & Hacienda)	Location: B-1
Address: 50 Tetley St, Hacienda Heights, CA	Elevation: 436.9
Job Number: 2738.00 / Client: The Olson Company	Date: 7/19/2018
Drill Method: Hollow-Stem Auger / Driving Weight: 140 lbs / 30 in	Logged By: ddalbus

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		ARTIFICIAL FILL (Af) Clay (CL): Brown, moist, very stiff, trace roots and pinhole pores, with silt							Max EI SO4 DS ATT
				28			11.3	95.9	
				29			8.1	99.2	
5		**ELEVATED ALLUVIUM (Qae)** Silty Sand (SM): Yellowish brown, damp, dense, fine grained sand, trace pinhole pores, with clay							
		@ 6 ft, medium dense, trace root and pinhole pores, caliche stringers		19			9.4	105.5	Consol
		Sandy Silt (ML): Yellowish brown, damp, stiff, fine grained sand, trace pinhole pores, with clay							
10				16			14.6	103.1	Consol SA Hydro
		Silty Sand (SM): Yellowish brown, medium dense							
15				9					SA Hydro

Albus & Associates, Inc.

EXPLORATION LOG

Project: Hacienda Heights (Tetley & Hacienda)		Location: B-1
Address: 50 Tetley St, Hacienda Heights, CA		Elevation: 436.9
Job Number: 2738.00	Client: The Olson Company	Date: 7/19/2018
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: ddalbus

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		@ 20 ft, trace gravel		14					
		<u>Sand with Gravel (SP):</u> Yellowish brown, damp, very dense, fine to coarse grained sand, trace silt and clay							
25		@ 23 ft, gravel zone		40					
		<u>Clay with Sand (CL):</u> Grayish brown to reddish brown, moist, hard, with silt							
		<u>Clay (CL):</u> Grayish brown to reddish brown, very moist, very stiff, with silt							
30				23					
		@ 31.5 ft, wet at sampler tip							
35				21					

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Plate A-3

EXPLORATION LOG

Project: Hacienda Heights (Tetley & Hacienda)			Location: B-1
Address: 50 Tetley St, Hacienda Heights, CA			Elevation: 436.9
Job Number: 2738.00	Client: The Olson Company		Date: 7/19/2018
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in		Logged By: ddalbus

Depth (feet)	Lith-ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
				24					
45				18					
50				26					

Total depth 51.5 feet
Groundwater at 33.8 feet

Percolation Well (2 Wells 10' offset):
10' of solid 3" pipe
5' of perf 3" pipe with filter sock

5' of solid 3" pipe
5' of perf 3" pipe with filter sock

perforated zones covered with 3/4" gravel

EXPLORATION LOG

Project: Hacienda Heights (Tetley & Hacienda)		Location: B-2
Address: 50 Tetley St, Hacienda Heights, CA		Elevation: 440.4
Job Number: 2738.00	Client: The Olson Company	Date: 7/19/2018
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: ddalbus

Depth (feet)	Lithology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
0		Asphalt (AC): 3 inches							
		Crushed Aggregate Base (CAB): 3 inches							
		ARTIFICIAL FILL (Af) Clay (CL): Brown to grayish brown, moist, very stiff, with silt		30	■		18.8	106	
				46	■		17.5	108.4	
5		**ELEVATED ALLUVIUM (Qae)** Silty Sand (SM): Yellowish brown, moist, loose, fine grained sand, trace pinhole pores, with clay		10	■		16.8	102.1	Consol
		Sand with Silt and Gravel (SP-SM): Light reddish brown, moist, medium dense, fine to coarse grained sand							
10				21	■		3.5		
		Silty Sand (SM): Grayish brown, moist, very dense, fine grained sand, with clay							
15				39	⧗				
		Sand with Silt and Gravel (SP-SM): Light reddish brown, damp, very dense, fine to coarse grained sand, trace clay							
		Total depth 16.5 feet No groundwater Boring backfilled with soil cuttings and capped with asphalt							

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Plate A-5

EXPLORATION LOG

Project: Hacienda Heights (Tetley & Hacienda)		Location: B-3
Address: 50 Tetley St, Hacienda Heights, CA		Elevation: 443.2
Job Number: 2738.00	Client: The Olson Company	Date: 7/19/2018
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: ddalbus

Depth (feet)	Lith- ology	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
		Asphalt (AC): 2 inches							
		Crushed Aggregate Base (CAB): 5.5 inches							
		ARTIFICIAL FILL (Af) Clay (CL): Grayish brown, moist, stiff, trace pinhole pores, with silt		23			20.3	102.4	
		@ 4 ft, hard, no pinhole pores		50			17.2	109	
5									
		ELEVATED ALLUVIUM (Qae) Silty Sand (SM): Yellowish brown, moist, dense, fine grained sand, trace root hairs and pinhole pores, with clay		28			17.5	102.4	Consol
10				38			2.2		
		Sand with Silt and Gravel (SP-SM): Light reddish brown, damp, dense, fine to coarse grained sand, trace clay							
		Sandy Silt / Silty Sand (ML/SM): Grayish brown, moist, stiff / medium dense, fine grained sand							
15				11					
		Total depth 16.5 feet No groundwater Boring backfilled with soil cuttings and capped with asphalt							

EXPLORATION LOG

Project: Hacienda Heights (Tetley & Hacienda)		Location: B-4
Address: 50 Tetley St, Hacienda Heights, CA		Elevation: 445.8
Job Number: 2738.00	Client: The Olson Company	Date: 7/19/2018
Drill Method: Hollow-Stem Auger	Driving Weight: 140 lbs / 30 in	Logged By: ddalbus

Depth (feet)	Material Description	Water	Samples: Blows Per Foot	Samples: Core	Samples: Bulk	Laboratory Tests: Moisture Content (%)	Laboratory Tests: Dry Density (pcf)	Laboratory Tests: Other Lab Tests
	ARTIFICIAL FILL (Af) Clay (CL): Grayish brown, damp to moist, very stiff, trace pinhole pores and root hairs, with silt							
			24			10.3	100.6	
5			27			9.3	97.2	
	ELEVATED ALLUVIUM (Qae) Silty Sand (SM): Yellowish brown, damp, dense, fine grained sand, with clay		24			8.6	98.3	
	@ 7 ft, Yellowish brown to reddish brown, trace pinhole pores							
10	Sandy Clay / Silty Clay (CL): Yellowish brown to reddish brown, moist, hard, fine grained sand		52			16.3	111.3	
15			31					
	Total depth 16.5 feet No groundwater Boring backfilled with soil cuttings							

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Plate A-7

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 D 2487). The samples were re-examined in the laboratory and classifications reviewed and then revised where appropriate. The assigned group symbols are presented on the Exploration Logs provided in Appendix A.

In-Situ Moisture Content and Dry Density

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

Laboratory Maximum Dry Density

Maximum dry density and optimum moisture content of onsite soils were determined for selected samples in general accordance with Method A of ASTM D 1557. Pertinent test values are given on Table B-1.

Expansion Potential

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

Soluble Sulfate Content

Chemical analysis was performed on selected samples to determine soluble sulfate content. The tests were performed in accordance with California Test Method No. 417. The test results are included on Table B-1.

Atterberg Limits

Atterberg Limits (Liquid Limit, Plastic Limit, and Plasticity Index) were performed in accordance with Test Method ASTM D4318. Pertinent test values are presented within Table B-1.

Particle-Size Analyses

Particle-size analyses were performed on selected samples in accordance with ASTM D 422. The results are presented graphically on the attached Plates B-1 and B-2.

Hydrometer

Hydrometer analyses were performed on representative samples of site materials in accordance with ASTM D 7928. The results are presented graphically on the attached Plates B-1 and B-2.

Consolidation

Consolidation tests were performed for selected soil samples in general conformance with ASTM D 2435. Axial loads were applied in several increments to a laterally restrained 1-inch-high sample.

Loads were applied in geometric progression by doubling the previous load, and the resulting deformations were recorded at selected time intervals. The test samples were inundated at selected loads to evaluate the effects of a sudden increase in moisture content (hydro-consolidation potential). Results of the tests are graphically presented on Plates B-3 to B-6.

Direct Shear

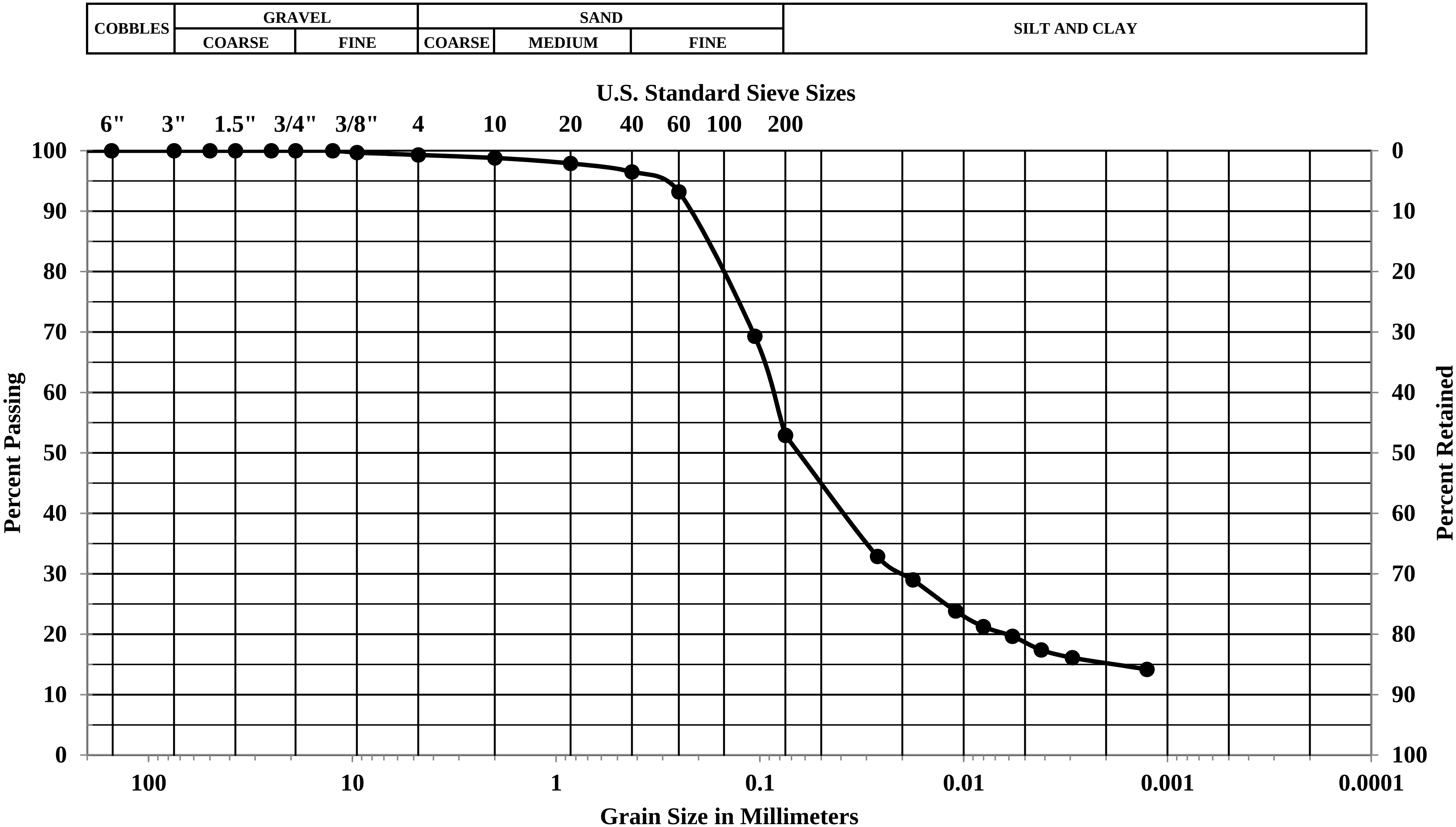
a bulk sample obtained from one our borings. The tests were performed in general conformance with Test Method ASTM D 3080. The sample was remolded to 90 percent of maximum dry density and at the optimum moisture content. Three specimens were prepared for each test, artificially saturated, and then sheared under varied loads at an appropriate constant rate of strain. Results are graphically presented on Plate B-7.

TABLE B-1
SUMMARY OF LABORATORY TEST RESULTS

Boring No.	Sample Depth (ft.)	Soil Description	Test Results	
B-1	0-5	Silty Clay (CL)	Maximum Dry Density (pcf): Optimum Moisture (%): Expansion Index: Soluble Sulfate Content: Liquid Limit: Plastic Index:	117.0 14.0 82 0.007 39 19

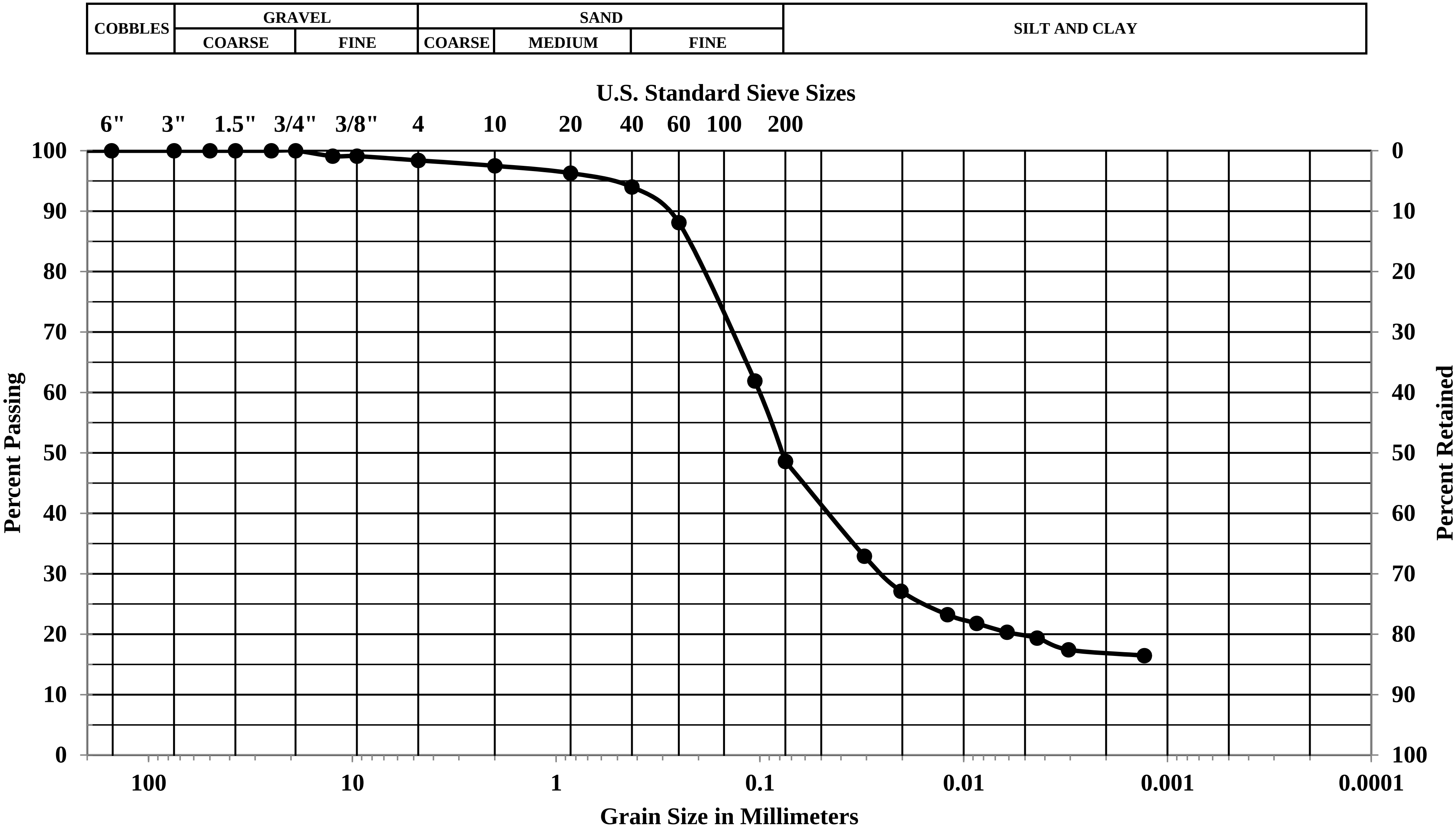
Note: Additional laboratory test results are provided on the boring logs in Appendix A.

GRAIN SIZE DISTRIBUTION



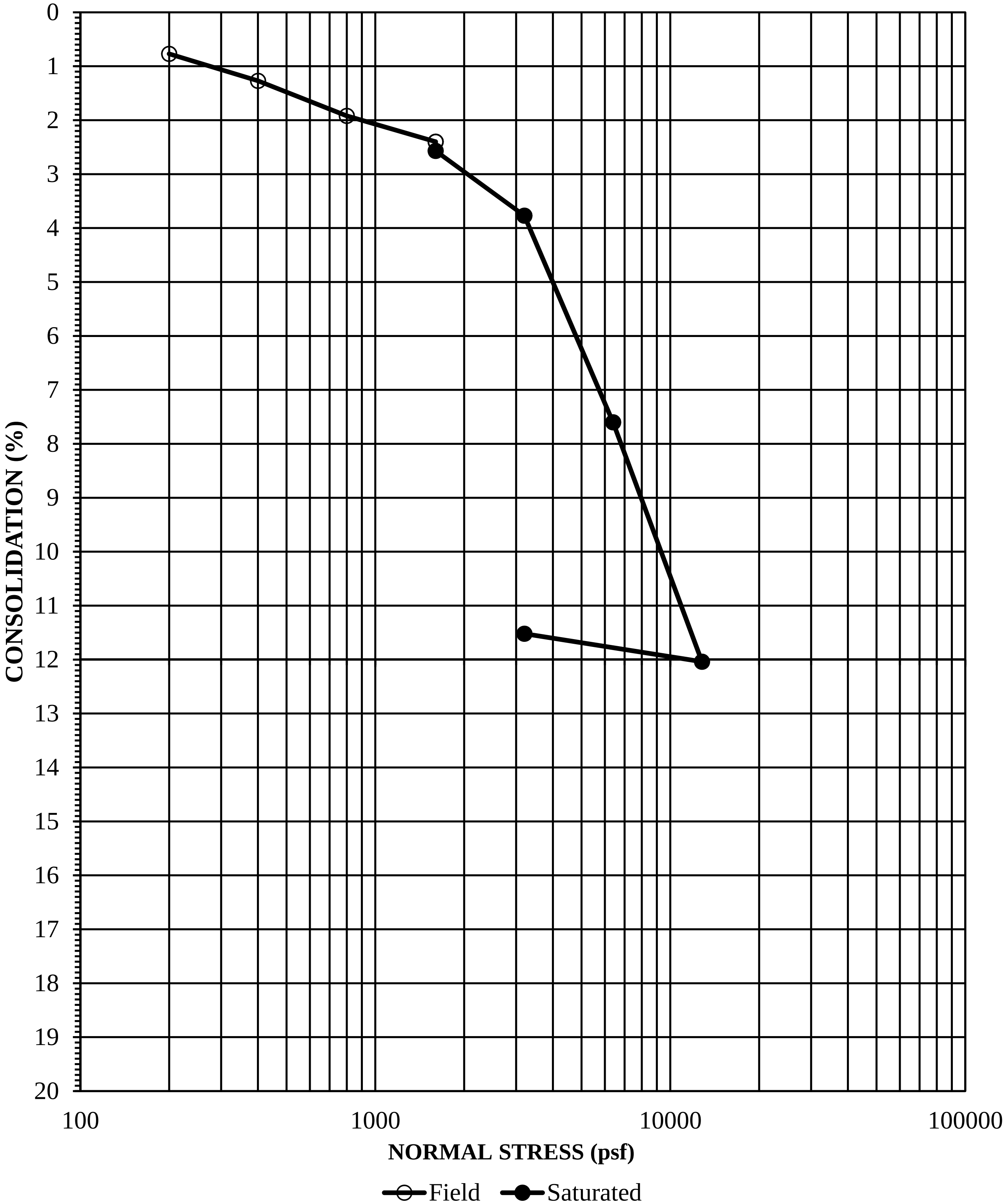
Job Number	Location	Depth	Description
2738.00	B-1	10	Sandy Silt (ML)

GRAIN SIZE DISTRIBUTION



Job Number	Location	Depth	Description
2738.00	B-1	15	Silty Sand (SM)

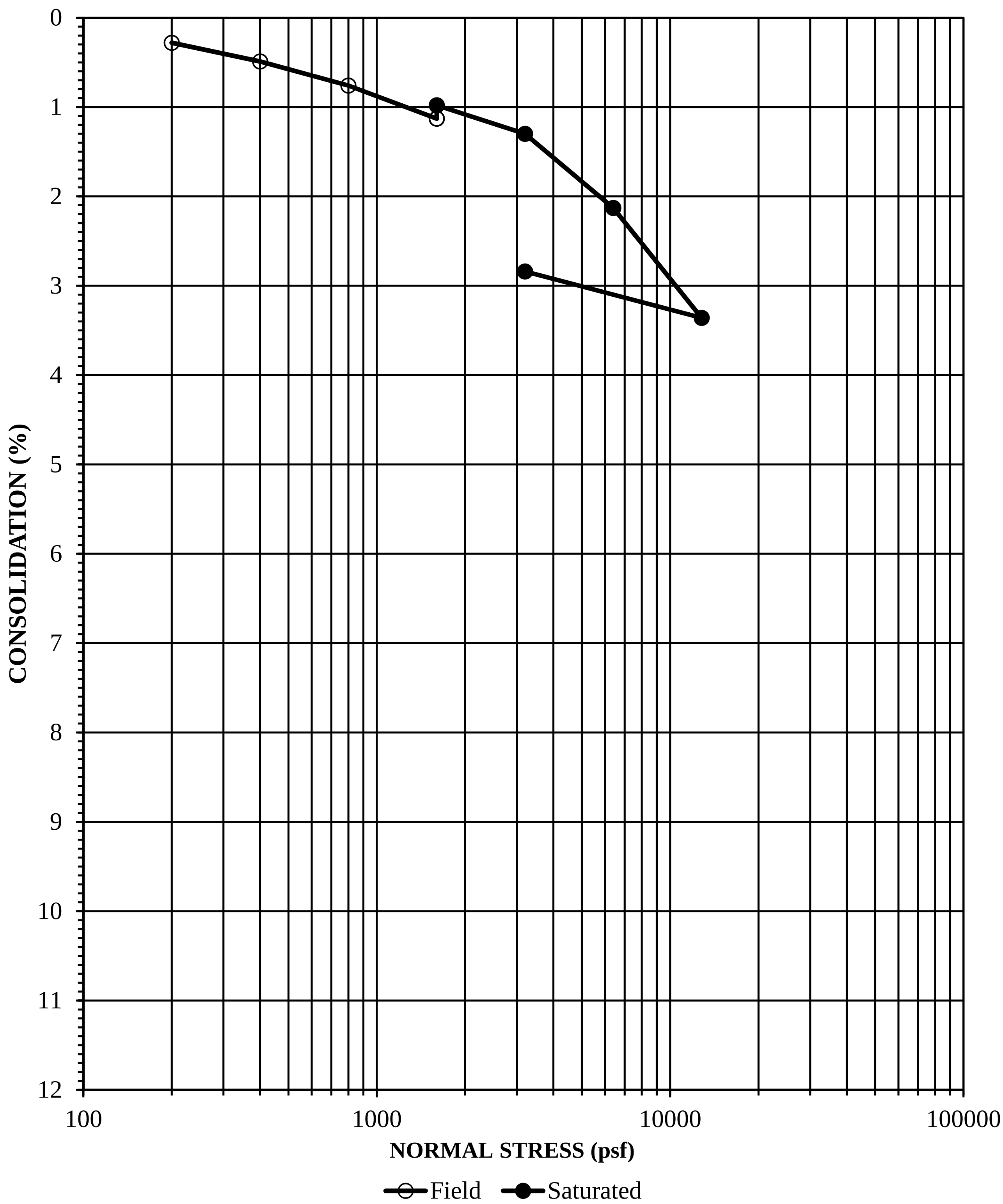
CONSOLIDATION



Job Number	Location	Depth	Description
2738.00	B-1	6	Silty Sand (SM)

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
96.2	9.9	20.7

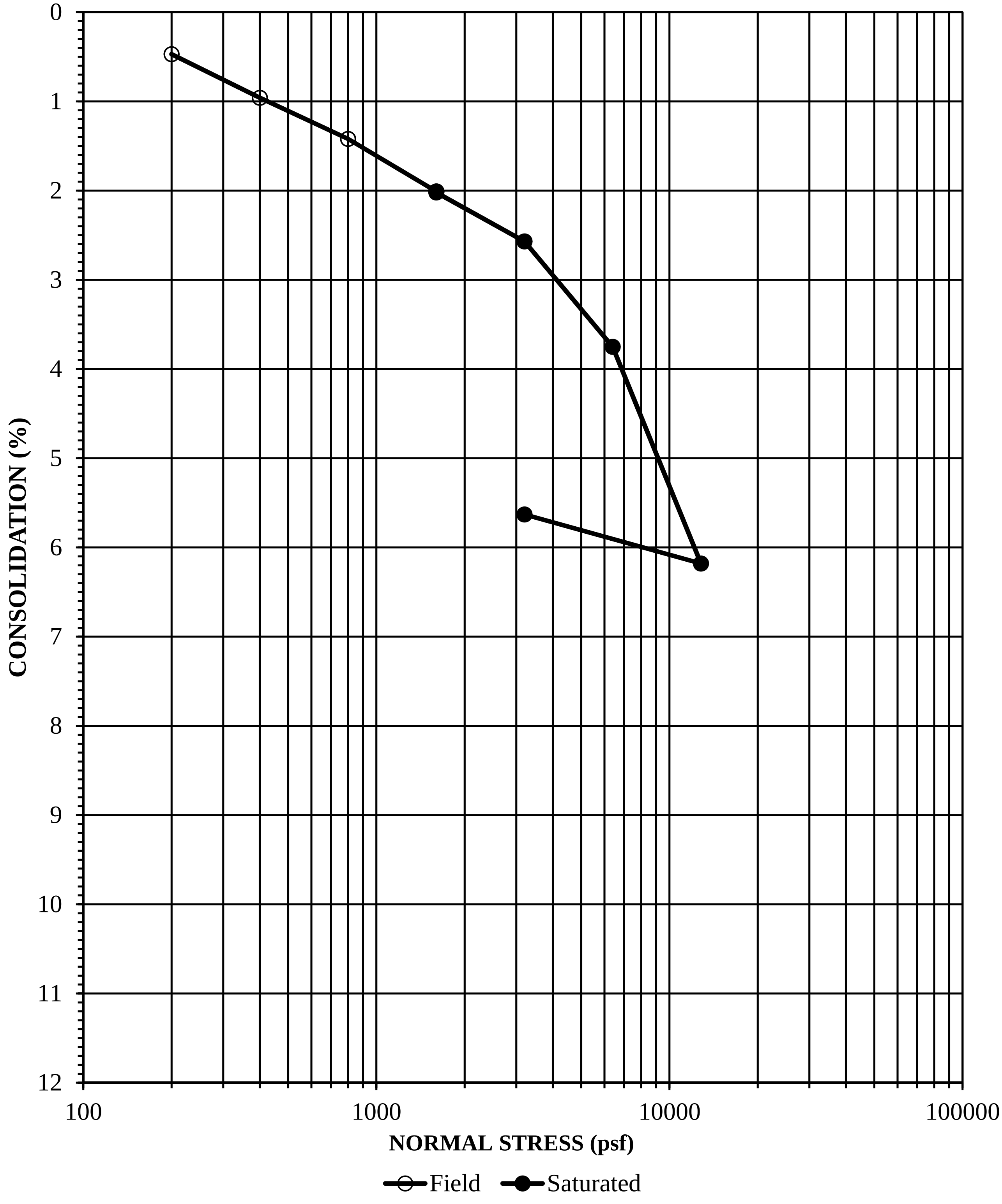
CONSOLIDATION



Job Number	Location	Depth	Description
2738.00	B-1	10	Sandy Silt (ML)

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
109.5	13.6	18.3

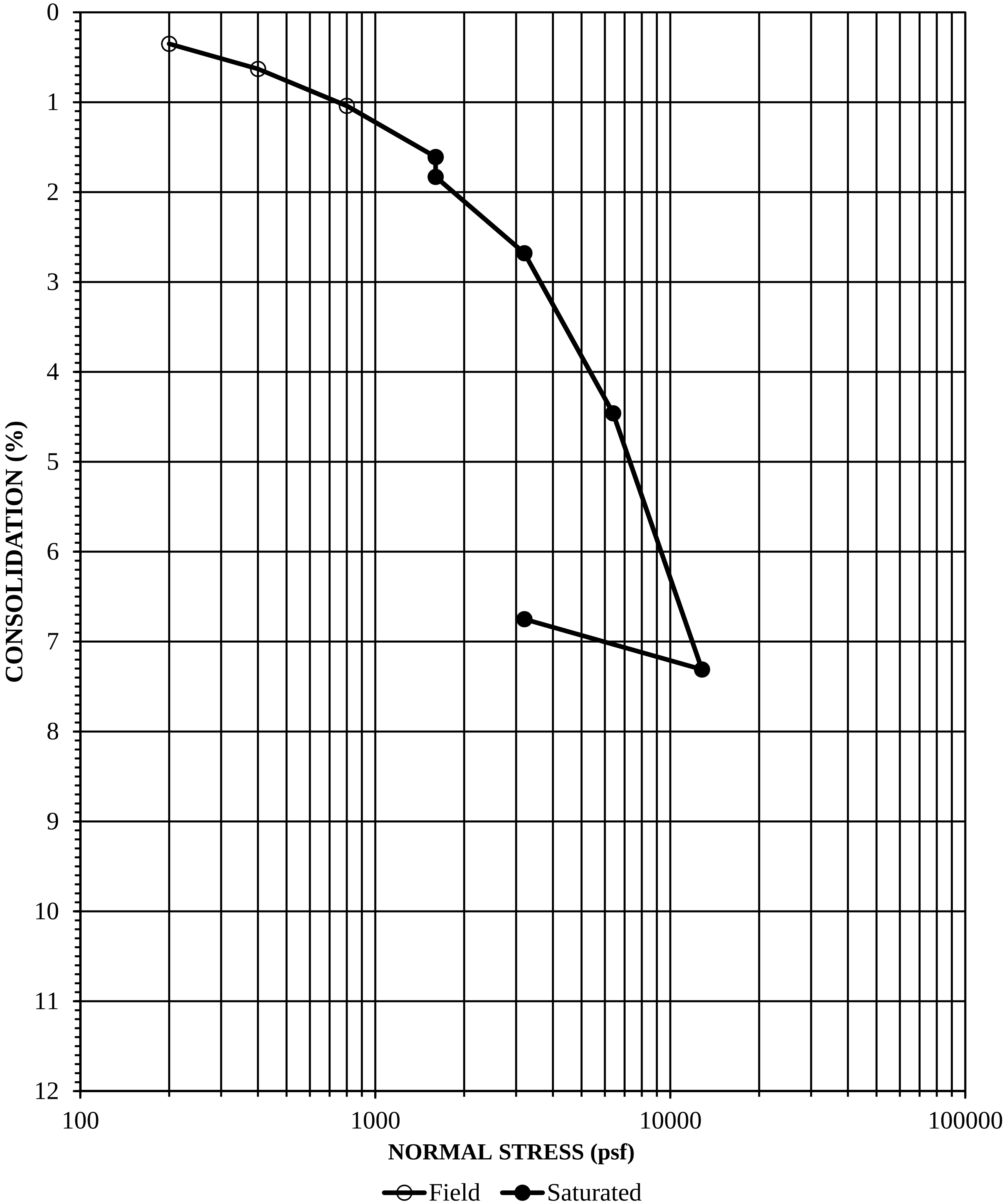
CONSOLIDATION



Job Number	Location	Depth	Description
2738.00	B-2	6	Silty Sand (SM)

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
103.4	18.3	19.7

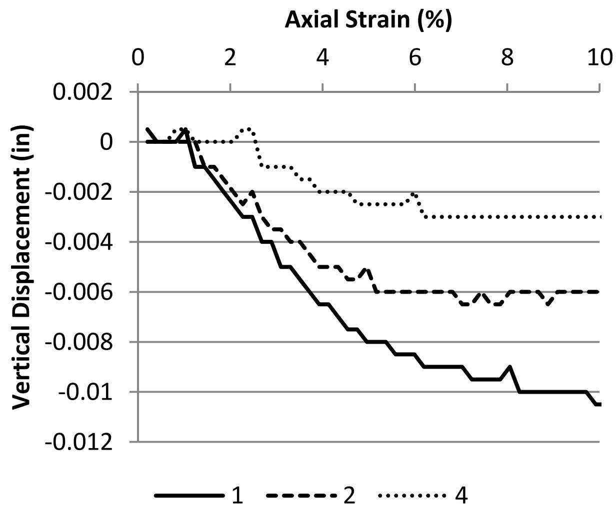
CONSOLIDATION

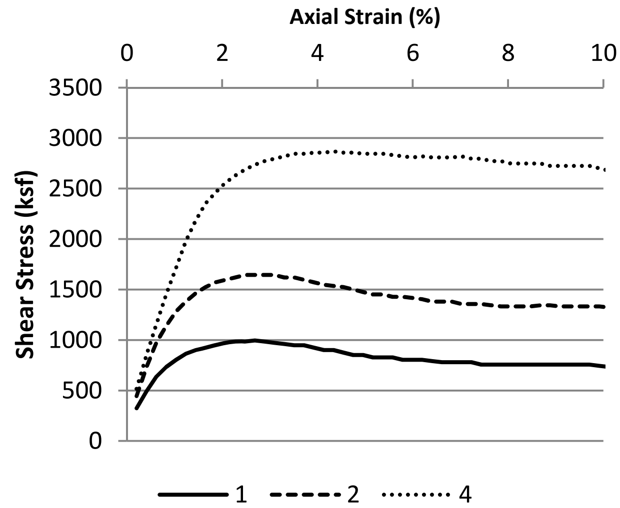


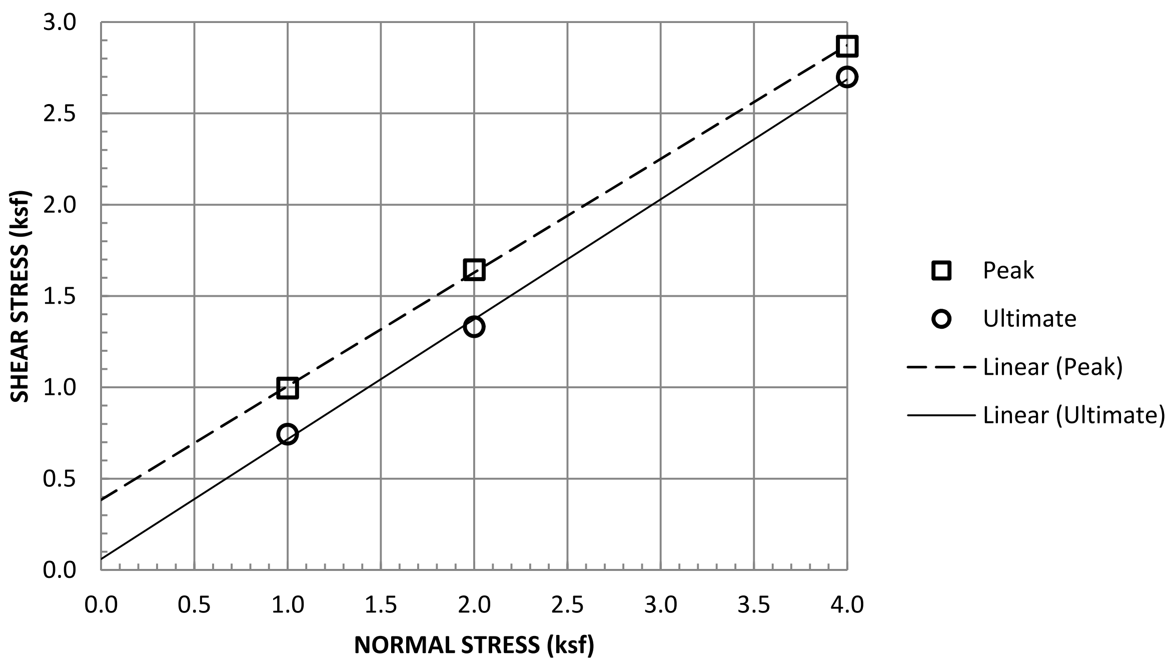
Job Number	Location	Depth	Description
2738.00	B-3	6	Silty Sand (SM)

Initial Dry Density (pcf)	Initial Moisture Content (%)	Final Moisture Concent (%)
105.8	16.9	18.9

DIRECT SHEAR







Sample Type:	Remolded, Saturated		
Normal Stress (ksf)	1	2	4
Peak Shear Stress (ksf)	0.996	1.644	2.868
Peak Displacement (in)	0.011	0.007	0.003
Ultimate Shear Stress (ksf)	0.744	1.332	2.7
Ultimate Displacement (in)	0.25	0.25	0.25
Initial Dry Density (pcf)	105.4	105.4	105.4
Initial Moisture Content (%)	14	14	14
Final Moisture Content (%)	19.6	20.4	20
Strain Rate (in/min)	0.005		

Job Number	Location	Depth	Description
2738.00	B-1	0-5	Clay (CL)

APPENDIX C

LIQUEFACTION ANALYSIS

TABLE C-1

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

Client: The Olson Company
J.N. 2738.00
Site: B-1

Hammer Type (D,S,A)	**A**	[Ce= D 0.75, S 0.95, A Hammer Efficiency]		
Boring Diameter, ID (in)	**4**			
Site Acceleration (g)	**0.862**	PGAm w/o MSF		
for a Magnitude (Mw) of	**6.76**	Corresponding to 2%PE in 50 yrs		
and MSF of	**1.36**		Analysis Type:	General
Depth to High GW	**25.0**	ft.	FS for Liquefaction:	**1.3**
Depth to GW during invest.	**33.0**	ft.	FS for Liqu. Settlement:	**1.3**
Hammer Efficiency	**84.1**	%	PI Threshold for Liquefaction:	**12**
Sublayer Thickness	**1.0**	ft.	Min. Moisture Cnt for Liqu. (%LL)	**85**
Depth of Analysis	**50.0**	ft.	Max FS for Plotting:	**5.0**

Layer Label (Auto)	**Depth Interval (ft)**		**Layer Mid-Depth (ft)**	**Soil Type (USCS)**	**Fines <#200 Sieve (%)**	**LL (%)**	**PI**	**M (%)**	**Field Nf (bls/ft)**	**Sample Type SPT/CA**	**Soil Wet Density (pcf)**
	Top	**Bottom**									
1	0.0	**4.0**	2.0	**CL**	**60**	**30**	**19**	**11.3**	**28**	**CA**	**107**
2	4.0	**5.0**	4.5	**CL**	**60**	**30**	**19**	**8.1**	**29**	**CA**	**107**
3	5.0	**10.0**	7.5	**SM**	**50**			**9.4**	**19**	**CA**	**116**
4	10.0	**15.0**	12.5	**SM**	**50**			**14.6**	**16**	**CA**	**118**
5	15.0	**20.0**	17.5	**SM**	**50**			**14.6**	**9**	**SPT**	**120**
6	20.0	**23.0**	21.5	**SM**	**50**			**14.6**	**14**	**SPT**	**120**
7	23.0	**26.0**	24.5	**SP**	**10**			**14.6**	**40**	**SPT**	**125**
8	26.0	**30.0**	28.0	**CL**	**60**			**20**	**20**	**SPT**	**120**
9	30.0	**35.0**	32.5	**CL**	**60**			**20**	**23**	**SPT**	**120**
10	35.0	**40.0**	37.5	**CL**	**60**			**20**	**21**	**SPT**	**120**
11	40.0	**45.0**	42.5	**CL**	**60**			**20**	**24**	**SPT**	**120**
12	45.0	**50.0**	47.5	**CL**	**60**			**20**	**26**	**SPT**	**120**
13	50.0	**51.5**	50.8	**CL**	**60**			**20**	**26**	**SPT**	**120**
14	51.5										
15	51.6										

TABLE C-2

ANALYSIS OF LIQUEFACTION POTENTIAL

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

CL

Client: The Olson Company

J.N. 2738.00

Site: B-1

Hammer Type (D,S,A)	A	
Boring Diameter, ID (in)	4	
Site Acceleration (g)	0.862	
for a Magnitude (Mw) of	6.76	
and MSF of	1.36	
Depth to High GW	25	ft.
Depth to GW during invest.	33	ft.
Hammer Efficiency	84.1	%
Sublayer Thickness	1	ft.
Depth of Boring	50	ft.

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

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

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

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

Layer Label	Depth Interval (ft) Top	Bottom	Layer Mid-Depth (ft)	Soil Type (USCS)	Fines <#200 Sieve (%)	LL (%)	PI	M (%)	Field Nf (bls/ft)	Sample Type SPT/CA	Soil Wet Density (pcf)	Total Stress (psf)(1)	Effec. Stress (psf)(1)	C_n	C_e	C_b	C_r	C_L	$(N_1)_{60}$ (lbs/ft)	α	β	$(N_1)_{60\text{-cs}}$ (lbs/ft)	Effec. Stress (psf)(2)	R_d	CRR (M=7.5)	Kσ	CSR	FS (3)	To Liquefy Y/N?
																												1.30	
1	0.0	1.0	0.5	CL	60	30	19	11.3	28	CA	107	54	54	1.7	1.40	1.00	0.75	1.0	38.0	5.0	1.20	50.6	54	1.00	NA	1.00	0.56	NA	N
1	1.0	2.0	1.5	CL	60	30	19	11.3	28	CA	107	161	161	1.7	1.40	1.00	0.75	1.0	38.0	5.0	1.20	50.6	161	1.00	NA	1.00	0.56	NA	N
1	2.0	3.0	2.5	CL	60	30	19	11.3	28	CA	107	268	268	1.7	1.40	1.00	0.75	1.0	37.1	5.0	1.20	49.5	268	0.99	NA	1.00	0.56	NA	N
1	3.0	4.0	3.5	CL	60	30	19	11.3	28	CA	107	375	375	1.6	1.40	1.00	0.75	1.0	35.7	5.0	1.20	47.8	375	0.99	NA	1.00	0.56	NA	N
2	4.0	5.0	4.5	CL	60	30	19	8.1	29	CA	107	483	483	1.5	1.40	1.00	0.75	1.0	35.6	5.0	1.20	47.7	483	0.99	NA	1.00	0.56	NA	N
3	5.0	6.0	5.5	SM	50			9.4	19	CA	116	635	635	1.5	1.40	1.00	0.75	1.0	16.4	5.0	1.20	24.6	635	0.99	NA	1.00	0.56	NA	N
3	6.0	7.0	6.5	SM	50			9.4	19	CA	116	751	751	1.4	1.40	1.00	0.80	1.0	16.8	5.0	1.20	25.2	751	0.99	NA	1.00	0.56	NA	N
3	7.0	8.0	7.5	SM	50			9.4	19	CA	116	866	866	1.4	1.40	1.00	0.80	1.0	16.3	5.0	1.20	24.5	866	0.98	NA	1.00	0.56	NA	N
3	8.0	9.0	8.5	SM	50			9.4	19	CA	116	982	982	1.3	1.40	1.00	0.80	1.0	15.7	5.0	1.20	23.9	982	0.98	NA	1.00	0.54	NA	N
3	9.0	10.0	9.5	SM	50			9.4	19	CA	116	1097	1097	1.3	1.40	1.00	0.85	1.0	16.2	5.0	1.20	24.4	1097	0.98	NA	1.00	0.54	NA	N
4	10.0	11.0	10.5	SM	50			14.6	16	CA	118	1239	1239	1.2	1.40	1.00	0.85	1.0	13.1	5.0	1.20	20.7	1239	0.98	NA	1.00	0.54	NA	N
4	11.0	12.0	11.5	SM	50			14.6	16	CA	118	1357	1357	1.2	1.40	1.00	0.85	1.0	12.7	5.0	1.20	20.2	1357	0.97	NA	1.00	0.54	NA	N
4	12.0	13.0	12.5	SM	50			14.6	16	CA	118	1475	1475	1.2	1.40	1.00	0.85	1.0	12.3	5.0	1.20	19.8	1475	0.97	NA	1.00	0.54	NA	N
4	13.0	14.0	13.5	SM	50			14.6	16	CA	118	1593	1593	1.1	1.40	1.00	0.85	1.0	12.0	5.0	1.20	19.4	1593	0.97	NA	1.00	0.54	NA	N
4	14.0	15.0	14.5	SM	50			14.6	16	CA	118	1711	1711	1.1	1.40	1.00	0.85	1.0	11.6	5.0	1.20	19.0	1711	0.97	NA	1.00	0.54	NA	N
5	15.0	16.0	15.5	SM	50			14.6	9	SPT	120	1860	1860	1.1	1.40	1.00	0.85	1.2	13.5	5.0	1.20	21.2	1860	0.96	NA	1.00	0.54	NA	N
5	16.0	17.0	16.5	SM	50			14.6	9	SPT	120	1980	1980	1.0	1.40	1.00	0.90	1.2	14.0	5.0	1.20	21.7	1980	0.96	NA	1.00	0.54	NA	N
5	17.0	18.0	17.5	SM	50			14.6	9	SPT	120	2100	2100	1.0	1.40	1.00	0.90	1.2	13.6	5.0	1.20	21.3	2100	0.96	NA	1.01	0.54	NA	N
5	18.0	19.0	18.5	SM	50			14.6	9	SPT	120	2220	2220	1.0	1.40	1.00	0.90	1.2	13.2	5.0	1.20	20.9	2220	0.96	NA	1.00	0.54	NA	N
5	19.0	20.0	19.5	SM	50			14.6	9	SPT	120	2340	2340	0.9	1.40	1.00	0.90	1.2	12.9	5.0	1.20	20.5	2340	0.96	NA	0.99	0.54	NA	N
6	20.0	21.0	20.5	SM	50			14.6	14	SPT	120	2460	2460	0.9	1.40	1.00	0.90	1.2	19.6	5.0	1.20	28.5	2460	0.95	NA	0.98	0.54	NA	N
6	21.0	22.0	21.5	SM	50			14.6	14	SPT	120	2580	2580	0.9	1.40	1.00	0.90	1.2	19.1	5.0	1.20	28.0	2580	0.95	NA	0.97	0.54	NA	N
6	22.0	23.0	22.5	SM	50			14.6	14	SPT	120	2700	2700	0.9	1.40	1.00	0.95	1.2	19.7	5.0	1.20	28.7	2700	0.95	NA	0.96	0.54	NA	N
7	23.0	24.0	23.5	SP	10			14.6	40	SPT	125	2938	2938	0.8	1.40	1.00	0.95	1.2	53.9	0.9	1.02	56.0	2938	0.95	NA	0.95	0.54	NA	N
7	24.0	25.0	24.5	SP	10			14.6	40	SPT	125	3063	3063	0.8	1.40	1.00	0.95	1.2	52.7	0.9	1.02	54.7	3063	0.94	NA	0.94	0.52	NA	N
7	25.0	26.0	25.5	SP	10			14.6	40	SPT	125	3188	3188	0.8	1.40	1.00	0.95	1.2	51.6	0.9	1.02	53.6	3156	0.94	NA	0.93	0.54	NA	N
8	26.0	27.0	26.5	CL	60			20	20	SPT	120	3180	3180	0.8	1.40	1.00	0.95	1.2	25.8	5.0	1.20	36.0	3086	0.94	NA	0.93	0.54	NA	N
8	27.0	28.0	27.5	CL	60			20	20	SPT	120	3300	3300	0.8	1.40	1.00	0.95	1.2	25.3	5.0	1.20	35.3	3144	0.94	NA	0.92	0.56	NA	N
8	28.0	29.0	28.5	CL	60			20	20	SPT	120	3420	3420	0.8	1.40	1.00	0.95	1.2	24.8	5.0	1.20	34.7	3202	0.93	NA	0.92	0.56	NA	N
8	29.0	30.0	29.5	CL	60			20	20	SPT	120	3540	3540	0.8	1.40	1.00	1.00	1.2	25.6	5.0	1.20	35.7	3259	0.93	NA	0.91	0.56	NA	N
9	30.0	31.0	30.5	CL	60			20	23	SPT	120	3660	3660	0.7	1.40	1.00	1.00	1.2	28.8	5.0	1.20	39.6	3317	0.93	NA	0.90	0.58	NA	N
9	31.0	32.0	31.5	CL	60			20	23	SPT	120	3780	3780	0.7	1.40	1.00	1.00	1.2	28.3	5.0	1.20	38.9	3374	0.92	NA	0.89	0.58	NA	N
9	32.0	33.0	32.5	CL	60			20	23	SPT	120	3900	3900	0.7	1.40	1.00	1.00	1.2	27.7	5.0	1.20	38.3	3432	0.91	NA	0.89	0.58	NA	N
9	33.0	34.0	33.5	CL	60			20	23	SPT	120	4020	3989	0.7	1.40	1.00	1.00	1.2	27.4	5.0	1.20	37.8	3490	0.90	NA	0.88	0.58	NA	N
9	34.0	35.0	34.5	CL	60			20	23	SPT	120	4140	4046	0.7	1.40	1.00	1.00	1.2	27.1	5.0	1.20	37.6	3547	0.89	NA	0.88	0.58	NA	N
10	35.0	36.0	35.5	CL	60			20	21	SPT	120	4260	4104	0.7	1.40	1.00	1.00	1.2	24.6	5.0	1.20	34.5	3605	0.89	NA	0.88	0.58	NA	N
10	36.0	37.0	36.5	CL	60			20	21	SPT	120	4380	4162	0.7	1.40	1.00	1.00	1.2	24.3	5.0	1.20	34.2	3662	0.88	NA	0.87	0.58	NA	N
10	37.0	38.0	37.5	CL	60			20	21	SPT	120	4500	4219	0.7	1.40	1.00	1.00	1.2	24.1	5.0	1.20	34.0	3720	0.87	NA	0.87	0.58	NA	N
10	38.0	39.0	38.5	CL	60			20	21	SPT	120	4620	4277	0.7	1.40	1.00	1.00	1.2	23.9	5.0	1.20	33.7	3778	0.86	NA	0.87	0.60	NA	N
10	39.0	40.0	39.5	CL	60			20	21	SPT	120	4740	4334	0.7	1.40	1.00	1.00	1.2	23.7	5.0	1.20	33.5	3835	0.85	NA	0.87	0.60	NA	N
11	40.0	41.0	40.5	CL	60			20	24	SPT	120	4860	4392	0.7	1.40	1.00	1.00	1.2	26.9	5.0	1.20	37.3	3893	0.85	NA	0.86	0.60	NA	N
11	41.0	42.0	41.5	CL	60			20	24	SPT	120	4980	4450	0.7	1.40	1.00	1.00	1.2	26.7	5.0	1.20	37.0	3950	0.84	NA	0.86	0.60	NA	N
11	42.0	43.0	42.5	CL	60			20	24	SPT	120	5100	4507	0.7	1.40	1.00	1.00	1.2	26.4	5.0	1.20	36.7	4008	0.83	NA	0.86	0.60	NA	N
11	43.0	44.0	43.5	CL	60			20	24	SPT	120	5220	4565	0.6	1.40	1.00	1.00	1.2	26.2	5.0	1.20	36.5	4066	0.82	NA	0.85	0.60	NA	N
11	44.0	45.0	44.5	CL	60			20	24	SPT	120	5340	4622	0.6	1.40	1.00	1.00	1.2	26.0	5.0	1.20	36.2	4123	0.81	NA	0.85	0.60	NA	N
12	45.0	46.0	45.5	CL	60			20	26	SPT	120	5460	4680	0.6	1.40	1.00	1.00	1.2	28.0	5.0	1.20	38.6	4181	0.81	NA	0.85	0.58	NA	N
12	46.0	47.0	46.5	CL	60			20	26	SPT	120	5580	4738	0.6	1.40	1.00	1.00	1.2	27.7	5.0	1.20	38.3	4238	0.80	NA	0.85	0.58	NA	N
12	47.0	48.0	47.5	CL	60			20	26	SPT	120	5700	4795	0.6	1.40	1.00	1.00	1.2	27.5	5.0	1.20	38.0	4296	0.79	NA	0.84	0.58	NA	N
12	48.0	49.0	48.5	CL	60			20	26	SPT	120	5820	4853	0.6	1.40	1.00	1.00	1.2	27.3	5.0	1.20	37.8	4354	0.78	NA	0.84	0.58	NA	N
12	49.0	50.0	49.5	CL	60			20	26	SPT	120	5940	4910	0.6	1.40	1.00	1.00	1.2	27.1	5.0	1.20	37.5	4411	0.77	NA	0.84	0.58	NA	N

Plate C-2

ALBUS ASSOCIATES, INC.

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

Client: The Olson Company

J.N. 2738.00

Site: B-1

Notes:

(1) Effective ER=55% normalized standard penetration resistance for clean sands, $(N_1)_{60\text{-cs}}$*1.1 (Seed, 1994).

(2) Volumetric strain (Ishihara and Yoshimine, 1992) using $(N_1)_{55\text{-cs}}$.

(3) Volumetric strain (Tokimatsu and Seed, 1987) using $(N_1)_{60\text{-cs}}$.

Depth Interval (ft)		Soil layer thickness (ft)	Fines <#200 Sieve (%)	$(N_1)_{60\text{-cs}}$	$(N_1)_{55\text{-cs}}$ (1)	FS	IY Percent ε_v (2)	CSR*	TS Percent ε_v (3)	IY δ (in.)	TS δ (in.)	Ave δ (in.)
									Total δ (in.)	0.00	0.00	0.00
Top	Bottom											
0.00	1.00	1.00	60	50.6	55.7	NA	0.00	0.56	NA	NA	NA	0
1.00	2.00	1.00	60	50.6	55.7	NA	0.00	0.56	NA	NA	NA	0
2.00	3.00	1.00	60	49.5	54.4	NA	0.00	0.56	NA	NA	NA	0
3.00	4.00	1.00	60	47.8	52.6	NA	0.00	0.56	NA	NA	NA	0
4.00	5.00	1.00	60	47.7	52.5	NA	0.00	0.56	NA	NA	NA	0
5.00	6.00	1.00	50	24.6	27.1	NA	0.00	0.56	NA	NA	NA	0
6.00	7.00	1.00	50	25.2	27.7	NA	0.00	0.56	NA	NA	NA	0
7.00	8.00	1.00	50	24.5	27.0	NA	0.00	0.56	NA	NA	NA	0
8.00	9.00	1.00	50	23.9	26.2	NA	0.00	0.54	NA	NA	NA	0
9.00	10.00	1.00	50	24.4	26.8	NA	0.00	0.54	NA	NA	NA	0
10.00	11.00	1.00	50	20.7	22.8	NA	0.00	0.54	NA	NA	NA	0
11.00	12.00	1.00	50	20.2	22.3	NA	0.00	0.54	NA	NA	NA	0
12.00	13.00	1.00	50	19.8	21.8	NA	0.00	0.54	NA	NA	NA	0
13.00	14.00	1.00	50	19.4	21.3	NA	0.00	0.54	NA	NA	NA	0
14.00	15.00	1.00	50	19.0	20.9	NA	0.00	0.54	NA	NA	NA	0
15.00	16.00	1.00	50	21.2	23.4	NA	0.00	0.54	NA	NA	NA	0
16.00	17.00	1.00	50	21.7	23.9	NA	0.00	0.54	NA	NA	NA	0
17.00	18.00	1.00	50	21.3	23.4	NA	0.00	0.54	NA	NA	NA	0
18.00	19.00	1.00	50	20.9	23.0	NA	0.00	0.54	NA	NA	NA	0
19.00	20.00	1.00	50	20.5	22.6	NA	0.00	0.54	NA	NA	NA	0
20.00	21.00	1.00	50	28.5	31.4	NA	0.00	0.54	NA	NA	NA	0
21.00	22.00	1.00	50	28.0	30.8	NA	0.00	0.54	NA	NA	NA	0
22.00	23.00	1.00	50	28.7	31.6	NA	0.00	0.54	NA	NA	NA	0
23.00	24.00	1.00	10	56.0	61.6	NA	0.00	0.54	NA	NA	NA	0
24.00	25.00	1.00	10	54.7	60.2	NA	0.00	0.52	NA	NA	NA	0
25.00	26.00	1.00	10	53.6	58.9	NA	0.00	0.54	NA	NA	NA	0
26.00	27.00	1.00	60	36.0	39.6	NA	0.00	0.54	NA	NA	NA	0
27.00	28.00	1.00	60	35.3	38.9	NA	0.00	0.56	NA	NA	NA	0
28.00	29.00	1.00	60	34.7	38.2	NA	0.00	0.56	NA	NA	NA	0
29.00	30.00	1.00	60	35.7	39.2	NA	0.00	0.56	NA	NA	NA	0
30.00	31.00	1.00	60	39.6	43.6	NA	0.00	0.58	NA	NA	NA	0
31.00	32.00	1.00	60	38.9	42.8	NA	0.00	0.58	NA	NA	NA	0
32.00	33.00	1.00	60	38.3	42.1	NA	0.00	0.58	NA	NA	NA	0
33.00	34.00	1.00	60	37.8	41.6	NA	0.00	0.58	NA	NA	NA	0
34.00	35.00	1.00	60	37.6	41.3	NA	0.00	0.58	NA	NA	NA	0
35.00	36.00	1.00	60	34.5	37.9	NA	0.00	0.58	NA	NA	NA	0
36.00	37.00	1.00	60	34.2	37.6	NA	0.00	0.58	NA	NA	NA	0
37.00	38.00	1.00	60	34.0	37.4	NA	0.00	0.58	NA	NA	NA	0
38.00	39.00	1.00	60	33.7	37.1	NA	0.00	0.60	NA	NA	NA	0
39.00	40.00	1.00	60	33.5	36.8	NA	0.00	0.60	NA	NA	NA	0
40.00	41.00	1.00	60	37.3	41.0	NA	0.00	0.60	NA	NA	NA	0
41.00	42.00	1.00	60	37.0	40.7	NA	0.00	0.60	NA	NA	NA	0
42.00	43.00	1.00	60	36.7	40.4	NA	0.00	0.60	NA	NA	NA	0
43.00	44.00	1.00	60	36.5	40.1	NA	0.00	0.60	NA	NA	NA	0
44.00	45.00	1.00	60	36.2	39.8	NA	0.00	0.60	NA	NA	NA	0
45.00	46.00	1.00	60	38.6	42.4	NA	0.00	0.58	NA	NA	NA	0
46.00	47.00	1.00	60	38.3	42.1	NA	0.00	0.58	NA	NA	NA	0
47.00	48.00	1.00	60	38.0	41.8	NA	0.00	0.58	NA	NA	NA	0
48.00	49.00	1.00	60	37.8	41.5	NA	0.00	0.58	NA	NA	NA	0
49.00	50.00	1.00	60	37.5	41.3	NA	0.00	0.58	NA	NA	NA	0

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

Client: The Olson Company
J.N. 2738.00
Site: B-1

Total Seismic Settlement of Unsaturated Soil w/ FS=2.0 (in):	2.39
Subtotal Seismic Settlement of Unsaturated Soil (in)	1.20
Total Thickness of Unsaturated Soil (ft)	25.0

GW Depth: 25 feet
EQ Magnitude 6.76
MSF 1.36

Layer Mid-Depth (ft.)	Soil Type	Eff. Stress σ'_{vo}(tsf)	Clean Sand $(N_1)_{60}$	CSR	(psf) τ_{avg} Avg. Shear Stress	(tsf) σ_m' Mean Bulk Stress	(tsf) G_{max} Max. Dyn.Shr. Mod.	γ_{eff} (G_{eff}/G_{max})	Eff. Cyclic Shr.Strain γ_{eff} Fig.11	Eff. Cyclic Shr.Strain γ_{eff} (%)	Volume Strain (%) Fig.13	EQ Mag. Factor	Layer Thickness (ft.)	Estimated Dry Sand Seismic Settlement (in.)
0.5	CL	0.03	50.6	0.56	30.0	0.02	215.3	6.96E-05	1.22E-04	1.22E-02	3.70E-03	1.36	1.0	0.000
1.5	CL	0.08	50.6	0.56	89.9	0.05	373.0	1.20E-04	3.41E-04	3.41E-02	1.04E-02	1.36	1.0	0.000
2.5	CL	0.13	49.5	0.56	149.8	0.09	477.8	1.57E-04	6.38E-04	6.38E-02	1.94E-02	1.36	1.0	0.000
3.5	CL	0.19	47.8	0.56	209.7	0.12	559.0	1.88E-04	1.78E-03	1.78E-01	5.41E-02	1.36	1.0	0.000
4.5	CL	0.24	47.7	0.56	270.2	0.16	634.3	2.13E-04	4.56E-03	4.56E-01	1.39E-01	1.36	1.0	0.000
5.5	SM	0.32	24.6	0.56	355.7	0.21	585.0	3.04E-04	5.49E-03	5.49E-01	4.06E-01	1.36	1.0	0.036
6.5	SM	0.38	25.2	0.56	420.4	0.24	640.7	3.28E-04	8.85E-03	8.85E-01	6.35E-01	1.36	1.0	0.056
7.5	SM	0.43	24.5	0.56	485.1	0.28	681.9	3.56E-04	9.12E-03	9.12E-01	6.78E-01	1.36	1.0	0.060
8.5	SM	0.49	23.9	0.54	530.1	0.32	719.6	3.68E-04	8.54E-03	8.54E-01	6.56E-01	1.36	1.0	0.058
9.5	SM	0.55	24.4	0.54	592.5	0.36	766.4	3.87E-04	8.21E-03	8.21E-01	6.14E-01	1.36	1.0	0.054
10.5	SM	0.62	20.7	0.54	669.1	0.40	771.6	4.34E-04	9.72E-03	9.72E-01	8.80E-01	1.36	1.0	0.078
11.5	SM	0.68	20.2	0.54	732.8	0.44	801.2	4.57E-04	9.97E-03	9.97E-01	9.25E-01	1.36	1.0	0.082
12.5	SM	0.74	19.8	0.54	796.5	0.48	829.2	4.80E-04	1.00E-02	1.00E+00	9.48E-01	1.36	1.0	0.084
13.5	SM	0.80	19.4	0.54	860.2	0.52	855.5	5.03E-04	1.00E-02	1.00E+00	9.65E-01	1.36	1.0	0.085
14.5	SM	0.86	19.0	0.54	923.9	0.56	880.5	5.25E-04	1.00E-02	1.00E+00	9.80E-01	1.36	1.0	0.087
15.5	SM	0.93	21.2	0.54	1004.4	0.60	953.3	5.27E-04	9.91E-03	9.91E-01	8.73E-01	1.36	1.0	0.077
16.5	SM	0.99	21.7	0.54	1069.2	0.64	991.1	5.39E-04	9.82E-03	9.82E-01	8.43E-01	1.36	1.0	0.074
17.5	SM	1.05	21.3	0.54	1134.0	0.68	1013.9	5.59E-04	1.00E-02	1.00E+00	8.78E-01	1.36	1.0	0.078
18.5	SM	1.11	20.9	0.54	1198.8	0.72	1035.7	5.79E-04	1.00E-02	1.00E+00	8.97E-01	1.36	1.0	0.079
19.5	SM	1.17	20.5	0.54	1263.6	0.76	1056.7	5.98E-04	1.00E-02	1.00E+00	9.16E-01	1.36	1.0	0.081
20.5	SM	1.23	28.5	0.54	1328.4	0.80	1208.3	5.50E-04	7.37E-03	7.37E-01	4.45E-01	1.36	1.0	0.039
21.5	SM	1.29	28.0	0.54	1393.2	0.84	1229.4	5.67E-04	7.34E-03	7.34E-01	4.56E-01	1.36	1.0	0.040
22.5	SM	1.35	28.7	0.54	1458.0	0.88	1268.2	5.75E-04	6.90E-03	6.90E-01	4.13E-01	1.36	1.0	0.036
23.5	SP	1.47	56.0	0.54	1586.3	0.95	1649.3	4.81E-04	2.63E-03	2.63E-01	8.00E-02	1.36	1.0	0.007
24.5	SP	1.53	54.7	0.52	1592.5	1.00	1671.7	4.76E-04	2.03E-03	2.03E-01	6.17E-02	1.36	1.0	0.005