



LGC Valley, Inc.

Geotechnical Consulting

***PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT
FOR THE PROPOSED RESIDENTIAL DEVELOPMENT
AT 6501 S. SEPULVEDA ROAD,
CITY OF LOS ANGELES, CALIFORNIA***

Site Address: 6501 S. Sepulveda Road

Dated: October 5, 2020

Project No. 203022-01

Prepared For:

***FRH Realty, LLC
5355 Mira Sorrento Place, Suite 100
San Diego, California 92121***



LGC Valley, Inc.

Geotechnical Consulting

October 5, 2020

Project No. 203022-01

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Subject: Preliminary Geotechnical Investigation Report for the Proposed Residential Development at 6501 S. Sepulveda Road, City of Los Angeles, California

Site Address: 6501 S. Sepulveda Road, City of Los Angeles, California

In accordance with your request, LGC Valley, Inc. (LGC) is providing this preliminary geotechnical investigation report for an approximate 2.5-acre site located at the southwest corner of S. Sepulveda Road and W. Centinela Avenue in the city of Los Angeles, California. The purpose of our investigation was to evaluate the existing onsite geotechnical conditions, review geotechnical and geologic data and maps pertinent to the site, and prepare a geotechnical report, with respect to the proposed residential development, indicating our findings, conclusions, opinions, and recommendations for site development. This report presents the results of our subsurface investigations, and our geotechnical analysis of the collected data, and provides our conclusions, opinions and recommendations with respect to the proposed residential site development.

LGC has reviewed the laboratory test data, procedures and results performed by EGLAB, Inc. (EGL) with respect to the subject site and concurs with and accepts responsibility as geotechnical engineer of record for their work (laboratory testing).

If you have any questions regarding our report, please contact this office. We appreciate this opportunity to be of service.

Respectfully submitted,

LGC VALLEY, INC.

Basil Hattar, GE 2734
Principal Engineer



Matthew Hawley, CEG 2122
President



BIH/MCH/

Distribution: (6) Addressee

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

The main purpose of our geotechnical services was to provide a preliminary geotechnical investigation for an approximate 2.5-acre site located at the southwest corner of S. Sepulveda Road and W. Centinela Avenue in the city of Los Angeles, California. During preparation of this report, LGC identified and evaluated the existing geologic and geotechnical conditions at the site, and provide preliminary geotechnical design criteria for the proposed residential development at 6501 S. Sepulveda Road, City of Los Angeles, California. Recommendations for grading construction, preliminary foundation design for the proposed structures, retaining walls and other relevant aspects of the proposed development are included herein to address the identified site geotechnical constraints. This report includes the results of site exploration, laboratory testing and engineering evaluation, and provides our conclusions, opinions and recommendations with respect to site development.

These items plus other geotechnical conditions are discussed and addressed within this document.

Our scope of services for preparation of this document included:

- Review of geotechnical reports, geologic maps and other documents relevant to the site (Appendix A).
- Perform a site visit to evaluate the existing condition, and mark the geotechnical boring locations.
- A subsurface investigation including the excavation, sampling, and logging of three small-diameter exploratory borings. The borings are labeled B-1 through B-3. Logs of the borings are presented in Appendix B, and their approximate locations are depicted on the Exploration Location Map (Figure 2). All of the excavations were sampled and logged under the supervision of a representative from our firm. The borings were excavated to evaluate the general characteristics of the subsurface conditions on the site including classification of site soils, determination of depth to groundwater, and to obtain representative soil samples.
- Laboratory testing of representative soil samples obtained during our subsurface investigation (Appendix C).
- Perform geotechnical analyses and evaluation of the data.
- Preparation of this report presenting our findings, conclusions, opinions and recommendations with respect to the evaluated geologic and geotechnical conditions at the site.

1.2 Site and Project Description

The subject site is located at in the West Los Angeles Area at 6501 S. Sepulveda Boulevard in the City of Los Angeles. The site is bounded by Centinela Avenue to the north, Sepulveda Boulevard to the east, Arizona Avenue to the west, and an existing hotel to the south. Based on our review, the site is currently in use as a commercial/retail/restaurant development with parking and appurtenant structures. We assume the future grading of the site is anticipated to consist of minimal design cuts and fills to achieve finish grades.

It is our understanding that the proposed development will consist of a mixed-use residential/retail development consisting of an 8-story podium, with 3-levels concrete and 5-levels of wood construction above over one level of subterranean parking. The development will consist of approximately 374 residential units, and approximately 7,000 SF of retail space on the first level.

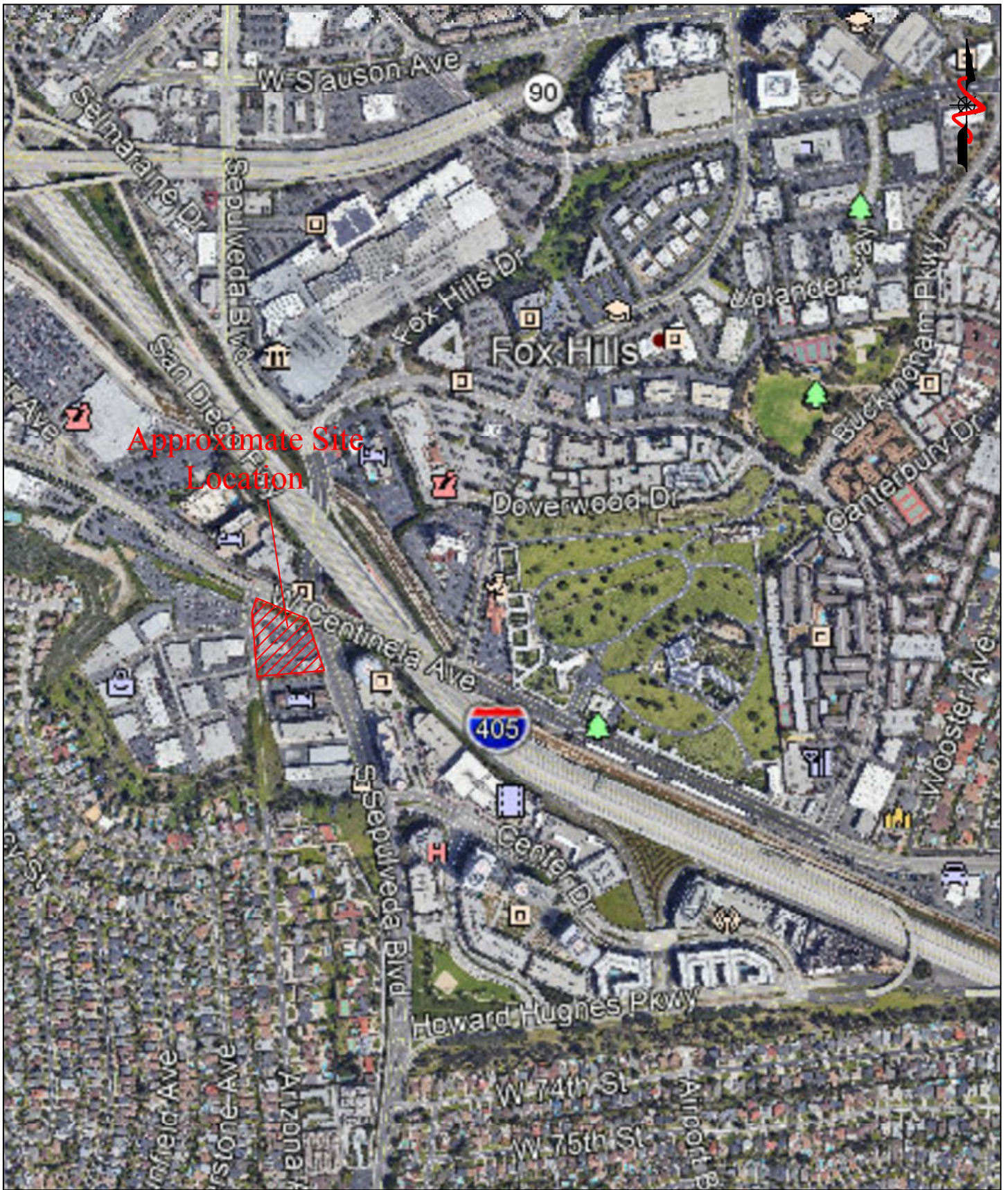
1.3 Subsurface Investigation and Laboratory Testing

Our subsurface investigation was performed on September 3, 2020, and consisted of three hollow stem auger borings (B-1 through B-3). The borings were extended to depths ranging from 31.5 to 51.5 feet. The approximate locations of the borings are shown on the Exploration Location Map (Figure 2). The exploration location map uses a preliminary base map of Level 1 of the proposed development, prepared by Carrier Johnson + Culture.

The borings were logged by observation of cuttings from the top of the boring, as well as samples collected within the borings at varying intervals to the total depth of the boring. Earth materials encountered within the borings were classified and logged in general accordance with the visual manual procedures of the Unified Soil Classification System. Boring logs of the test holes are presented in Appendix B. Our preliminary review indicates that the site is anticipated to be underlain by existing undocumented fills over alluvial deposits with a historical high groundwater table of between 10 to 20 feet below the existing surface. Other geotechnical items of review included liquefaction and seismically-induced dry sand settlement, remedial removal depths, static settlements, and expansive/corrosive soils.

Based on a review of the seismic hazard zone map for the Venice Quadrangle prepared by the California Geological Survey (CGS, 1999), the majority of the site is located within a State mapped liquefaction hazard area. Because of this zoning, LGC advanced a boring to a depth of approximately 51.5 feet below the ground surface to address the potential for liquefaction. All boring data were used to evaluate the liquefaction potential and to characterize the near-surface geotechnical characteristics of the site. The borings were sampled and logged from the surface under the supervision of a geologist from LGC.

During the subsurface investigation, representative bulk and relatively undisturbed samples were collected for laboratory testing. Laboratory testing was performed by EGLAB, Inc. (EGL), a City of Los Angeles approved testing lab. Laboratory testing was performed on representative soil samples and included moisture and density tests, maximum density and optimum moisture content, sieve analysis, direct shear, Atterberg Limits, expansion, consolidation, and corrosion testing. A summary of the test procedures and printouts of the laboratory test results are presented in Appendix C. The moisture and density test results were presented on the boring logs included in Appendix B.



LGC	Figure 1: Site Location Map West LA 6501 S. Sepulveda Road, City of Los Angeles, California	Project Name	FRH Realty/West LA
		Project No.	203022-01
		Eng. / Geol.	BIH
		Scale	not to scale
		Date	October 2020

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional and Local Geology

The site lies within the Los Angeles Basin, a structural trough located within southern California. The Los Angeles Basin (Basin) is a northwest-trending alluvial lowland plain about 50 miles long and 20 miles wide. Mountains and hills that generally expose late Mesozoic to late Pleistocene-age sedimentary and igneous rocks bound the Basin along the north, northeast, east and southeast. The basin is part of the Peninsular Ranges Geomorphic Province of California, which is characterized by regional compression due to the bend in the San Andreas Fault and sub-parallel blocks sliced longitudinally by young, steeply dipping northwest-trending fault zones. The Basin is a site of active sedimentation, and strata are interpreted to be as much as 31,000 feet thick in the center of the trough. The subject site is located approximately 3.5 miles from the Pacific Ocean and lies on the eastern edge of the Venice USGS 7.5' Quadrangle.

2.2 Site-Specific Geology

The site is composed of undocumented fills placed as a part of previous site improvements underlain by alluvium generally derived from the Santa Monica Mountains and Baldwin Hills. Centinela Creek lies just north of the site. A short distance to the north and west Centinela Creek merges with Ballona Creek which drains to the ocean. Alluvium and Older Alluvium were encountered across the site, mantled by undocumented artificial fill. As encountered during our site investigation, alluvial soils consist predominantly of medium dense to very dense, light to medium brown silty sands with minor gravel, with layers of stiff to very stiff silty clays. The undocumented fills and upper alluvial soils to a depth of 8 feet are generally less dense and are unsuitable to support future improvements, but below the upper 8 feet, the soils are generally medium dense to very dense/stiff to very stiff and slightly moist to moist to the maximum explored depth of approximately 51.5 feet. Groundwater was encountered in the borings at depths of 29 to 32 feet below the existing site grades.

2.2.1 Undocumented Artificial Fill (Afu)

The site is covered by a veneer of undocumented artificial fill soils, which were placed during previous development of the site. The fill soils encountered were approximately 7 to 8 feet thick and could be thicker elsewhere onsite (i.e. such as underlying the existing structures). These undocumented fills are composed primarily of silty to clayey sands. In general, the fill soils encountered on the site were found to be loose to medium dense and dry to slightly moist. The undocumented fills are not suitable for support of the proposed site improvements.

2.2.2 Quaternary Alluvium (Qal)

Alluvial soil was encountered below the undocumented fill at the site. As encountered, these soils generally consisted of variable brown, slightly moist to moist, medium dense to very dense silty to clayey sand to sand with local silty clay layers that are medium stiff to very stiff to an approximate depth of 20 to 25 feet across the site. Alluvial soils below the upper 8 feet (below existing grade) were found to be suitable and competent for structural support.

2.2.3 Quaternary Older Alluvium (Qoal)

Older Alluvial soils were encountered below the alluvium at depths below 20 to 25 feet across the site. As encountered, these soils generally consisted of variable light brown to medium gray, slightly moist to wet/saturated, medium dense to very dense silty/clayey sands to sands to the maximum explored depth of approximately 51.5 feet. The encountered older alluvial soils were found to be suitable and competent for structural support.

2.3 Geologic Structure

The site is composed of alluvium underlain by bedrock at significant depth. The alluvium is interpreted as generally massive with some poorly-defined, gradational changes between soil types.

2.4 Landslides

Based on the relatively flat nature of the site and our review of the geologic literature pertinent to the site, there are no indications of landslides close to or within the limits of the site.

2.5 Groundwater

Groundwater was encountered during our site investigations at depths of 29 and 32 feet from south to north; however based on the proposed site design (one level subterranean excavation – bottom approximately 10 feet below existing grades), groundwater is not anticipated to be encountered during site subterranean excavation and construction. Based on a review of the Seismic Hazard Zone Report, the historically highest groundwater is approximately 10 to 20 feet below the existing surface at the site (CGS, 1999). The proposed subterranean excavation is not anticipated to encounter ground water during site construction. In general, groundwater levels in alluvium fluctuate with seasonal variations and local zones of perched groundwater may occur within the near-surface deposits when precipitation is high. For design analysis, a historic high groundwater elevation of 10 feet was considered in the analysis.

2.6 Surface Water

Based on our review of local maps, the site is generally flat with sheet flow generally to the west toward Arizona Avenue. Surface water runoff relative to project design is the purview of the project civil engineer and should be directed away from the planned structure.

2.7 Seismicity, Faulting and Related Effects

2.7.1 Seismicity

The main seismic parameters to be considered when discussing the potential for earthquake-induced damage onsite are the distances to the causative faults, earthquake magnitudes, and expected ground accelerations. We have performed site-specific analysis based on these seismic parameters for the site and the onsite geologic conditions. The results of our analysis are discussed in terms of the potential seismic events that could be produced by the maximum probable earthquakes. A maximum probable earthquake is the maximum earthquake likely to occur given the known tectonic framework. The active Newport-Inglewood Fault (Los Angeles Basin Segment) is located approximately 2.1 miles (3.4 km).

2.7.2 Seismic Design Criteria

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2019 California Building Code (CBC) and ASCE 7-16. Representative site coordinates for the subject site of latitude 33.980668° N and longitude -118.39535° W were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) and adjusted design spectral response acceleration parameters (S_{DS} and S_{D1}) for Site Class D are provided in the following Table 1.

Table 1
Seismic Design Parameters

Selected Parameters from 2019 CBC, Section 1613 - Earthquake Loads	Seismic Design Values
Site Class per Chapter 20 of ASCE 7	D
Risk-Targeted Spectral Acceleration for Short Periods (S_S)	1.884g
Risk-Targeted Spectral Accelerations for 1-Second Periods (S_1)	0.663g
Site Coefficient F_a per Table 1613.3.3(1)	1.0
Site Coefficient F_v per Table 1613.3.3(2)	N/A
Site Modified Spectral Acceleration for Short Periods (S_{MS}) for Site Class D [Note: $S_{MS} = F_a S_S$]	1.884g
Site Modified Spectral Acceleration for 1-Second Periods (S_{M1}) for Site Class D [Note: $S_{M1} = F_v S_1$]	N/A
Design Spectral Acceleration for Short Periods (S_{DS}) for Site Class D [Note: $S_{DS} = (2/3)S_{MS}$]	1.256g
Design Spectral Acceleration for 1-Second Periods (S_{D1}) for Site Class D [Note: $S_{D1} = (2/3)S_{M1}$]	N/A
Seismic Design Category (per Section 1613.2.5)	D

Section 1803.5.12 of the 2019 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean (MCE_G) Peak Ground Acceleration (PGA) should be used for geotechnical evaluations. The PGA_M for the site is equal to 0.887 g (USGS, 2013).

A deaggregation of the PGA based on a 2,475-year average return period indicates that an earthquake magnitude of 6.36 at a distance of approximately 4.1 km (2.55 mi) from the site would contribute the most to this ground motion (USGS, 2008).

2.7.3 Faulting

Based on our review of geologic maps, the subject site is not located within an Alquist-Priolo Special Studies Zone (Hart, 1994) and no active faults are mapped projecting through the subject site. The possibility of damage due to ground rupture from earthquake fault rupture is considered low since active faults are not known to cross the site.

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the southern California region, which may affect the site, include soil liquefaction and dynamic settlement. Other secondary seismic effects include shallow ground rupture, and seiches and tsunamis. In general, these secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependent on the distance between the site and causative fault and the onsite geology. The major active fault that could produce these secondary effects is the Newport-Inglewood Fault (Los Angeles Basin Segment) as it is located approximately 2.1 miles (3.4 km) from the site. Other active but more distant faults that may result in shaking to the site include the Santa Monica Fault, Palos Verdes Fault, Hollywood Fault, Malibu Coast Fault, Elysian Park Thrust Fault, Raymond Fault, and the Verdugo Fault, among others.

A discussion of liquefaction and these secondary effects is provided in the following sections.

2.7.4 Shallow Ground Rupture

Shallow ground rupture due to active faulting is not likely to occur on site due to the distance from likely seismic events. Therefore, this phenomenon is not considered a significant hazard, although it is a possibility at any site.

2.7.5 Liquefaction and Dry sand Settlement

Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similarly to a fluid when subject to high-intensity ground shaking. Liquefaction occurs when three general conditions exist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. Liquefaction is typified by a buildup of pore-water pressure in the affected soil layer to a point where a total loss of shear strength occurs, causing the soil to behave as a liquid. Studies indicate that saturated, loose to medium dense, near surface cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils and cohesive soils exhibit low to negligible liquefaction potential.

Based on a review of seismic hazard zone map for the Venice Quadrangle prepared by the California Geological Survey (CGS, 1999), a portion of the site is located within a State of California Seismic Hazard Zone mapped liquefaction hazard area. Effects of liquefaction on level ground include potential seismic settlement, sand boils, ground oscillation, and bearing capacity failures below structures.

Historic high groundwater elevation is approximately 10 to 20 feet below the ground surface near the location of the subject site (CGS, 1999). Groundwater was encountered in the geotechnical boring advanced on site at depths ranging from 29 to 32 feet below the existing site grades. A conservative groundwater depth of 10 feet was utilized in the liquefaction analysis.

Our evaluation utilized the information collected from the excavations and laboratory test results, along with utilizing the more recent studies as indicated in SP 117A by Bray and Sancio, 2006 as a screening tool to determine if the encountered fine grained soils (clays) are susceptible to liquefaction and analyzed as such. Our evaluation included performing grain size distribution, Atterberg limit, and moisture content testing on representative fine-grained layers (i.e. clayey/silty Sands) encountered within the geotechnical borings excavated on-site. The laboratory test results indicated that the encountered fine grained clay layers between 15 and 25 feet were considered as being not susceptible to liquefaction.

The liquefaction analysis was performed using the LiquefyPro program with a user provided factor of safety of 1.3. The liquefaction analysis was performed considering the existing condition below with potentially liquefiable soils located from a depth of 10 feet from the ground surface with the highest historic groundwater elevation at a depth of 10 feet below the ground surface.

The liquefaction analysis was performed using the following input data:

- Groundwater at a depth of 10 feet below the ground surface during seismic event, and boring groundwater at elevations of 29 or 32 feet where encountered in the boring excavations.
- A Peak Horizontal Ground Acceleration (PGA_M) of 0.887g for a Design Earthquake Magnitude of 6.36.
- Fines content as determined from laboratory testing during this investigation.
- The hammer used for determining blow-counts for both the ring and SPT sampling was an auto-trip hammer with a 140 lb weight and a 30 inch drop. Therefore, based on previous discussions with city reviewers and based on the type of hammer used, an energy correction factor (CE) of 1.3 is considered acceptable for use in the analysis.

Based on our site evaluation, liquefaction analysis, and our professional opinion, the potential for specific layers to liquefy within the upper 51.5 feet of site soils is low. The graphical output of our liquefaction analysis which also shows the graphical output of seismically induced saturated and dry sand settlement is included in Appendix B of this response. The factor of safety value used is shown on the bottom left of the graphical output plot and on Number 9 of the input data on the summary output sheets.

Based on the results of the liquefaction/seismically induced settlement analysis, we estimated the amount of total liquefaction-induced and dry sand settlement possible for the design conditions is up to approximately 0.25-inches, and a differential settlement of approximately 0.15-inches. We estimated these settlements based on the procedures proposed by Tokimatsu and Seed (1987). The printout of the liquefaction analysis of boring B-1 through and B-3 is included in Appendix D.

2.7.6 Tsunamis and Seiches

Based on the elevation of the proposed development at the site with respect to sea level and its distance from large open bodies of water, the potential of seiche and/or tsunami is considered to be nil.

2.8 Slope Stability

No significant permanent slopes currently exist onsite or are planned for the subject site, therefore slope stability is not considered an issue with respect to site development.

2.9 Laboratory Testing

Laboratory testing of the onsite soils was performed on representative samples obtained from the borings and included moisture and density tests, maximum density and optimum moisture content, sieve analysis, direct shear, Atterberg Limits, expansion, consolidation, and corrosion testing. Laboratory testing was performed by EGLAB, Inc. (EGL). LGC has reviewed the laboratory test data, procedures and results performed by EGL with respect to the subject site and concurs with and accepts responsibility as geotechnical engineer of record for their work (laboratory testing). A discussion of the tests performed and printout of the laboratory test results are presented in Appendix C. The moisture and density test results are presented on the boring logs in Appendix B.

These results should be confirmed at the completion of site grading.

Expansion potential testing indicated expansion index of 3, “Very Low” (2019 California Building Code, CBC). Sulfate testing indicated soluble sulfate content was 0.018 percent (“So/Negligible” ACI 318R Table 4.3.1).

A corrosion suite (pH, resistivity, and chloride content) was performed on a representative sample of the onsite soils. The result for resistivity test was indicated a minimum resistivity value of 4,700 ohm-centimeters, pH value of 7.84, and chloride content of 175 parts-per-million (ppm). Caltrans defines a corrosive area where any of the following conditions exist: the soil contains more than 500 ppm of chlorides, more than 2,000 ppm (0.2 percent) of sulfates, or a pH of 5.5 or less. Test results are provided in Appendix C.

These results/assumptions should be confirmed at the completion of site grading.

3.0 CONCLUSIONS

Based on the results of our geotechnical evaluation and review, it is our opinion that the proposed site development is feasible from a geotechnical standpoint, provided the following recommendations included in this report are incorporated into the project plans and specifications, and followed during site grading and construction.

Our geotechnical conclusions are as follows:

- The site is not located within an Alquist-Priolo Earthquake Fault Zone (Hart, 1997).
- The site is located within an area deemed to have a potential for liquefaction (CGS, 1999); however based on our liquefaction evaluation the potential for liquefaction on site is considered low and is not a concern for the subject site; however seismically induced settlements of approximately 0.15 of an inch in 30 feet should be included in foundations design.
- Total static and seismically induced settlements of up to 1.5 inches with differential settlements of up to $\frac{3}{4}$ of an inch in 30 feet should be considered in the foundation design.
- Groundwater was encountered in our geotechnical borings at depths ranging from 29 to 32 feet below the existing grade and is not anticipated to be a concern for the project.
- Based on the subsurface exploration and our review, the site is underlain by undocumented artificial fill over alluvium. The undocumented fill and alluvial soils are considered potentially compressible/collapsible in the upper 8 feet.
- Active or potentially active faults are not known to exist on the site.
- No known oil fields or oil wells (active or abandoned) were identified within the subject site during our review.
- Laboratory test results of the onsite soils indicate a very low expansion potential; however based on the soil types onsite a low expansion potential should be considered in the design.
- Laboratory test results of the onsite soils indicate negligible soluble sulfates and are considered mildly corrosive to metals.
- Laboratory test results of the onsite soils indicate a negligible potential of hydro-collapse underlying the recommended remedial removals.
- The onsite soils below recommended remedial grading/excavation depths have a low potential for static settlement (i.e., slightly compressible).
- From a geotechnical perspective, the existing onsite soils are suitable for use as fill, provided they are relatively free from rocks (larger than 6 inches in maximum dimension), construction debris, and organic material.

4.0 RECOMMENDATIONS

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of site preparation followed by excavation for subterranean level followed by construction of slab-on-grade type foundations for the proposed subterranean structure, installation of utilities, subsequently followed by paving/pouring of driveways.

We recommend that earthwork onsite be performed in accordance with the recommendations herein, the City of Los Angeles, and the General Earthwork and Grading Specifications for Rough Grading included in Appendix E. In case of conflict, the recommendations in the following sections shall supersede those included as part of Appendix E.

4.1.1 Site Preparation

Prior to grading of areas to receive structural fill or engineered structures, all ground surfaces should be cleared of obstructions, any existing debris and stripped of vegetation. Heavy vegetation and debris should be removed and properly disposed of offsite. All debris from any demolition activities at the site should also be removed and disposed off-site. Holes or depressions resulting from the removal of buried obstructions should be replaced with compacted fill.

Following remedial removals, areas to receive fill should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition, and recompacted to at least 90 percent relative compaction (based on American Standard of Testing and Materials [ASTM] Test Method D1557).

4.1.2 Removal and Recomaction

As discussed in Section 2.2, the proposed site is underlain by unsuitable soils, which may settle under the addition of water, surcharge of fill and/or foundation loads. Compressible materials not removed by the planned grading/subterranean excavations should be excavated to competent material (approximately 8-feet below existing grades) and replaced with compacted fill soils. We anticipate that the design cuts/excavations (approximately 10 feet) for the subterranean level will remove all unsuitable soils; however, localized, deeper removals should be anticipated where deemed necessary by the geotechnical consultant based on observations during grading/subterranean excavation. Once the excavation is completed to the design bottom, the bottom should be evaluated by the geotechnical consultant, and if deemed suitable, the removal bottom should be scarified and recompacted to a minimum 90 percent relative compaction.

Compressible materials, within areas planned to support pavement or other appurtenant structures outside of the subterranean excavation area, should be excavated to competent material and replaced with compacted fill soils. We anticipate these removals on the site to be on the order of approximately 4 feet below existing grade; however, localized, deeper removals should be anticipated where deemed necessary by the geotechnical consultant based on

observations during grading. Removal bottoms should be scarified to a minimum depth of 12 inches, brought to at least optimum-moisture content, and recompact.

Based on our site investigation groundwater was encountered at depths of 29 to 32 feet below existing grades; therefore, based on the site design with subterranean excavations to a depth of approximately 10 feet, groundwater is not anticipated to be encountered during site excavation. However, groundwater levels in alluvium fluctuate with seasonal variations and local zones of perched groundwater may occur within the near-surface deposits when precipitation is high., and based on the historic high groundwater level of approximately 10 feet and the conceptual design consisting of one level of subterranean parking, groundwater may be encountered near the bottom of the subterranean excavations, although not anticipated.

At the time of construction, if the design foundation level is below the ground water table, the anticipated subgrade soils (i.e. dense sand soils) are likely to be wet to nearly saturated. Construction of a minimum 2-inch thick “mud” (lean concrete) slab may be necessary with a required waterproofing membrane placed above the mud slab prior to foundation construction. At no time should any traffic be allowed by the contractor that causes deflection of the mud slab. The mud slab should be installed to allow for foot and light traffic to allow for construction.

From a geotechnical perspective, material that is removed may be placed as fill provided the material is relatively free from rocks (greater than 6 inches in maximum dimension), organic material and construction debris, is moisture-conditioned or dried (as needed) to obtain above-optimum moisture content, and then recompact prior to additional fill placement or construction.

4.1.3 Shrinkage/Bulking

Based on the site soils, bulking is not anticipated at the site. The preliminary estimated shrinkage factors of 5 to 10 percent for the undocumented fill and alluvium may be used for consideration of earthwork calculations. These are preliminary rough estimates which will vary with depth of removal, stripping losses, field conditions at the time of grading, etc. In addition, handling losses are not included in the estimates.

4.1.4 Temporary Excavation Stability

Due to the recommended depth of remedial removals below existing grades (approximately 8 feet), the temporary stability of the excavations along the perimeter of the site needs to be considered. All excavations for the proposed development should be performed in accordance with current OSHA (Occupational Safety and Health Agency) regulations and those of other regulatory agencies, as appropriate.

Temporary excavations maybe cut vertically up to five feet. Excavations over five feet should be slot-cut, shored, or cut no steeper than 1H: 1V (horizontal, H: vertical, V) slope gradient. Surface water should be diverted away from the exposed cut, and not be allowed to pond on top of the excavations. Temporary cuts should not be left open for an extended period of time. Planned temporary conditions should be reviewed by the geotechnical consultant of record in

order to reduce the potential for sidewall failure. The geotechnical consultant may provide recommendations for controlling the length of sidewall exposed.

Where sufficient space is not available for sloped cuts directly adjacent to existing structures or improvements the cut shall be performed by the A-B-C slot method as outlined below.

1. The banks of the excavation shall be made at 1H:1V or a combination of vertical cut and a 1H :1V.
2. Vertical cuts, not exceeding 8 feet in width are made in the locations of the first slot "A".
3. Back-fill and compact the first slot.
4. The second adjacent slot, "B" is excavated.
5. Back-fill and compact the second slot.
6. Then the third slot "C" is excavated.
7. Back-fill and compact the third slot.
8. Repeat the above steps until all the required excavations are performed adjacent to the existing improvements.

4.1.5 Temporary Shoring

The following preliminary geotechnical parameters may be utilized by the shoring consultant for design of the temporary shoring system. Temporary shoring is generally considered to have a service life of two years or less. The geotechnical conditions outside of the perimeter of the proposed structure have not been investigated as part of this report. The recommendations provided herein with regard to shoring of the proposed excavation are based on assumed conditions, extrapolated from the data gathered from our site investigations. The shoring designer should independently evaluate the parameters provided, and conduct an additional investigation if they consider necessary.

Prior to construction, the contractor should verify underground clearance of any existing utility lines or structures that must be removed or protected in place during construction, or may conflict with any proposed shoring system. Any tieback anchors and/or soil nails that extend beyond the site property limits will require permission from the adjacent property owner. Special attention will be required to protect existing settlement sensitive improvement in close proximity to the proposed excavation, such as any adjacent structures or streets located along the boundary of the site.

Typical cantilever temporary shoring, where deflection of the shoring will not impact the performance of adjacent structures or streets, may be designed using the active equivalent fluid pressures of 40 pounds per square foot (psf) per foot of depth (or pcf). Braced (i.e. internal bracing -rakers) or tied-back shoring is recommended in areas where the shoring will be located close to existing structures or streets in order to limit shoring defections or required due to the proposed depth of excavation. Braced or tied-back shoring with a level backfill may be designed using an active trapezoidal soil pressure of $24H$ in pounds per square foot (psf), where H is equal to the depth in feet of the excavation being shored (shape of the trapezoid should be 0.2H, 0.6H, 0.2H). Any building, equipment, or traffic loads located within a 1:1 (horizontal to vertical) projection from the base of the shoring should be added to

the applicable lateral earth pressure. A minimum additional uniform lateral pressure of 100 psf for the upper 10 feet should be added to the appropriate lateral earth pressures to account for typical vehicle traffic loading. The proposed shoring should be designed for a maximum shoring deflection of up to 1-inch adjacent to the street (non-surcharged condition) and up to a maximum of 0.5-inches adjacent to existing buildings (surcharged condition).

In addition, the above noted lateral earth pressures for temporary shoring does not include hydrostatic pressures since the current groundwater level was encountered below the anticipated depth of the subterranean structure. Consideration should be given to increasing the provided lateral earth pressures and/or design factors of safety in order to further limit shoring deflections and subsequent potential impacts on adjacent structures and improvements, as necessary.

If temporary gravity grouted tie-backs are used anchors may be designed using a preliminary bond stress of 400 pounds per square foot (psf), and if pressure/post-grouted tieback anchors are used, anchors may be designed using a preliminary bond stress of up to 2,500 pounds per square foot (psf). However, the tieback designer should make an independent evaluation in order to verify the preliminary bond stress is adequate for site conditions. Tieback bond stress should be verified by field testing. Tieback anchors should minimally be designed, constructed, and tested in accordance with the requirements of the Post-Tensioning Institute (PTI). For design purposes, tieback should obtain their load-carrying capacity from the soil behind a plane taken to be 3 horizontal feet from the bottom of the shoring facing and inclined at an angle of 60 degrees measured from the horizontal extending to the top of the excavation. Passive resistance of soldier piles may be assumed to be an equivalent fluid pressure of 350 pcf to a maximum value of 3,500 psf. The passive earth pressure may be increased by 100 percent for isolated piles. Piles with spacing greater than 3 times of pile diameter can be considered as isolated piles. In order to develop the full lateral resistance, firm contact between the soldier pile and undisturbed soils must be assured. For vertical shoring capacity, an allowable skin friction of 500 psf may be used for the portion of pier below the proposed development excavation. End bearing should be neglected. Drilling of shafts for soldier piles may require casing or drilling mud to prevent caving.

Due to the nature of the site soils, it is expected that continuous lagging between soldier piles will be required. The time between lagging excavation and lagging placement should be as short as possible. Soldier piles should be designed for the full-anticipated pressures. Due to arching in the soils, the pressure on the lagging will be less. However, it is recommended that the lagging be designed for the full design active fluid pressure but be limited to a maximum of 400 psf. Therefore, the design lagging pressure should consider a parabolic earth pressure distribution with an active equivalent fluid pressures of 40 pounds per square foot (psf) per foot of depth (or pcf) up to a maximum of 400 psf. The maximum span for lagging for this project should be 10 feet.

The components of the shoring system should be designed by a California licensed structural and/or civil engineer specializing in the design of shoring systems. Field pullout testing should be performed during construction to verify the estimated pullout resistance used in the design and/or post grout tubes should be used to ensure adequate design capacities are obtained.

Ultimately, it is the specialty contractor's responsibility to obtain the required pullout capacity, which may require design and/or field modifications.

LGC should review the shoring plans prior to construction to verify that geotechnical recommendations are properly implemented into the project plans

It is highly recommended that a program of documentation and monitoring be devised and put into practice before the onset of any groundwork. The contractor should establish survey points on the shoring, adjacent streets, and neighboring buildings within 100 feet of the excavation perimeter prior to any excavation. These survey points should be used to monitor the movement of the shoring and existing improvements during construction excavation.

The monitoring program should include, but not necessarily be limited to detailed documentation of the existing improvements, buildings and utilities around the excavation, with particular attention to any distress that is already present prior to the start of work.

A licensed surveyor should be retained to establish monuments on the shoring and the surrounding ground prior to excavation. Such monuments should be monitored for horizontal and vertical movement during construction. Results of the monitoring program should be provided immediately to the project structural (shoring) engineer and LGC for review and evaluation.

I-Beam soldier piles may also be vibrated into place as a means of installation. When using the vibration method of installing the soldier beams, the minimum embedment depth shall be 5 feet (when designed with tiebacks and rakers) and 10 feet (for cantilever design) below the lowest excavated plane. Predrilling may be necessary by the shoring contractor to facilitate installation of soldier piles. The available passive resistance of the pile may be determined using the diagonal length from the outer edges of opposite flange sections. Passive resistance of soldier piles may be assumed to be an equivalent fluid pressure of 300 pcf to a maximum value of 3,000 psf. The passive earth pressure may be increased by 100 percent for isolated piles. Piles with spacing greater than 3 times of pile diameter can be considered as isolated piles.

It is recommended that the diameter of the predrilled holes should not exceed two-thirds of the depth of the web of the I-beam. The depth of the predrilled holes should not exceed the planned excavation depth. In addition, when predrilling, the auger shall be backspun out of the pilot holes, leaving the soils in place. Installation with vibration should be limited to approximately ½ inch per second peak particle velocity. All shoring (predrilling, installation of shoring piles, and lagging) shall be performed under the continuous inspections by a deputy grading inspector of this firm.

4.1.6 Fill Placement and Compaction

From a geotechnical perspective, the onsite soils are suitable for use as compacted fill, provided they are screened of rocks greater than 6 inches in maximum dimension, organic material, and construction debris. Areas prepared to receive structural fill and/or other surface improvements should be scarified to a minimum depth of 6 inches, brought to at least optimum-moisture content, and recompact to at least 90 percent relative compaction (based on ASTM Test Method D1557). The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts generally not exceeding 8 inches in loose thickness. Placement and compaction of fill should be performed in accordance with local grading ordinances under the observation and testing of the geotechnical consultant.

4.1.7 Trench Backfill and Compaction

The onsite soils may generally be suitable as trench backfill provided they are screened of rocks and other material over 6 inches in diameter and organic matter. Trench backfill should be compacted in uniform lifts (generally not exceeding 8 inches in compacted thickness) by mechanical means to at least 90 percent relative compaction (per ASTM Test Method D1557).

If trenches are shallow and the use of conventional equipment may result in damage to the utilities; clean sand, having sand equivalent (SE) of 30 or greater, should be used to bed and shade the utilities. Sand backfill should be densified. The densification may be accomplished by jetting or flooding and then tamping to ensure adequate compaction. A representative from LGC should observe, probe, and test the backfill to verify compliance with the project specifications.

4.2 Foundations

4.2.1 General

Preliminary recommendations for foundation design and foundation construction are presented herein. When the structural loads for the proposed structures are known they should be provided to our office to verify the recommendations presented herein.

The following foundation recommendations are provided. The two foundations recommended for the proposed structures are: (1) Conventional foundation; or (2) Mat foundations. For preliminary design purposes a medium expansion potential should be considered for design. The as-graded soil conditions should be verified.

The information and recommendations presented in this section are not meant to supersede design by the project structural engineer or civil engineer specializing in the structural design nor impede those recommendations by a corrosion consultant. Should conflict arise, modifications to the foundation design provided herein can be provided.

4.2.2 Conventional Foundations

Continuous/Individual footings should have minimum widths of 24 inches for the proposed structure. Based on the proposed one level subterranean structure (i.e. foundations at approximate depths of 10 to 12 feet below existing grades), the following bearing capacity is considered suitable for the proposed development. Based on our review and evaluation of the proposed subterranean foundations founded into competent native soils, the proposed foundations may be designed for a maximum allowable bearing capacity of 4,000 lb/ft². This bearing capacity was found to be achievable and is considered acceptable from a geotechnical point of view. A factor of safety greater than 3 was used in evaluating the above bearing capacity values. Bearing values indicated above are for total dead loads and frequently applied live loads. The above vertical bearing may be increased by one-third for short durations of loading which will include the effect of wind or seismic forces. Lateral forces on footings may be resisted by passive earth resistance and friction at the bottom of the footing. Foundations may be designed for a coefficient of friction of 0.35, and a passive earth pressure of 250 lb/ft²/ft. The passive earth pressure incorporates a factor of safety of about 1.5.

All footing excavations should be cut square and level, and should be free of sloughed materials and trash. Subgrade soils should be pre-moistened for the assumed low expansion potential (to be confirmed at the end of grading).

The subgrade should be moisture-conditioned and proof-rolled just prior to construction to provide a firm, relatively unyielding surface, especially if the surface has been loosened by the passage of construction traffic.

Subgrade soils should be pre-saturated to 1.2 times optimum moisture content to a depth of 12 inches for a low expansion potential. The minimum thickness of the floor slabs should be at least 5 inches, and joints should be provided per usual practice.

4.2.3 Bearing Capacity for Shallow Appurtenant Structures

Shallow foundations may be designed for an allowable bearing capacity of 2,000 lb/ft² (gross), for continuous footings a minimum of 12 inches wide and 18 inches deep and spread footings 24 inches wide and 18 inches deep, into certified compacted fill. A factor of safety greater than 3 was used in evaluating the above bearing capacity value. This value may be increased by 300 psf for each additional foot in depth and 200 psf for each additional foot of width to a maximum value of 3,500 psf. These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Once building loads are available the bearing capacity and settlements should be confirmed/reevaluated.

Lateral forces on footings may be resisted by passive earth resistance and friction at the bottom of the footing. Foundations may be designed for a coefficient of friction of 0.35, and a passive earth pressure of 250 lb/ft²/ft. The passive earth pressure incorporates a factor of safety of greater than 1.5.

Bearing values indicated above are for total dead loads and frequently applied live loads. The above vertical bearing may be increased by one-third for short durations of loading which will include the effect of wind or seismic forces.

4.2.4 Mat Foundation

Mat foundation can be used for support of the proposed building structures. An allowable soil bearing pressure of 2,500 psf may be used for the design of the mat slab. The allowable bearing value is for total dead loads and frequently applied live loads and may be increased by one-third for short durations of loading which will include the effect of wind or seismic forces. A coefficient of vertical subgrade reaction, k , of 150 pounds per cubic inch (pci) may be used to evaluate the pressure distribution beneath the mat foundation. The magnitude of total and differential settlements of the mat foundation will be a function of the structural design and stiffness of the mat.

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. A coefficient of friction of 0.35 may be used. Frictional resistance along the bottom of the mat foundation should be reduced if a waterproofing membrane is installed. Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. Frictional resistance along the bottom of the mat foundation should be reduced due to the presence of a waterproofing membrane. A coefficient of friction of 0.15 may be used for Paraseal membranes. If a membrane other than Paraseal is desired, LGC should review the material specification in order to provide a coefficient of friction.

The underslab moisture retarder (i.e. an equivalent capillary break method) should consist of a 15-mil thick polyolefin (or equivalent) in conformance with ASTM E 1745 Class A material underlain by a minimum 1-inch of sand, as needed. The sand layer requirements above the vapor barrier are the purview of the foundation engineer/structural engineer, and should be provided in accordance with ACI Publication 302 "Guide for Concrete Floor and Slab Construction". These recommendations must be confirmed (and/or altered) by the foundation engineer, based upon the performance expectations of the foundation. Ultimately, the design of the moisture retarder system and recommendations for concrete placement and curing are the purview of the foundation engineer, in consideration of the project requirements provided by the architect and developer.

4.2.5 Foundation Settlement

Based on our current understanding of the project, the results of our site investigation and the recommended remedial grading of 8 feet, with shallow foundations embedded into compacted fills, we estimate the total design static and seismic settlement of the site to be up to 1.5-inches with a differential settlement of approximately of $\frac{3}{4}$ of an inch in 30 feet. Site foundation should be designed considering a differential settlement from static and seismically induced settlements of up to $\frac{3}{4}$ of an inch in 30 feet.

4.3 Lateral Earth Pressures for Subterranean Walls

The following section provides lateral earth pressures for proposed subterranean retaining walls. It is anticipated that site subterranean walls will be constructed directly against temporary shoring. If backfill is required, it should meet the project specifications outlined in Section 4.1.6.

Lateral earth pressures are provided as equivalent fluid unit weights, in psf/ft of depth or pcf. These values do not contain an appreciable factor of safety. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of soil.

If the wall can sufficiently yield to mobilize the full shear strength of the soil, it can be designed for “active” pressure. If the wall cannot yield under the applied load, the shear strength of the soil cannot be mobilized and the earth pressure will be higher. Such walls (basement walls) should be designed for “at-rest” conditions. If a structure moves toward the soils, the resulting resistance developed by the soil is the “passive” resistance. The following lateral pressures for drained and un-drained native soils are presented on Tables 2 and 3. The soil parameters below, assume there is no support provided by the temporary shoring system.

TABLE 2
Lateral Earth Pressures

Conditions	Equivalent Fluid Unit Weight (pcf)	
	Level Backfill (Static)	Seismic Earth Pressure (pcf) *
Active	40	13
At-Rest	60 (Triangular) or 37.5H (Trapezoidal)	20.5

*This dynamic pressure should be added to the pressures given in Table 2 and considered as an inverted triangular distribution with the resultant acting at 0.6H in relation to the base of the retaining wall footing (where H is the retained height). The aforementioned incremental seismic load was determined in general accordance with the standard of practice in the industry (using the Mononobe-Okabe method for active and Woods method for at-rest) for determining earth pressures as a result of seismic events.

The equivalent fluid pressure values stated above do not include hydrostatic pressures. For designing subterranean walls with a hydrostatic pressure (un-drained) the following lateral earth pressures that include a buoyant and hydrostatic lateral pressure may be used for the portion of the wall in an un-drained condition.

Given the location of groundwater encountered during the field investigations (i.e. 29 to 32 feet below existing grade), and the previously documented historic high groundwater depth of 10-feet below the existing grade, and the proposed depth of the bottom parking level, the structure will be located near/at the historic high groundwater level. Based on the latest design including one subterranean level and the depth of the historical high groundwater level of 10 feet, the subterranean wall does not need to be

designed including the hydrostatic pressure; however, if the subterranean wall does extend below 10 feet, hydrostatic pressures starting from a depth of 10 feet below the existing grade should be considered in the evaluation the basement wall design.

TABLE 3
Lateral Earth Pressures (un-drained)

Conditions	Equivalent Fluid Unit Weight (pcf)
	Level Backfill
Active	90
At-Rest	100

Surcharge loading effects from any adjacent structures should be evaluated by the structural engineer. Any building or traffic loads located within a 1:1 (horizontal to vertical) projection from the base of the retaining structure should be added to the applicable lateral earth pressure. A minimum additional uniform lateral pressure of 100 psf for the upper 10 feet should be added to the recommended lateral earth pressures to account for typical vehicle traffic loading located within the zone of influence of the proposed retaining structure.

A passive lateral earth pressure of 350 psf per foot to a maximum passive pressure of 3,500 psf may be used. The passive pressure may be increased by one-third due to wind or seismic forces.

4.4 Waterproofing

We recommend a waterproofing consultant be retained to determine the most appropriate system, if necessary. The design, installation and observation of the waterproofing system are not the purview of the geotechnical consultant. Adequate waterproofing of subterranean walls should be provided to reduce the potential for ground water seepage below the groundwater table as well as nuisance water issues that may develop above the groundwater table.

4.5 Preliminary Pavement Recommendations **Asphaltic Concrete**

Based on an assumed R-value of 20, we recommend the following preliminary minimum street sections for Traffic Indices of 5, 6, and 7 (Table 4). These recommendations should be confirmed with R-value testing of representative near-surface soils at the completion of grading. Final street sections should be confirmed by the project civil engineer based upon the projected Traffic Index. In addition, additional sections can be provided based on other traffic indices.

Table 4
Preliminary Pavement Design Sections

Assumed Traffic Index	5	6	7
R-Value Subgrade	20	20	20
AC Thickness	3.0 inches	3.5 inches	4 inches
Base Thickness	8.0 inches	9.5 inches	12 inches

The aggregate base material should conform to the specifications for Crushed Aggregate Base or Crushed Miscellaneous Base (Standard Specifications for Public Works Construction –SSPWC Section 200-2). The subgrade should achieve a minimum relative compaction of 90 percent. The base material should be compacted to achieve a minimum relative compaction of 95 percent. Base and subgrade materials should be moisture-conditioned to a relatively uniform moisture content at or slightly over optimum.

Portland Cement Concrete Pavement

Portland Cement Concrete Pavement (PCCP) may be designed using a minimum of 6-inches of Portland cement concrete over 6-inches of compacted aggregate base. The modulus of rupture of the concrete should be a minimum of 500 pounds per square inch (psi) at 28 days. Contraction joints should be placed at maximum 15-foot spacing. Where the outer edge of a concrete pavement connects to an asphalt pavement, the concrete slab should be thickened by 50 percent at a taper not to exceed a slope of 1 in 10. This section is only applicable for passenger car driveway areas and should be thickened if heavy truck loading is anticipated. In addition, additional sections can be provided based on other desired anticipated traffic loadings.

The aggregate base material should conform to the specifications for Crushed Aggregate Base or Crushed Miscellaneous Base (Standard Specifications for Public Works Construction –SSPWC Section 200-2). The subgrade should achieve a minimum relative compaction of 90 percent. The base material should be compacted to achieve a minimum relative compaction of 95 percent. Base and subgrade materials should be moisture-conditioned to a relatively uniform moisture content at or slightly over optimum.

Vehicular Concrete Pavers

Vehicular pavers are typically 3-1/8 inches in thickness and are underlain by 1-inch of sand.

Based on ASCE 58-10 for interlocking pavers, considering a Traffic Index (TI) of 6 and an R-value of 20 for the subgrade soils, we recommend the following base section underlying the proposed pavers. The proposed pavers and sand should be underlain by a minimum of 12-inches of aggregate base. As an alternative interlocking pavers and sand bedding can also be placed directly on the design asphaltic concrete base course over aggregate base, considering a TI of 6, or can be placed on a minimum of 5-inches of concrete over 6 inches of aggregate base. The design pavement sections provided herein are considered suitable to support the imposed loads from a fire apparatus.

The aggregate base material should conform to the specifications for Crushed Aggregate Base or Crushed Miscellaneous Base (Standard Specifications for Public Works Construction –SSPWC Section 200-2). The subgrade should achieve a minimum relative compaction of 90 percent per ASTM- D1557. The base material should be compacted to achieve a minimum relative compaction of 95 percent. Base and subgrade materials should be moisture-conditioned to a relatively uniform moisture content at or slightly over optimum.

Fire lane Turf Block

Turf block should be per manufactures specifications. For preliminary recommendations turf block should have a minimum thickness of 4 inches and a minimum 28-day compressive strength of 3,000 psi. The turf block pavement should be underlain by a minimum of 12-inches of aggregate base. The aggregate base material should conform to the specifications for Crushed Aggregate Base or Crushed Miscellaneous Base (Standard Specifications for Public Works Construction – SSPWC Section 200-2). The subgrade should achieve a minimum relative compaction of 90 percent per ASTM- D1557. The base material should be compacted to achieve a minimum relative compaction of 95 percent. Base and subgrade materials should be moisture-conditioned to a relatively uniform moisture content at or slightly over optimum.

4.6 Corrosivity to Concrete and Metal

The National Association of Corrosion Engineers (NACE) defines corrosion as “a deterioration of a substance or its properties because of a reaction with its environment.” From a geotechnical viewpoint, the “environment” is the prevailing foundation soils and the “substances” are the reinforced concrete foundations or various buried metallic elements such as rebar, piles, pipes, etc., which are in direct contact with or within close vicinity of the foundation soil.

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. ACI 318R-08 Table 4.3.1, provides specific guidelines for the concrete mix design when the soluble sulfate content of the soils exceeds 0.1 percent by weight or 1,000 ppm. The minimum amount of chloride ions in the soil environment that are corrosive to steel, either in the form of reinforcement protected by concrete cover, or plain steel substructures such as steel pipes or piles, is 500 ppm per California Test 532.

Based on site soil testing, the onsite soils are classified as having a negligible sulfate exposure condition in accordance with ACI 318R Table 4.3.1. As a preliminary recommendation due to results of sulfate content testing, concrete in contact with onsite soils should be designed in accordance with ACI 318R Table 4.3.1 for the So/negligible category. It is also our opinion that onsite soils should be considered mildly corrosive to buried metals. The client and/or other members of the design team should consider this potential as they determine necessary. LGC is not a corrosion consultant and does not provide recommendations related to corrosion.

4.7 Nonstructural Concrete Flatwork

Concrete Flatwork

Concrete flatwork (such as walkways, patios, entryways, etc.) have a high potential for cracking due to changes in soil volume related to soil-moisture fluctuations because these slabs are typically much thinner than foundation slabs and are not reinforced with the same dynamic as foundation elements. To reduce the potential for excessive cracking and lifting, concrete should be designed in accordance with the minimum guidelines outlined below. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints, but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

TABLE 5
Nonstructural Concrete Flatwork

Minimum Thickness (in.)	4
Presaturation	Presoak to 12 inches
Reinforcement	No. 3 at 24 inches on centers or 6x6 No. 6 x No. 6 WWM
Crack Control Joints	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness
Maximum Joint Spacing	5 feet

Pedestrian Concrete Pavers

Concrete pavers should be installed per manufactures recommendations. The following are considered minimum recommendations for the concrete pavers and are not meant to supersede more restrictive manufactures recommendations. Concrete pavers should be designed to be underlain by a minimum of 1 inch of leveling sand over a minimum of approximately 4-inches of compacted aggregate base.

The subgrade should achieve a minimum relative compaction of 90 percent. The base material should be compacted to achieve a minimum relative compaction of 95 percent. Base and subgrade materials should be moisture-conditioned to a relatively uniform moisture content at or slightly over optimum.

4.8 Control of Surface Water and Drainage Control

Positive drainage of surface water away from structures is very important. No water should be allowed to pond adjacent to buildings. Positive drainage may be accomplished by providing drainage away from buildings at a gradient of at least 2 percent for a distance of at least 5 feet, and further maintained by a swale or drainage path at a gradient of at least 1 percent. Where necessary, drainage paths may be shortened by use of area drains and collector pipes.

Planters with open bottoms adjacent to buildings should be avoided. Planters should not be designed adjacent to buildings unless provisions for drainage, such as catch basins, liners, and/or area drains, are made. Overwatering must be avoided.

4.9 Construction Observation and Testing

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. The interpolated subsurface conditions should be checked in the field during construction by a representative of LGC.

Construction observation and testing should also be performed by the geotechnical consultant during future grading, excavations, backfill of utility trenches, foundation or retaining wall construction or when an unusual soil condition is encountered at the site. Grading plans, foundation plans, and final project drawings should be reviewed by this office prior to construction.

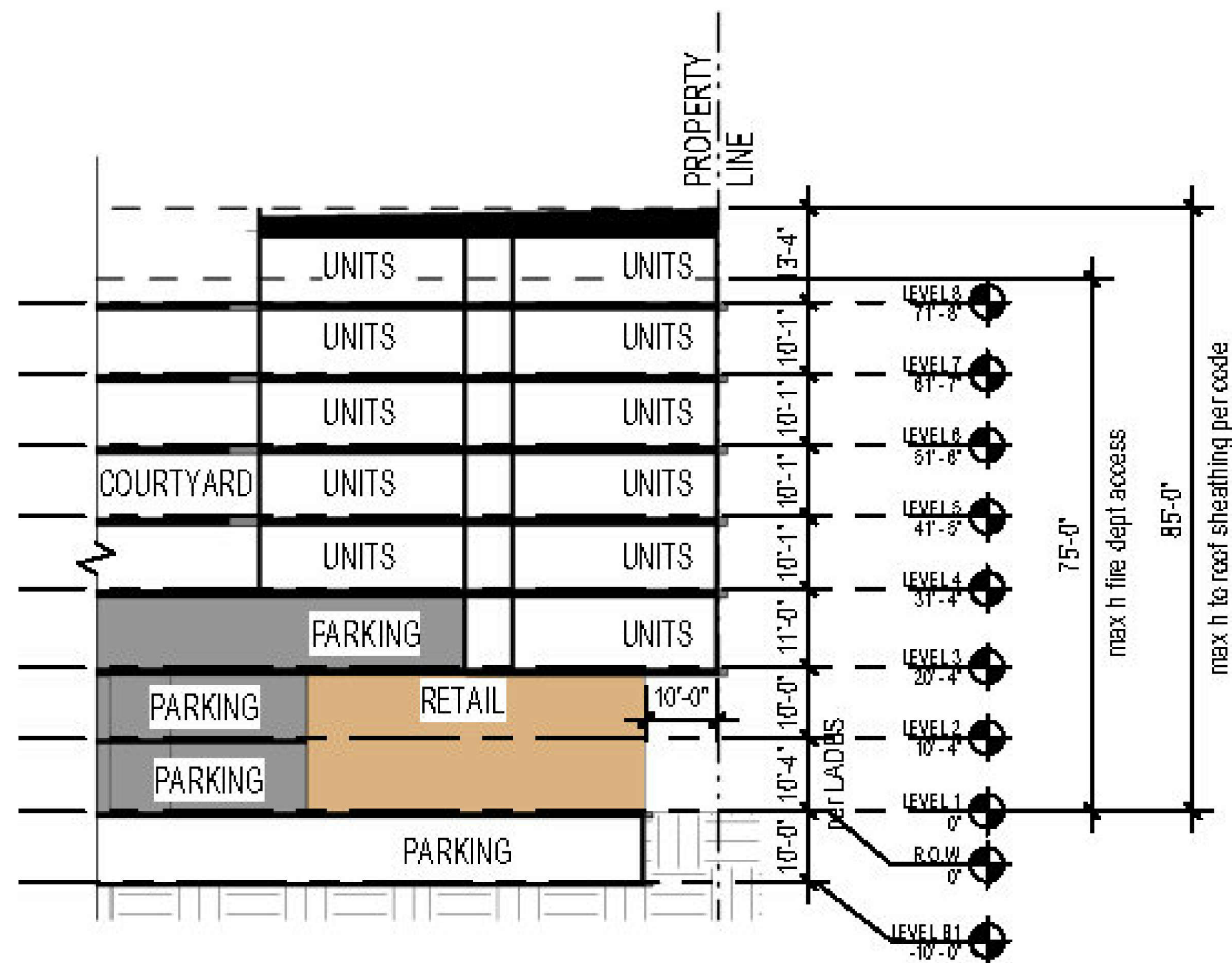
5.0 LIMITATIONS

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report. The samples taken and submitted for laboratory testing, the observations made and the in-situ field testing performed are believed representative of the entire project; however, soil and geologic conditions revealed by excavation may be different than our preliminary findings. If this occurs, the changed conditions must be evaluated by the project soils engineer and geologist and design(s) adjusted as required or alternate design(s) recommended.

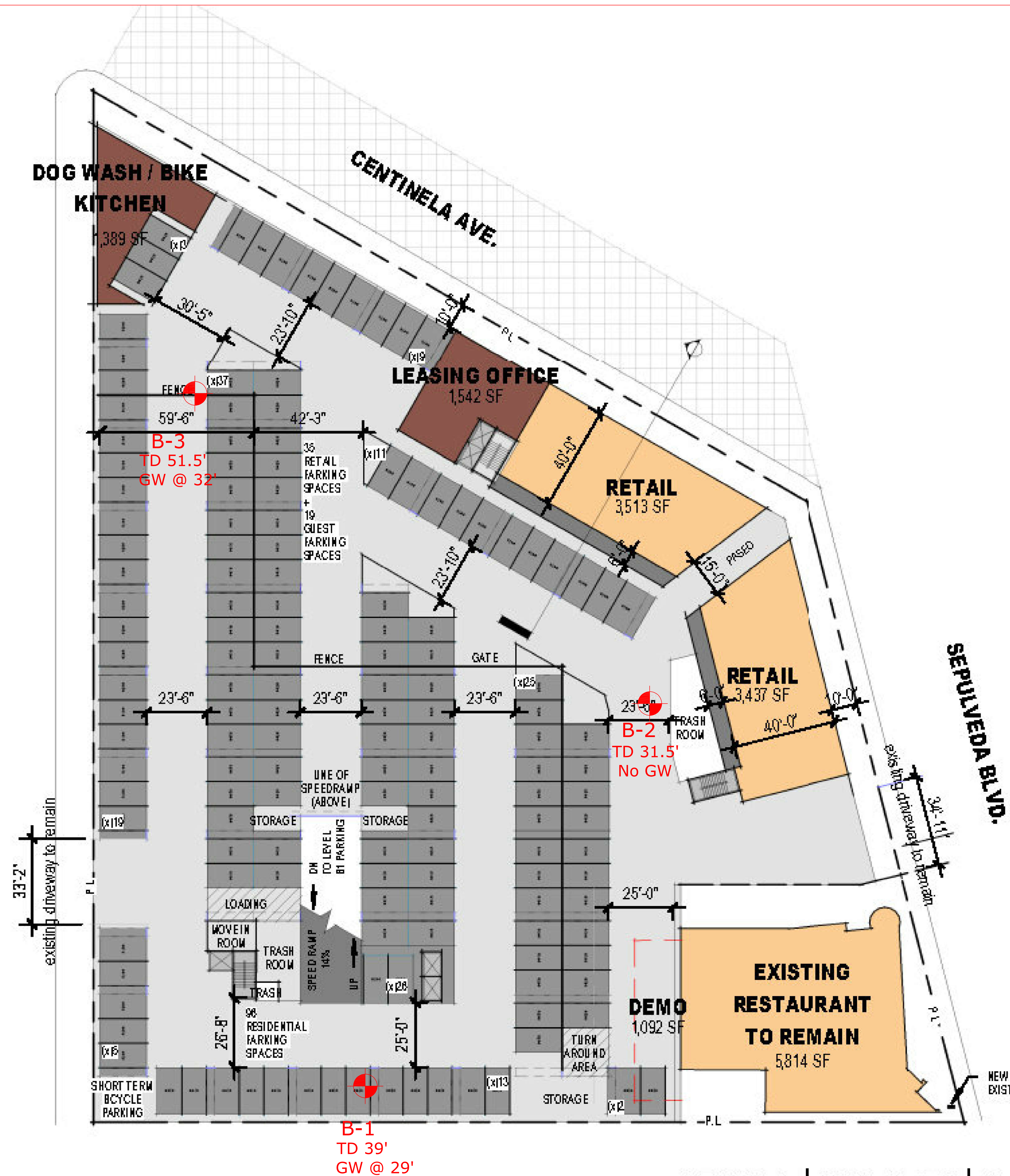
This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and/or project engineer and incorporated into the plans, and the necessary steps are taken to see that the contractor and/or subcontractor properly implements the recommendations in the field. The contractor and/or subcontractor should notify the owner if they consider any of the recommendations presented herein to be unsafe.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties.

In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control.



ARIZONA AVE.



FAIRFIELD
RESIDENTIAL

SEPULVEDA &
CENTINELA

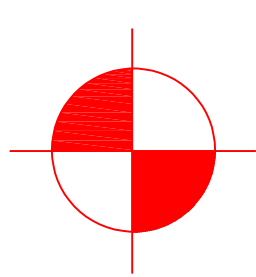
LEVEL 1 FLOOR PLAN
6501 S. SEPULVEDA BLVD.

carrierjohnson + CULTURE
architecture + environments + brand strategy + graphics

03

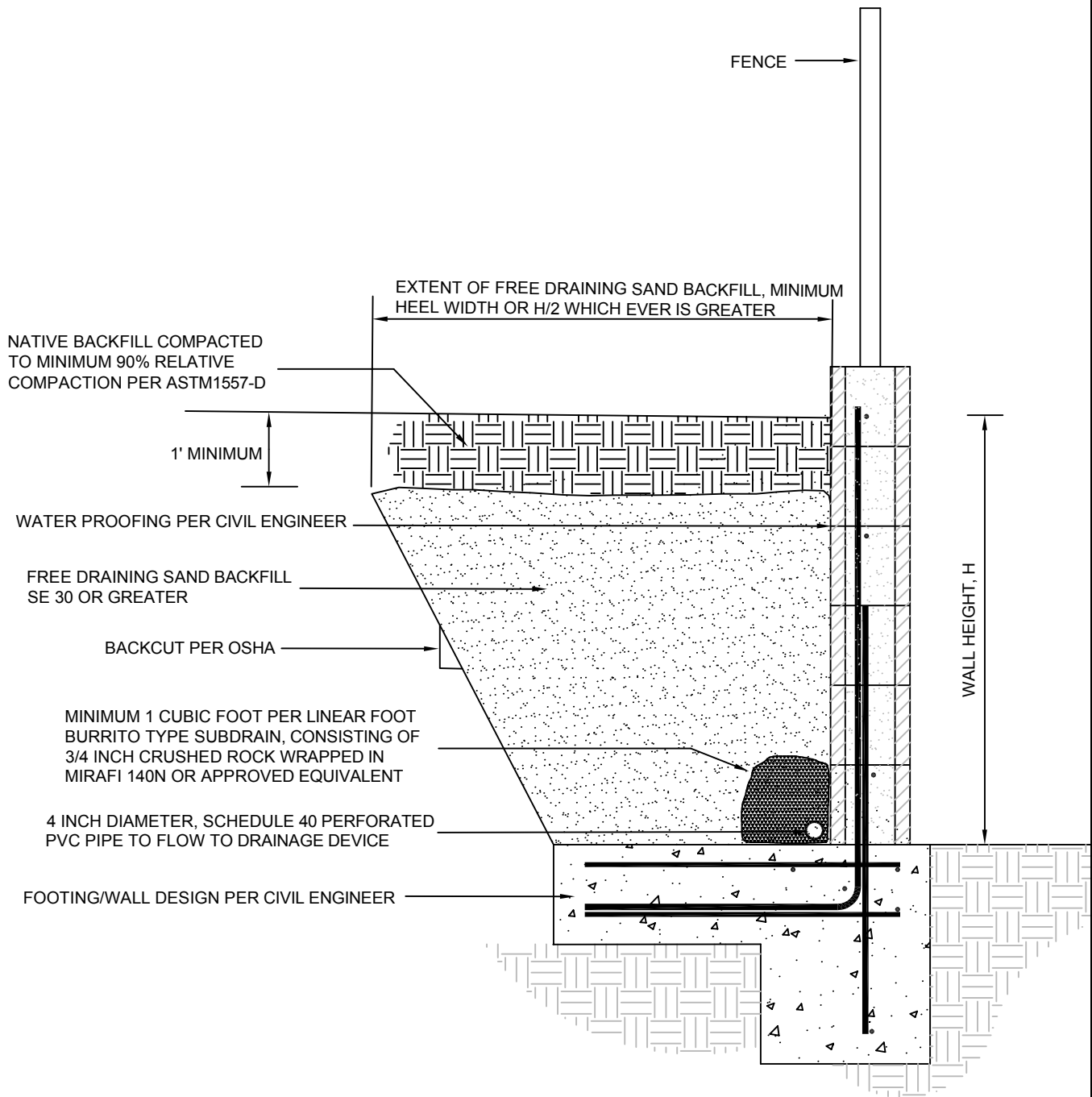
LEGEND

B-3
TD 51.5'
GW @ 32'



Approximate Location of Boring by LGC
with Total Depth Drilled Sept.3, 2020

Exploration Location Plan 6501 S. Sepulveda Road City of Los Angeles, California		Figure 2	
LGC	LGC Valley, Inc.		PROJECT NAME
	28532 Constellation Road Valencia, CA 91355		PROJECT NO.
	TEL: (661) 702-8474 FAX: (661) 702-8475		ENG. / GEOL.
			SCALE
		DATE	October 2020



**Figure 3:
Retaining Wall
Detail, Sand
Backfill**

Project Name	FRH Realty/West LA
Project No.	203022-01
Eng. / Geol.	BIH/MCH
Scale	N/A
Date	October 2020

LGC

APPENDIX A

References

- American Concrete Institute, 1985, Manual of Concrete Practice, Parts 1 and 2.
- Barrows, A.G., Irvine, P.J. and Tan, S.S., 1995, Geologic surface effects triggered by the Northridge earthquake *in* Woods, M.C. and Seiple, W.R. eds., The Northridge, California, Earthquake of 17 January 1994: CDMG SP-116, p. 39-46.
- California Building Code, 2019 Version
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APPENDIX A

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APPENDIX B

Geotechnical Boring Logs

Geotechnical Boring Log B-1

Date:	9/3/2020	Page:1
Project Name:	FF/ West LA	Project Number: 203022-01
Drilling Company:	Choice Drilling	Type of Rig: Hollow Stem
Drive Weight: 140lbs	Drop: 30"	Hole Dia: 8"
Elevation of Top of Hole:	Hole Location: See Map	

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
								Logged By: LF Sampled By: LF	
	0		A				SM	0-3" Asphalt	
			1	8 12 13		5.1		<u>Undocumented Fill (Afu):</u> 3"- Light to medium brown, silty fine SAND, dry to slightly moist, medium dense	DS
	5		2	8 9 10	103.4	14.0	SM	5'- Same as above, slightly moist to moist.	CON
			3	16 17 20	131.8	8.7	SM-SC	<u>Quaternary Alluvium (Qa):</u> 7.5'- Medium brown, silty very clayey fine to coarse SAND, slightly moist, medium dense	CON
	10		4	14 20 24	121.9	13.2	SM-SC	10'- Medium brown, silty clayey fine SAND, slightly moist, medium dense	
			5	10 14 17		6.0	SM-SW	12.5'- Light to medium brown, silty fine to coarse SAND, dry to slightly moist, dense; minor subangular gravel	SH/AL
	15		6	7 8 9		21.0	CH	15'- Light brown fine sandy silty CLAY, moist, very stiff; oxidized zones, high plasticity	
			7	5 6 11		27.6	CH	17.5'- Same as above	SH/AL
	20		8	4 5 6		31.1	CL	20'- Same as above, stiff, medium plasticity	SH/AL
			9	13 17 20			SM	<u>Quaternary Old Alluvium (Qoa):</u> 22.5'- Light brown minor silt fine to coarse SAND, dry to slightly moist, dense	SA
	25		10	13 15 18		7.0	SM	25'- Same as above	
			11	15 28 37		16.7	SM	27.5'- Same as above, becomes wet	
	30							29'- Groundwater encountered	

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**LGC VALLEY, INC.
GEOTECHNICAL CONSULTING**

Geotechnical Boring Log B-1

Date:	9/3/2020	Page:2
Project Name:	FF/ West LA	Project Number: 203022-01
Drilling Company:	Choice Drilling	Type of Rig: Hollow Stem
Drive Weight: 140lbs	Drop: 30"	Hole Dia: 8"
Elevation of Top of Hole:	Hole Location: See Map	

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
								Logged By: LF Sampled By: LF	
	30		12	25 26 27			SM	30'- Light to medium gray, minor silty fine to coarse SAND, saturated, very dense	SA
			13	11 24 50/6"			SM	32.5'- Same as above	
	35		14	17 24 26			SM	35'- Same as above	
			15	20 50/5"			SM	37.5'- Same as above	
	40							39'- Refusal	
	45								
	50								
	55								
	60								

LGC

**LGC VALLEY, INC.
GEOTECHNICAL CONSULTING**

Geotechnical Boring Log B-2

Date:	9/3/2020	Page:1
Project Name:	FF/ West LA	Project Number: 203022-01
Drilling Company:	Choice Drilling	Type of Rig: Hollow Stem
Drive Weight: 140lbs	Drop: 30"	Hole Dia: 8"
Elevation of Top of Hole:	Hole Location: See Map	

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
								Logged By: LF Sampled By: LF	
	0		A				SM	0-3"- Asphalt	EI
			1	8 15 22	113.0	10.8		<u>Undocumented Fill (Afu):</u> 0.3"- Medium brown, minor clay silty fine SAND, slightly moist, medium dense	COR
	5		B				SM	5'- Medium to dark brown, clayey silty fine to coarse SAND, moist, loose	CON
			2	5 6 7	113.1	12.5			
			3	9 12 23	126.0	9.6	CL	<u>Quaternary Alluvium (Qa):</u> 7.5'- Very dark brown, silty fine to coarse sandy CLAY, moist, very stiff	CON
	10		4	10 18 20	121.8	12.2	SM-SC	10'- Medium to dark brown, silty very clayey fine to coarse SAND, moist, medium dense	
	15		5	50/6"	127.3	9.0	CL	15'- Medium brown to light gray, silty CLAY with trace sand, dry to slightly moist, hard	
	20		6	9 11 14		33.5	CL	20'- Same as above, very stiff; oxidized orange, moist	SH/AL
	25		7	50/6"	97.7	26.2	SM	<u>Quaternary Old Alluvium (Qoa):</u> 25'- Light to medium gray, minor silt, fine to coarse SAND, moist to wet, dense	
	30								

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**LGC VALLEY, INC.
GEOTECHNICAL CONSULTING**

Geotechnical Boring Log B-2

Date:	9/3/2020	Page:2
Project Name:	FF/ West LA	Project Number: 203022-01
Drilling Company:	Choice Drilling	Type of Rig: Hollow Stem
Drive Weight: 140lbs	Drop: 30"	Hole Dia: 8"
Elevation of Top of Hole:	Hole Location: See Map	

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
								Logged By: LF Sampled By: LF	
30			8	13 25 28			SM	30'- Medium gray, minor silt, fine to coarse SAND, wet, very dense; minor subrounded gravel	SA
35								Total Depth 31.5'	
40								No Ground water was encountered	
45								Backfilled September 3,2020 using bentonite cap and native soil.	
50									
55									
60									

LGC

**LGC VALLEY, INC.
GEOTECHNICAL CONSULTING**

Geotechnical Boring Log B-3

Date:	9/3/2020	Page:1
Project Name:	FF/ West LA	Project Number: 203022-01
Drilling Company:	Choice Drilling	Type of Rig: Hollow Stem
Drive Weight: 140lbs	Drop: 30"	Hole Dia: 8"
Elevation of Top of Hole:	Hole Location: See Map	

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
								Logged By: LF Sampled By: LF	
	0						SM	0-3"- Asphalt	
			1	7 8 8				<u>Undocumented Artificial Fill (Afu):</u> 3"- Medium brown, silty fine to coarse SAND, slightly moist, loose	Max
	5		A					2.5' No recovery	DS
			2	3 4 5	111.7	14.6	SM	5'- Medium to dark brown, silty clayey fine SAND, slightly moist to moist, loose	CON
			3	7 18 21	129.3	12.0	CL	<u>Quaternary Alluvium (Qa):</u> 7.5'- Dark brown, silty fine to coarse sandy CLAY, slightly moist, very stiff	CON
	10		4	12 16 22	127.9	10.4	SM	10'- Medium to dark brown, silty very clayey fine to coarse SAND, slightly moist, medium dense; some subrounded gravel	
	15		5	6 8 9		17.7	CL	15'- Light to medium brown, silty fine to medium sandy CLAY, slightly moist to moist, very stiff; minor oxidation	SH/AL
	20		6	11 27 28	115.7	17.4	SM	<u>Quaternary Old Alluvium (Qoa):</u> 20'- Light to medium brown, clayey very silty fine SAND, moist, dense; minor oxidation	
	25		7	12 28 30		4.9	SM-SP	25'- Light orange brown, minor silt and clay, fine to coarse SAND, dry, dense	SA
	30								

LGC

**LGC VALLEY, INC.
GEOTECHNICAL CONSULTING**

Geotechnical Boring Log B-3

Date:	9/3/2020	Page:2
Project Name:	FF/ West LA	Project Number: 203022-01
Drilling Company:	Choice Drilling	Type of Rig: Hollow Stem
Drive Weight: 140lbs	Drop: 30"	Hole Dia: 8"
Elevation of Top of Hole:	Hole Location: See Map	

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
								Logged By: LF Sampled By: LF	
30			8	15 30 40	119.0	13.7	SM -SP	30'- Light to medium gray, minor silt/clay, fine to coarse SAND, moist, dense 32'- Groundwater Encountered	
35			9	9 15 17			SM-SP	35'- Same as above, becomes saturated	
40			10	10 15 18		18.7	SM -SP	40'- Same as above; minor subrounded gravel	
45			11	17 27 31			SM	45'- Medium gray, silty fine to coarse SAND, saturated, very dense; minor subrounded gravel	
50			12	12 16 20		23.3	SM	50'- Medium gray silty fine SAND, saturated, dense	
55								Total Depth 51.5' Groundwater encountered at 32' Backfilled September 3,2020 using bentonite cap and native soil.	
60									

LGC

**LGC VALLEY, INC.
GEOTECHNICAL CONSULTING**

Key to Laboratory Test Symbols

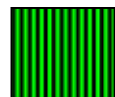
Symbol	Laboratory Test
SA	Sieve Analysis
H	Hydrometer Analysis
SHA	Sieve & Hydrometer Analysis
-200	Percent Passing #200 Sieve
AL	Atterberg Limits
MAX	Maximum Density
DS	Undisturbed Direct Shear
RDS	Remolded Direct Shear
TRI	Triaxial Shear
EI	Expansion Index
P	Permeability
CN	Consolidation
COL	Collapse
UC	Unconfined Compression
S	Sulfate Content
pHR	pH & Resistivity
COR	Corrosion Suite (pH, Resistivity, Chloride, Sulfate)
RV	R-Value



ASPHALT
CONCRETE



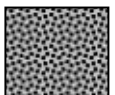
CLAY



SILT



SAND



GRAVEL/COBBLES



APPROXIMATE
GROUNDWATER LEVEL

APPENDIX C

Laboratory Testing Results by EGLAB, Inc.

Laboratory testing was performed by EGLAB, Inc. The laboratory testing program was directed towards providing quantitative data relating to the relevant engineering properties of the soils. Samples considered representative of site conditions were tested in general accordance with American Society for Testing and Materials (ASTM) procedure and/or California Test Methods (CTM), where applicable. The following summary is a brief outline of the test type and the results are presented on the following pages.

LGC has reviewed the laboratory test data, procedures and results performed by EGL, with respect to the subject site, and concurs with and accepts responsibility as geotechnical engineer of record for their work (laboratory testing).

Moisture and Density Determination Tests: Moisture content (ASTM D2216) and dry density determinations (ASTM D2937) were performed on relatively undisturbed samples obtained from the test borings and/or trenches. The results of these tests are presented in the boring logs. Where applicable, only moisture content was determined from undisturbed or disturbed samples.

Grain Size Distribution: Representative samples were dried, weighed, and soaked in water until individual soil particles were separated (per ASTM D421) and then washed on a No. 200 sieve. The portion retained on the No. 200 sieve was dried and then sieved on a U.S. Standard brass sieve set in accordance with ASTM D422 (CTM 202).

Atterberg Limits: The liquid and plastic limits (“Atterberg Limits”) were determined in accordance with ASTM Test Method D4318 for engineering classification of fine-grained material.

Soil Classification: Soils were classified according the Unified Soil Classification System (USCS) in accordance with ASTM Test Methods D2487 and D2488. This system uses relies on the Atterberg Limits and grain size distribution of a soil. The soil classifications (or group symbol) are shown on the laboratory test data, and boring logs.

Expansion Index: The expansion potential of selected samples were evaluated by the Expansion Index Test, U.B.C. Standard No. 18-2 and/or ASTM D4829. Specimens are molded under a given compactive energy to approximately the optimum moisture content and approximately 50 percent saturation or approximately 90 percent relative compaction. The prepared 1-inch-thick by 4-inch-diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with tap water until volumetric equilibrium is reached.

Maximum Density Tests: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM D1557.

Consolidation: Consolidation tests were performed on selected, relatively undisturbed ring samples (Modified ASTM Test Method D2435). Samples (2.42 inches in diameter and 1 inch in height) were placed in a consolidometer and increasing loads were applied. The samples were allowed to consolidate under “double drainage” and total deformation for each loading step was recorded. The percent consolidation for each load step was recorded as the ratio of the amount of vertical compression to the original sample height.

Corrosion Testing: Chloride content was tested in accordance with Caltrans Test Method (CTM) 422. The soluble sulfate contents of selected samples were determined by standard geotechnical methods (CTM 417). The soluble sulfate content is used to determine the appropriate cement type and maximum water-cement ratios. The test results are presented in the table below: Minimum resistivity and pH tests were performed in general accordance with CTM 643 and standard geochemical methods. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. As a results of soil’s resistivity decreasing, corrosivity increases.

Direct Shear: Direct shear tests were performed, in accordance with ASTM D3080, on selected remolded and/or undisturbed samples, which were soaked for a minimum of 24 hours under a surcharge equal to the applied normal force during testing. After transfer of the sample to the shear box, and reloading the sample, pore pressures set up in the sample due to the transfer were allowed to dissipate for a period of approximately 1 hour prior to application of shearing force. The samples were tested under various normal loads, a motor-driven, strain-controlled, direct-shear testing apparatus at a strain rate of less than 0.001 to 0.5 inch per minute (depending upon the soil type).

EGLAB, INC.,
11819 Goldring Road, Unit D, Arcadia, CA 91006
Ph: 626-263-3588; Fax: 626-263-3599; Email: ryan@eglab.com

September 30, 2020

LGC Valley, Inc.
28532 Constellation Rd.
Valencia, CA 91355

Attn: Mr. Basil Hattar

RE: LABORATORY TEST RESULTS/REPORT

Project Name: FF / West LA

Project No: 203022-01

EGL Job No. 20-059-010

Dear Mr. Hattar:

We have completed the testing program conducted on samples from the above project. The tests were performed in accordance with testing procedures as follows:

TEST	METHOD
Modified Proctor	ASTM D1557
Corrosion	CT-417, 422, 643
Expansion Index	ASTM D4829
Moisture & Dry Density	ASTM D2937
Consolidation	ASTM D2435
Direct Shear	ASTM D3080
Atterberg Limits	ASTM D4318
Grain Size Analysis	ASTM D422

Enclosed is the Summary of Test Results.

We appreciate the opportunity to provide testing services to LGC Valley, Inc. Should you have any questions, please call the undersigned.

Sincerely yours,
EGLAB, Inc.



Ryan Jones, GE
Principal Engineer



SUMMARY OF TEST RESULTS

PROJECT NAME: FF / West LA

EGLAB JOB No.: 20-059-010

PROJECT No.: 203022-01

CLIENT: LGC Valley, Inc.

DATE: 9/15/2020

SUMMARIZED BY: JT

Boring No.	Sample No.	Depth (ft)	pH CalTrans 643	Chloride Content CalTrans 422 (ppm)	Sulfate Content CalTrans 417 (% by weight)	Minimum Resistivity CalTrans 643 (ohm-cm)	Expansion Index ASTM D 4829
B-2	A	2.0-3.0	7.84	175	0.018	4,700	3

SUMMARY OF LABORATORY TEST RESULTS

PROJECT NAME: FF / West LA

EGLAB JOB NO.: 20-059-010

PROJECT NO.: 203022-01

CLIENT: LGC Valley, Inc.

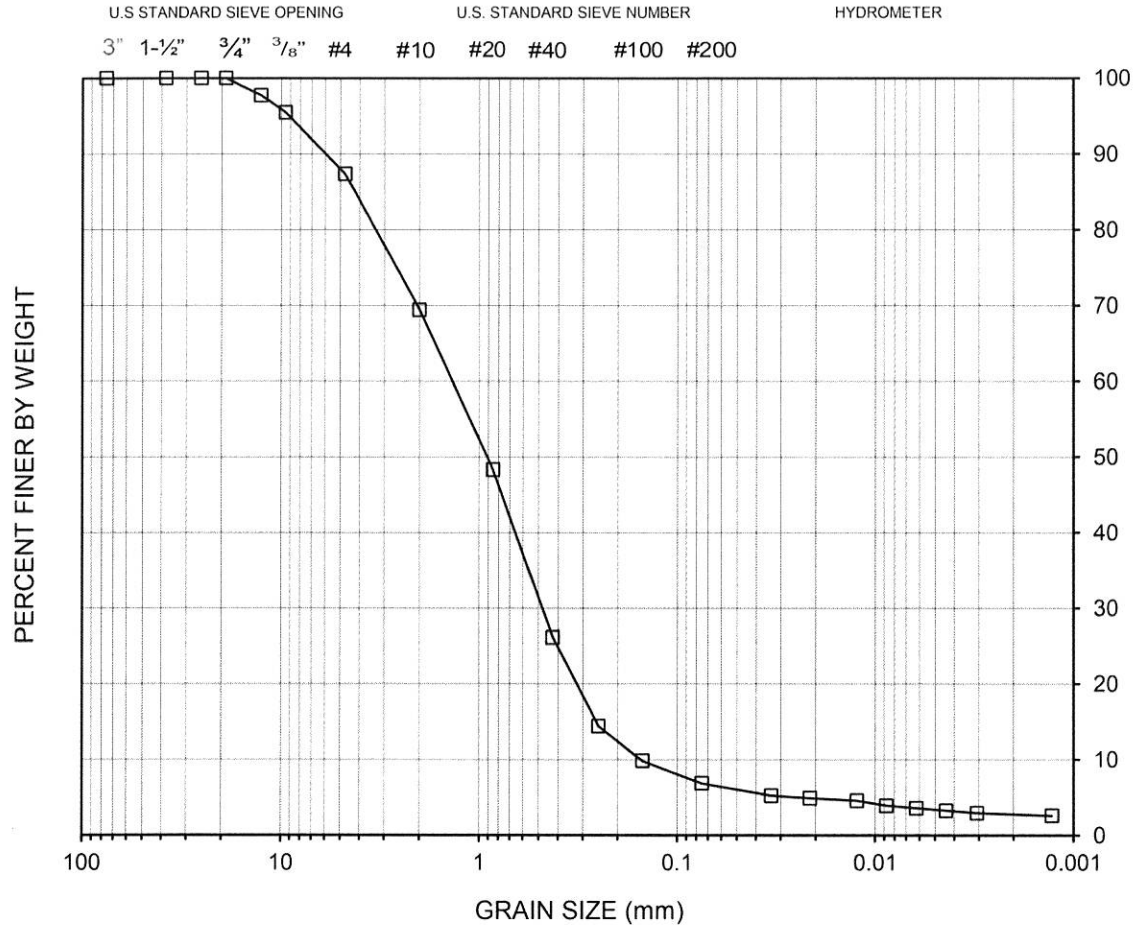
DATE: 9/18/2020

SUMMARIZED BY: JT

BORING NO.	SAMPLE NO.	DEPTH (in.)	MOISTURE CONTENT ASTM D2216 (%)	DRY DENSITY ASTM D2937 (PCF)	ATTERBERG LIMITS ASTM D4318 *(LL,PL,PI)
B-1	4	10.0	13.2	121.9	
B-1	5	12.5	6.0		Non Plastic
B-1	6	15.0	21.0		
B-1	7	17.5	27.6		50,23,27
B-1	8	20.0	31.1		42,24,18
B-1	10	25.0	7.0		
B-1	11	27.5	16.7		
B-1	14	35.0	14.7		
B-2	1	2.5	10.8	113.0	
B-2	4	10.0	12.2	121.8	
B-2	5	15.0	9.0	127.3	
B-2	6	20.0	33.5		49,24,25
B-2	7	25.0	26.2	97.7	
B-3	4	10.0	10.4	127.9	
B-3	5	15.0	17.7		32,16,16
B-3	6	20.0	17.4	115.7	
B-3	7	25.0	4.9		
B-3	8	30.0	13.7	119.0	
B-3	10	40.0	18.7		
B-3	12	50.0	23.3		

*LL,PL,PI = LIQUID LIMIT, PLASTIC LIMIT, PLASTICITY INDEX

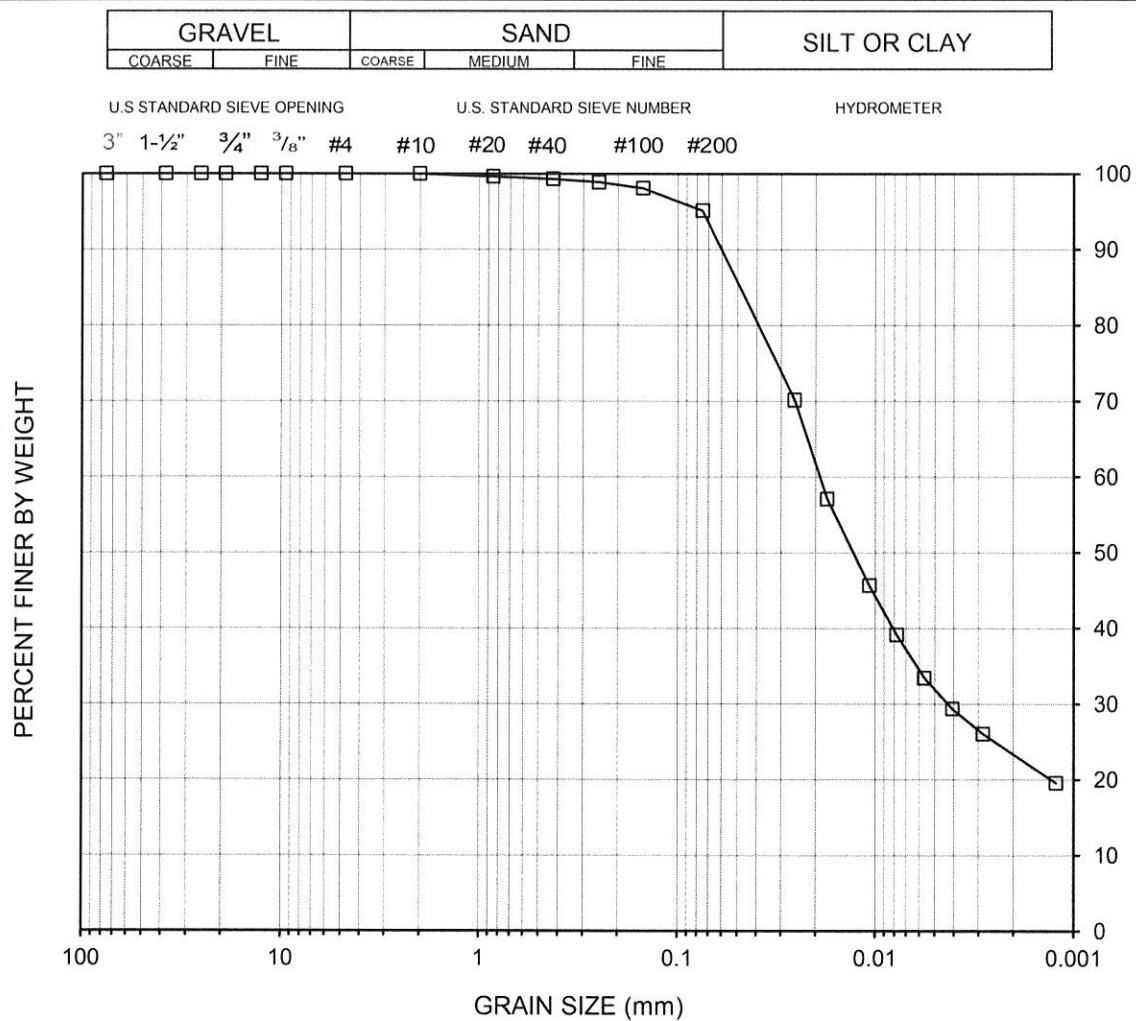
GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SYMBOL	BORING No.	SAMPLE ID.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-1	5	12.5	Bag	SW-SM	N/A	N/A

Clay	3.4%
Sand	93.2%
Silt	3.4%

EGLAB, INC.	Project Name: FF / West LA
	Client: LGC Valley, Inc. Job No.: 203022-01 EGLAB Project No.: 20-059-010
GRAINSIZE DISTRIBUTION CURVE	
9/18/20	(ASTM D422) FIGURE

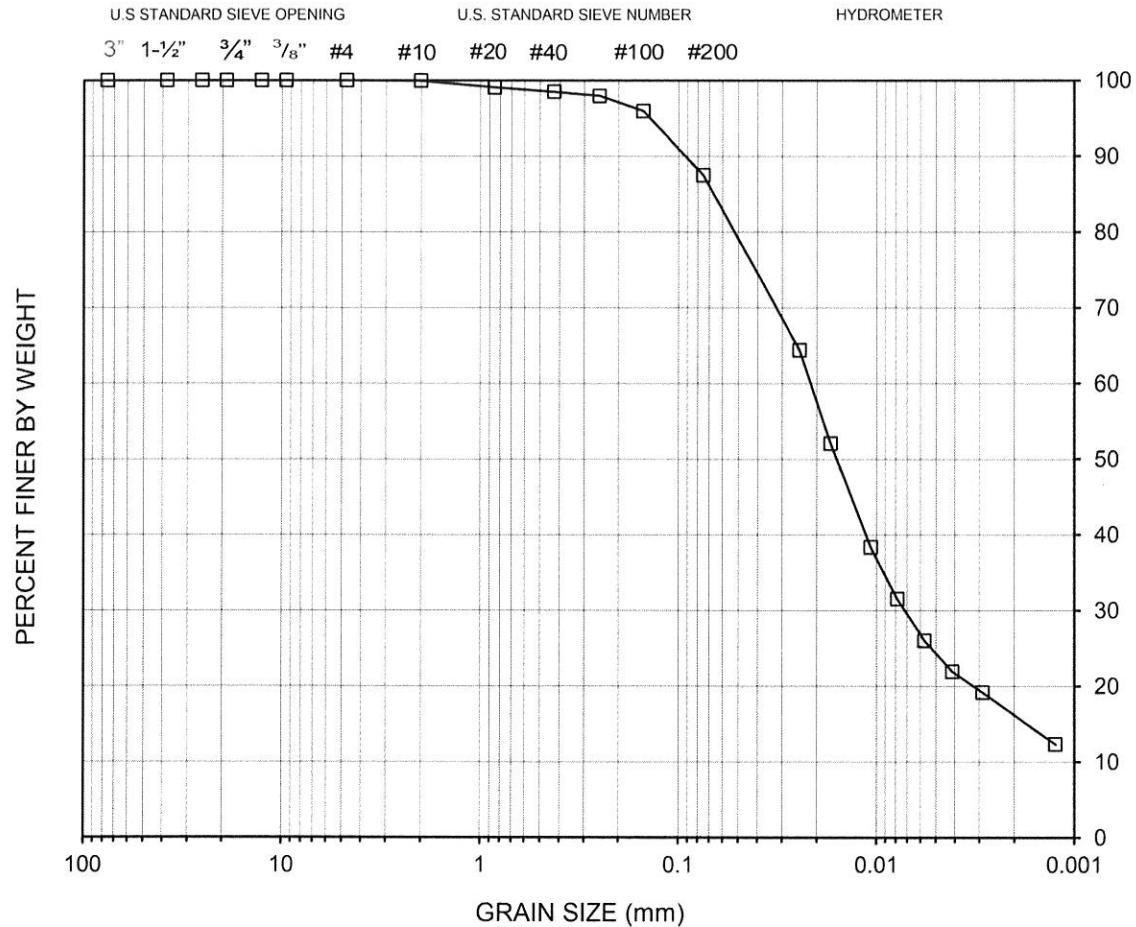


SYMBOL	BORING No.	SAMPLE ID.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-1	7	17.5	Bag	CH	50	27

Clay	31.7%
Sand	4.9%
Silt	63.4%

EGLAB, INC.	Project Name: FF / West LA
	Client: LGC Valley, Inc. Job No.: 203022-01 EGLAB Project No.: 20-059-010
GRAINSIZE DISTRIBUTION CURVE	
9/18/20	(ASTM D422)
FIGURE	

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SYMBOL	BORING No.	SAMPLE ID.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-1	8	20.0	Bag	CL	42	18

Clay	24.2%
Sand	12.5%
Silt	63.3%

EGLAB, INC.	Project Name: FF / West LA
	Client: LGC Valley, Inc.
	Job No.: 203022-01
	EGLAB Project No.: 20-059-010
GRAINSIZE DISTRIBUTION CURVE	
9/18/20	(ASTM D422)
FIGURE	

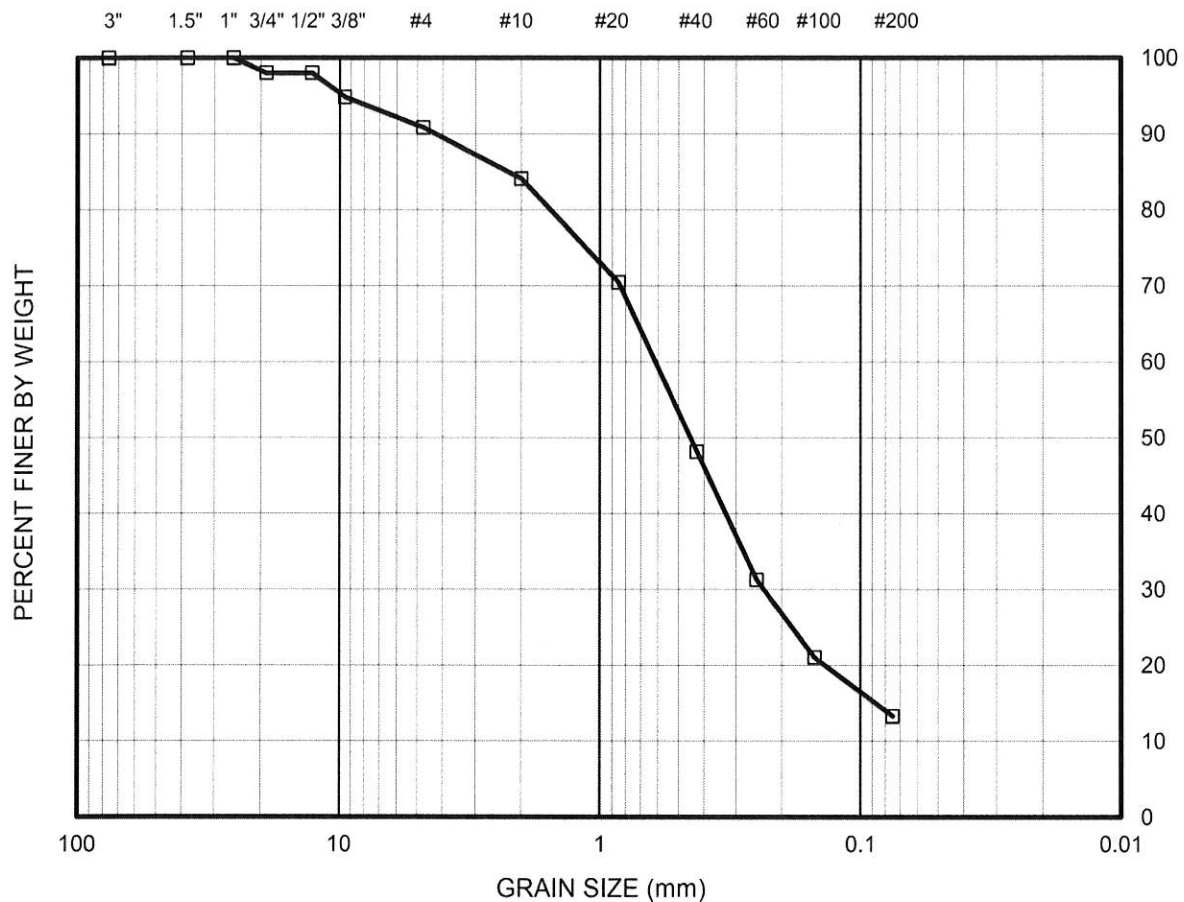


GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

U S STANDARD SIEVE OPENING

U.S. STANDARD SIEVE NUMBER

HYDROMETER



SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-1	12	30.0	Bag	SM	N/A	N/A

Gravel:	9.2%
Sand:	77.6%
Fine:	13.2%

EGLAB, INC.

Project Name:

Client Job No.: 203022-01
 Client Name: LGC Valley, Inc.
 EGLAB Project No.: 20-059-010

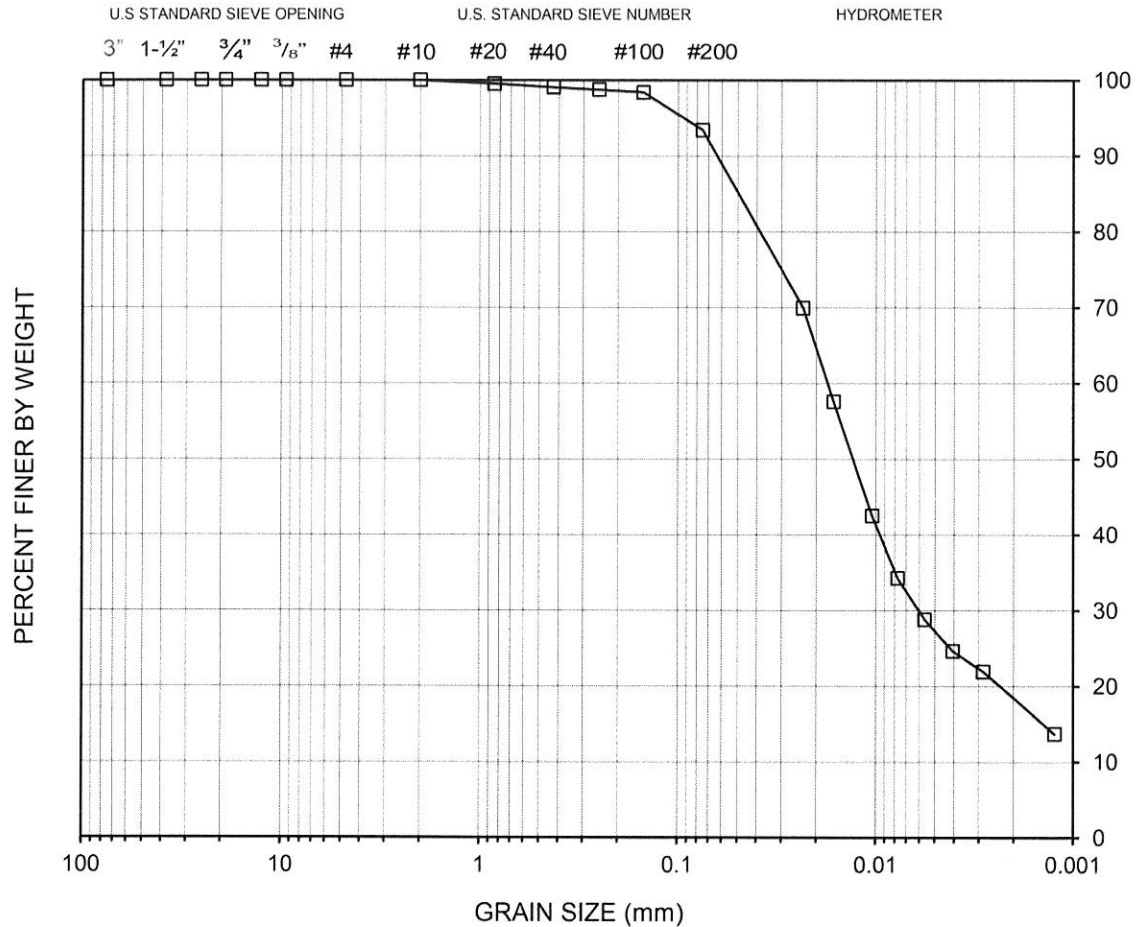
**GRAIN SIZE
DISTRIBUTION CURVE**

09/18/20

(ASTM D422)

FIGURE

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SYMBOL	BORING No.	SAMPLE ID.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-2	6	20.0	Bag	CL	49	25

Clay	27.1%
Sand	6.6%
Silt	66.2%

EGLAB, INC.	Project Name: FF / West LA
	Client: LGC Valley, Inc. Job No.: 203022-01 EGLAB Project No.: 20-059-010
GRAINSIZE DISTRIBUTION CURVE	
9/18/20	FIGURE

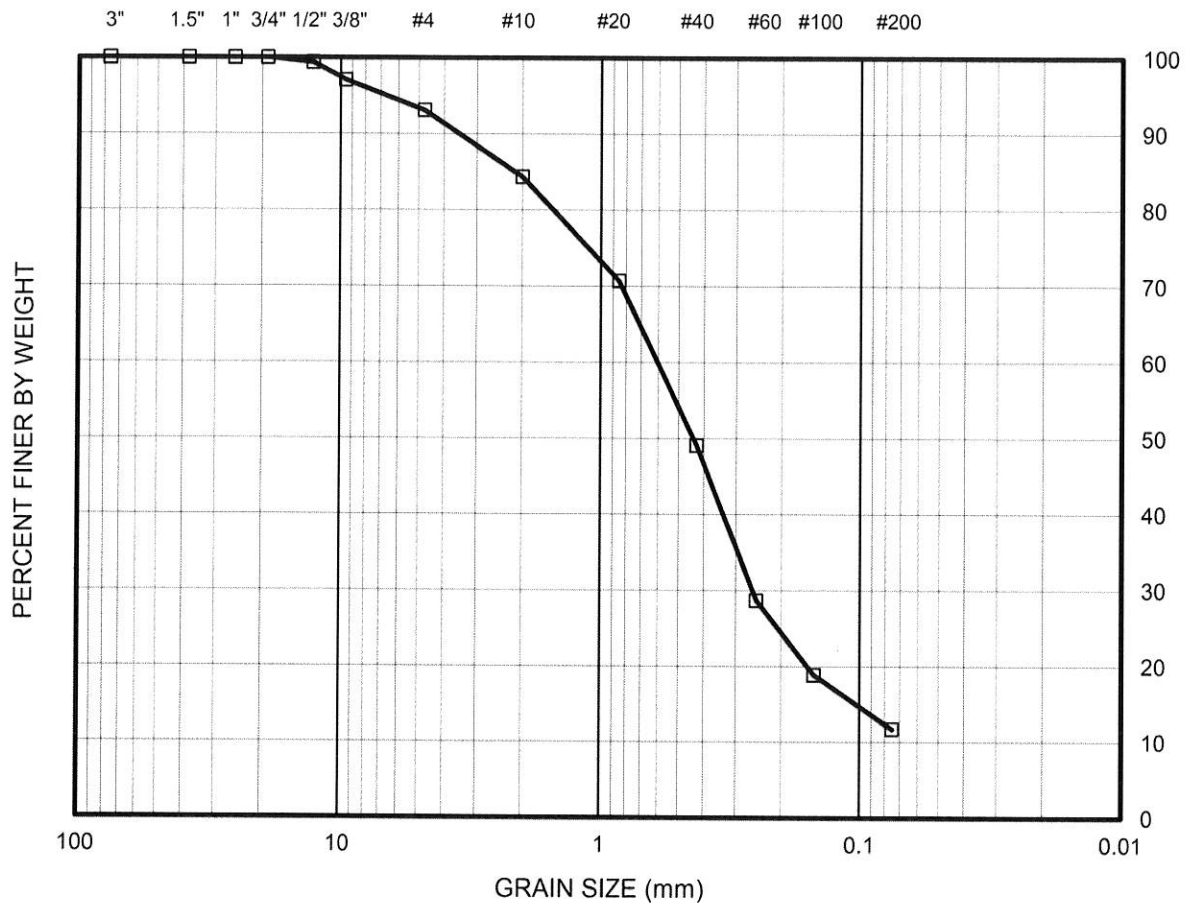
(ASTM D422)

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

U.S. STANDARD SIEVE OPENING

U.S. STANDARD SIEVE NUMBER

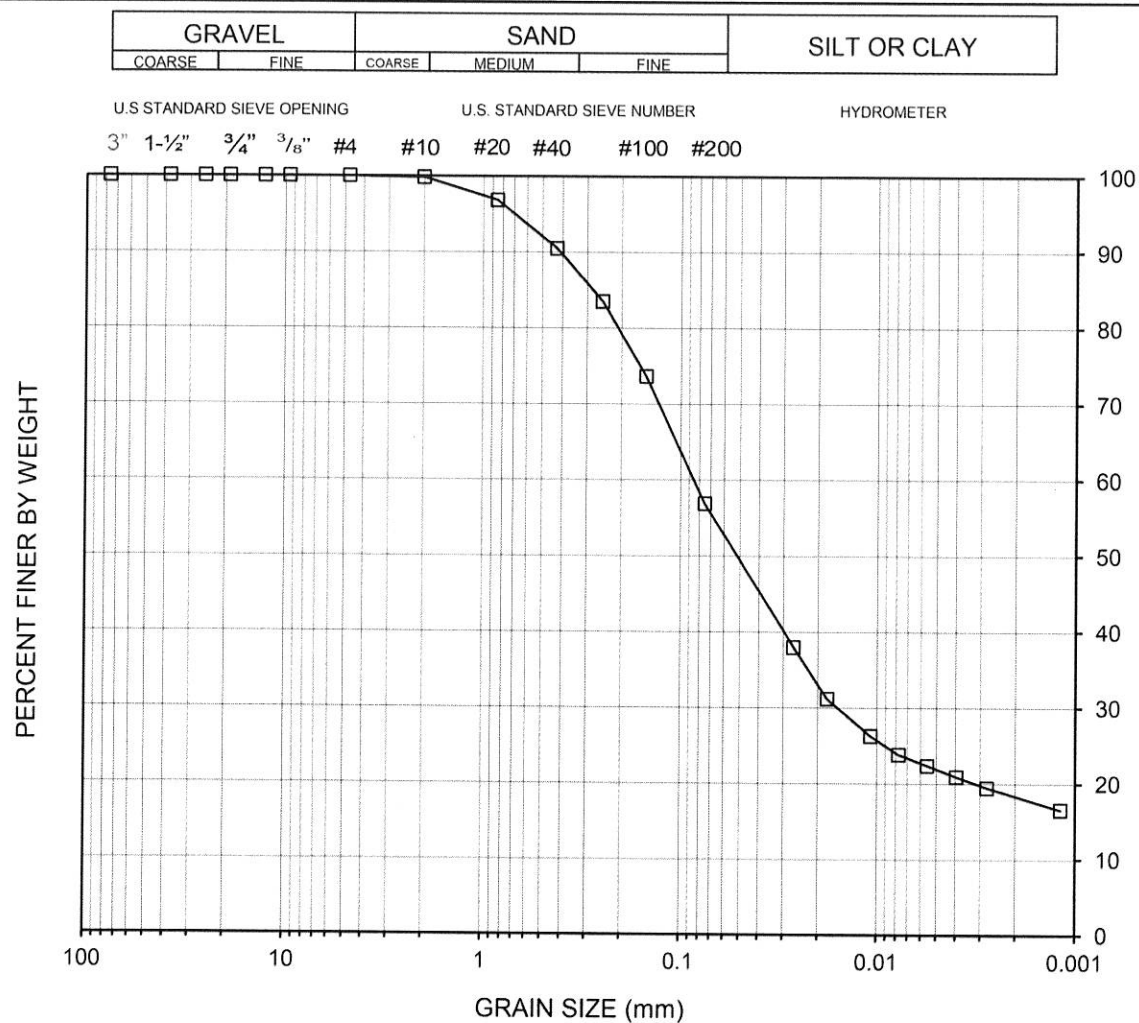
HYDROMETER



SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-2	8	30.0	Bag	SW-SM	N/A	N/A

Gravel:	6.9%
Sand:	81.4%
Fine:	11.7%

EGLAB, INC.	Project Name:	
	Client Job No.: 203022-01	Client Name: LGC Valley, Inc.
EGLAB Project No.: 20-059-010		
GRAIN SIZE DISTRIBUTION CURVE		
09/18/20	(ASTM D422)	FIGURE



SYMBOL	BORING No.	SAMPLE ID.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-3	5	15.0	Bag	CL	32	16

Clay	22.0%
Sand	43.1%
Silt	34.9%

EGLAB, INC.	Project Name: FF / West LA
	Client: LGC Valley, Inc. Job No.: 203022-01 EGLAB Project No.: 20-059-010
GRAINSIZE DISTRIBUTION CURVE	
9/18/20	FIGURE

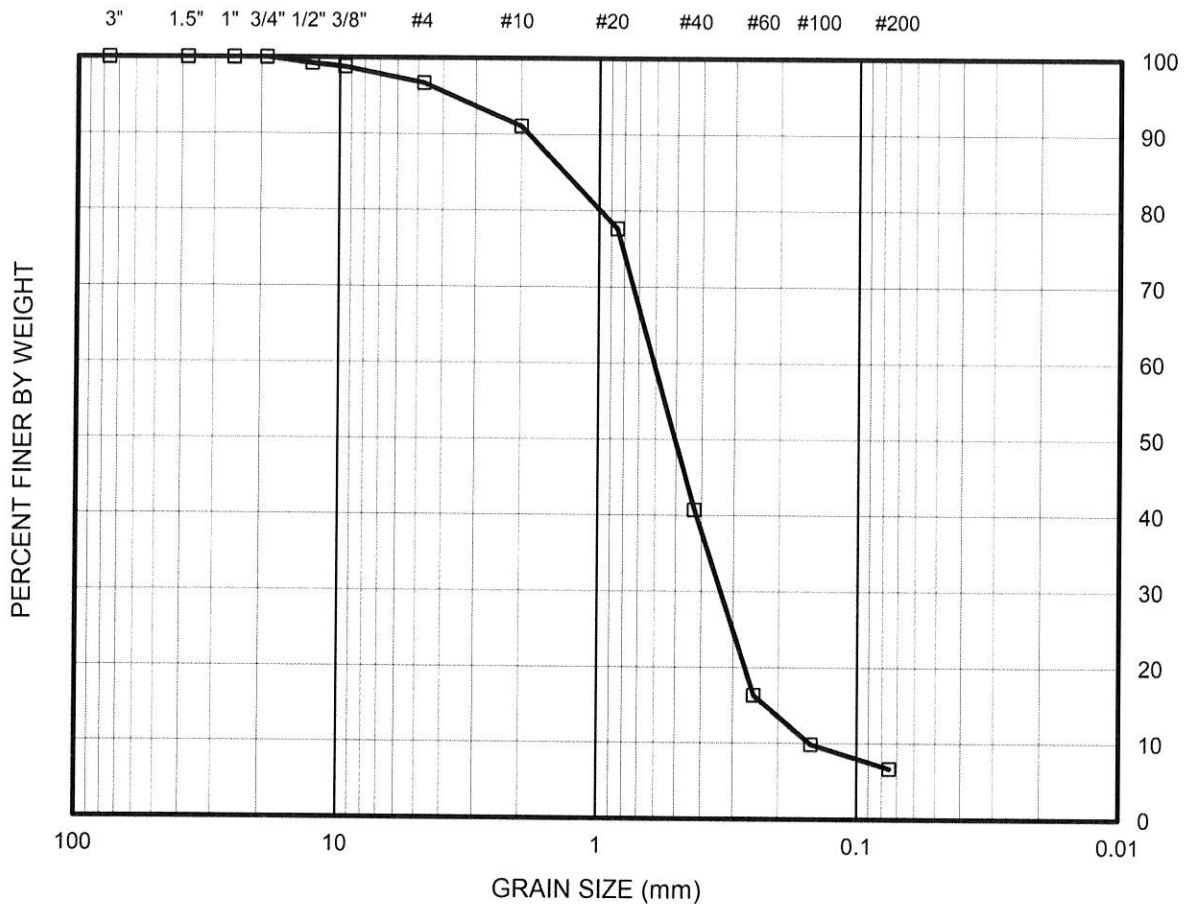
(ASTM D422)

GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

U.S. STANDARD SIEVE OPENING

U.S. STANDARD SIEVE NUMBER

HYDROMETER

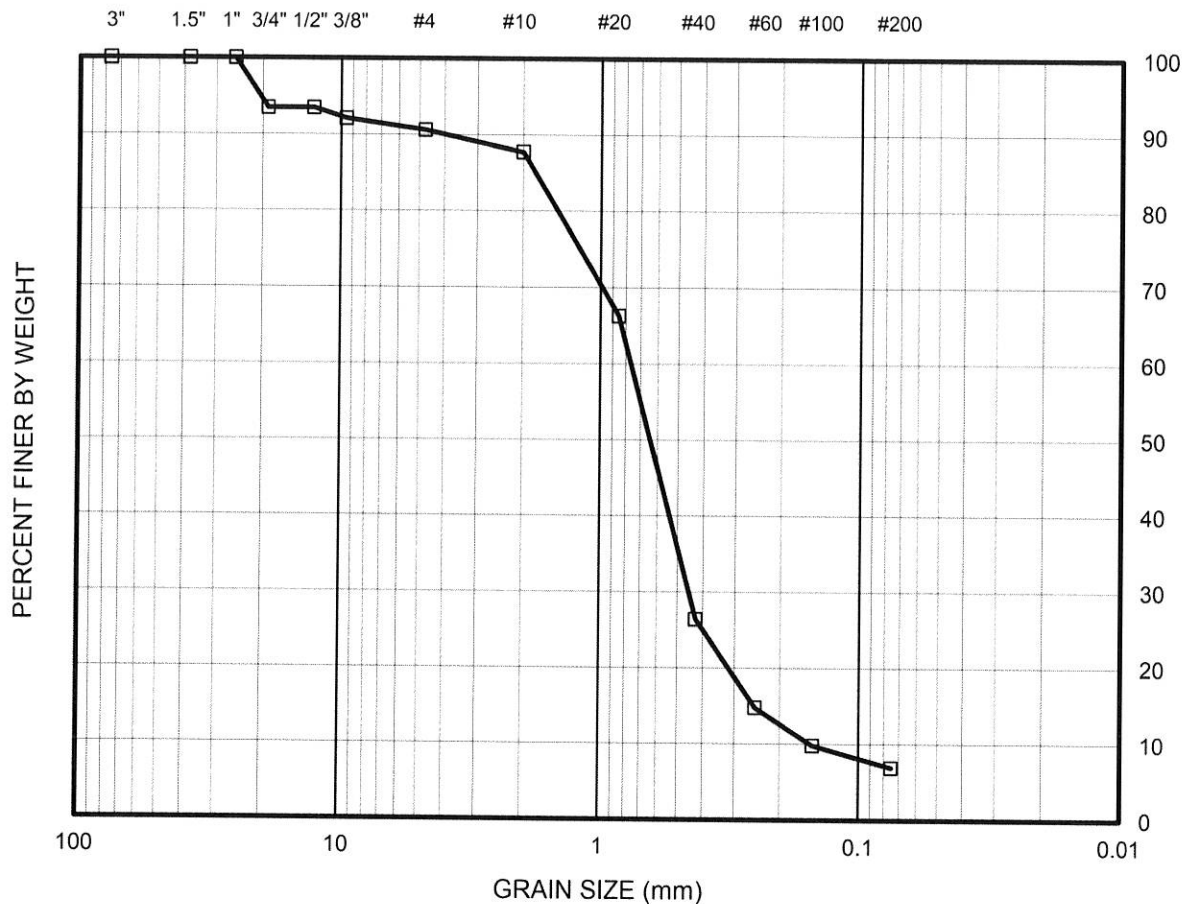


GRAVEL		SAND			SILT OR CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

U.S. STANDARD SIEVE OPENING

U.S. STANDARD SIEVE NUMBER

HYDROMETER



SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-3	9	35.0	Bag	SP-SM	N/A	N/A

Gravel:	9.4%
Sand:	83.7%
Fine:	6.9%

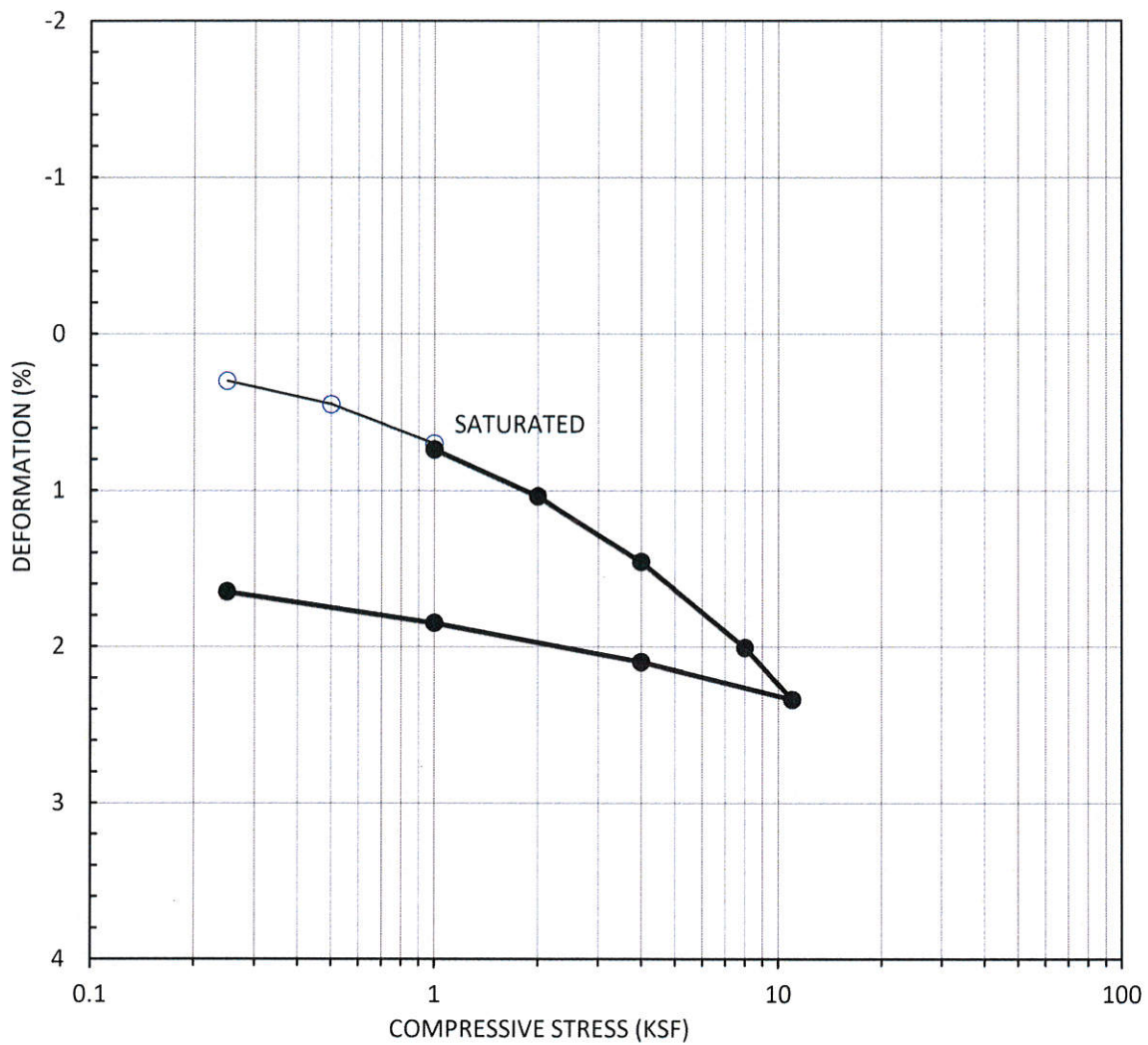
EGLAB, INC.	Project Name:	
	Client Job No.:	203022-01
	Client Name:	LGC Valley, Inc.
	EGLAB Project No.:	20-059-010
GRAIN SIZE DISTRIBUTION CURVE		
09/18/20	(ASTM D422)	FIGURE



SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	LIQUID LIMIT	PLASTICITY INDEX
□	B-3	12	50.0	Bag	SM	N/A	N/A

Gravel:	1.5%
Sand:	83.9%
Fine:	14.6%

EGLAB, INC.	Project Name:	
	Client Job No.:	203022-01
	Client Name:	LGC Valley, Inc.
	EGLAB Project No.:	20-059-010
GRAIN SIZE DISTRIBUTION CURVE		
(ASTM D422)		
09/18/20	FIGURE	



Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-1	2	5.0	SM	14.0	103.4	0.630

EGLAB, INC.

Project Name:

FF / West LA

Client:

LGC Valley, Inc.

Job No.:

203022-01

EGLAB Project No.:

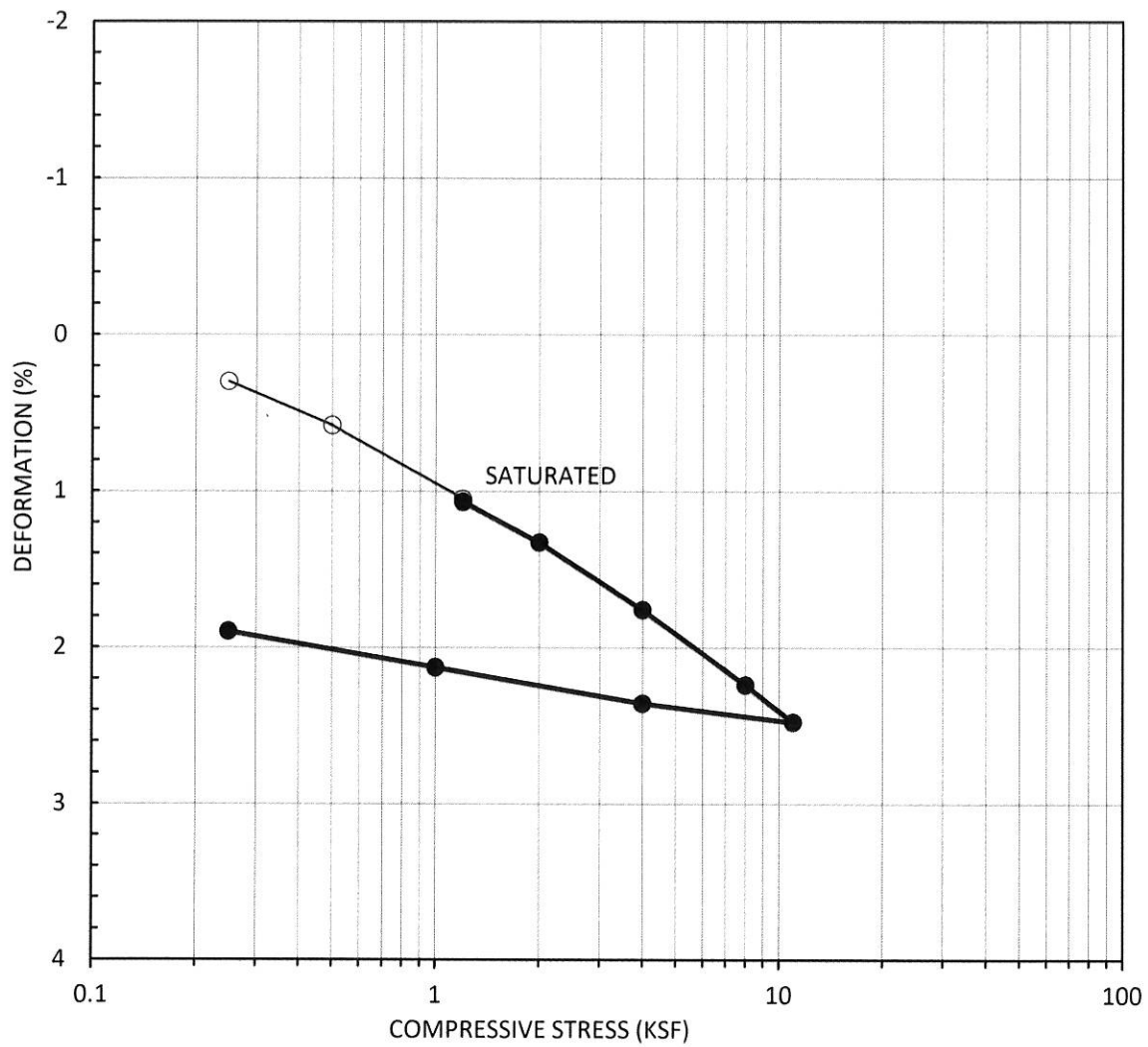
20-059-010

CONSOLIDATION

09/20

(ASTM D2435)

Figure



Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-1	3	7.5	SM	8.7	131.8	0.278

EGLAB, INC.

Project Name:

FF / West LA

Client:

LGC Valley, Inc.

Job No.:

203022-01

EGLAB Project No.:

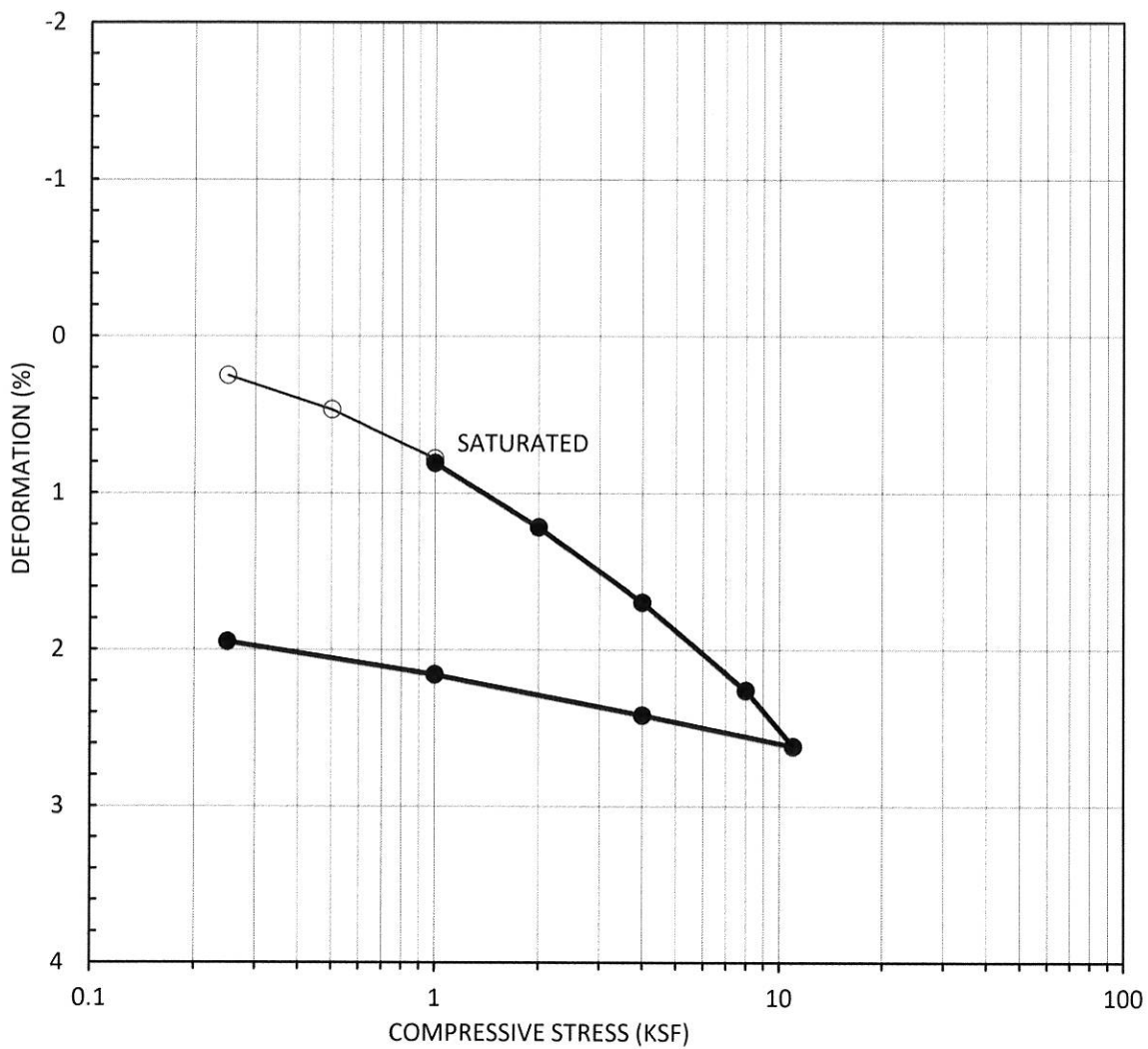
20-059-010

CONSOLIDATION

09/20

(ASTM D2435)

Figure



Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-2	2	5.0	SC	12.5	113.1	0.489

EGLAB, INC.

Project Name:

FF / West LA

Client:

LGC Valley, Inc.

Job No.:

203022-01

EGLAB Project No.:

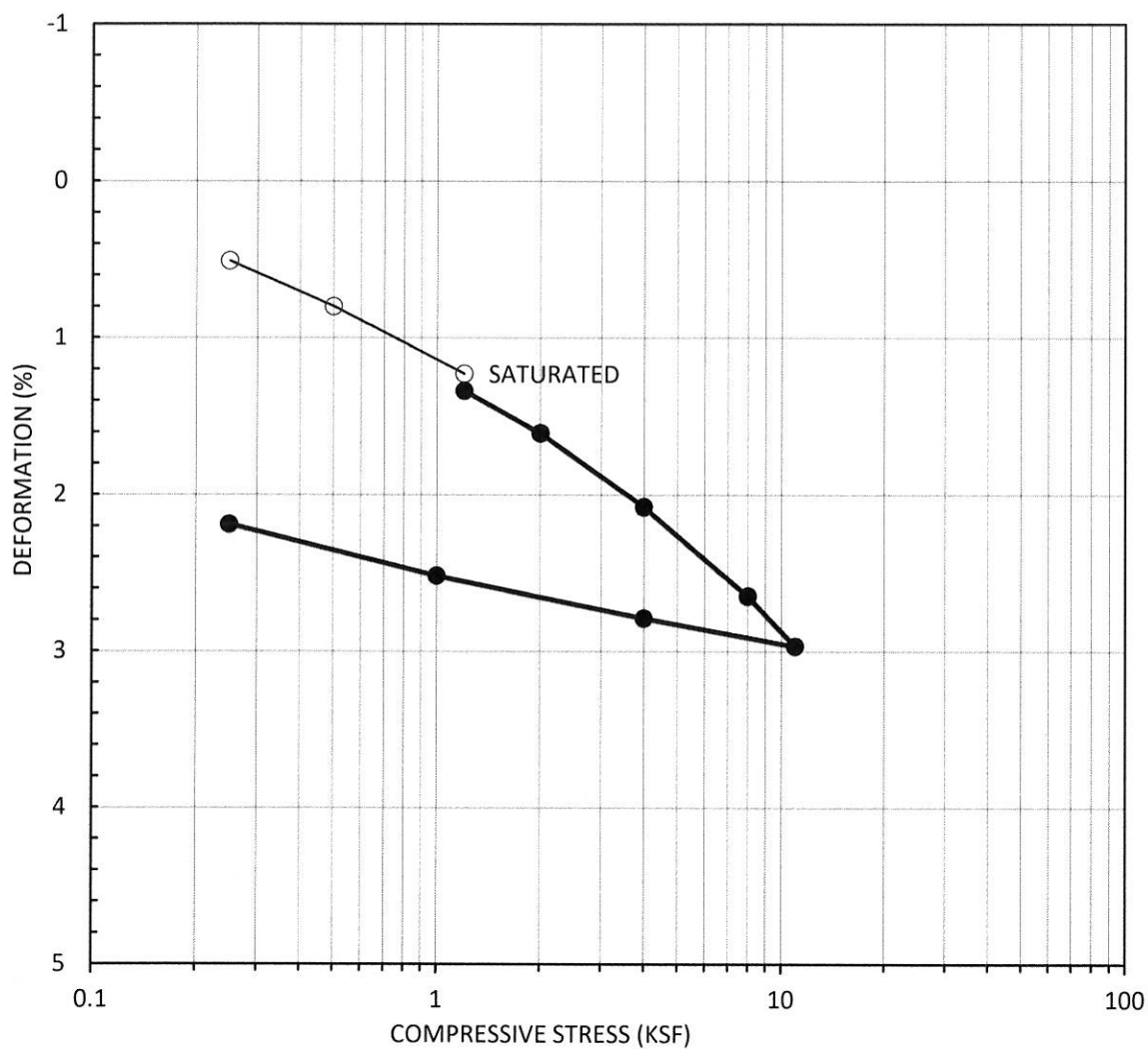
20-059-010

CONSOLIDATION

09/20

(ASTM D2435)

Figure



Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-2	3	7.5	SM	9.6	126.0	0.337

EGLAB, INC.

Project Name:

FF / West LA

Client:

LGC Valley, Inc.

Job No.:

203022-01

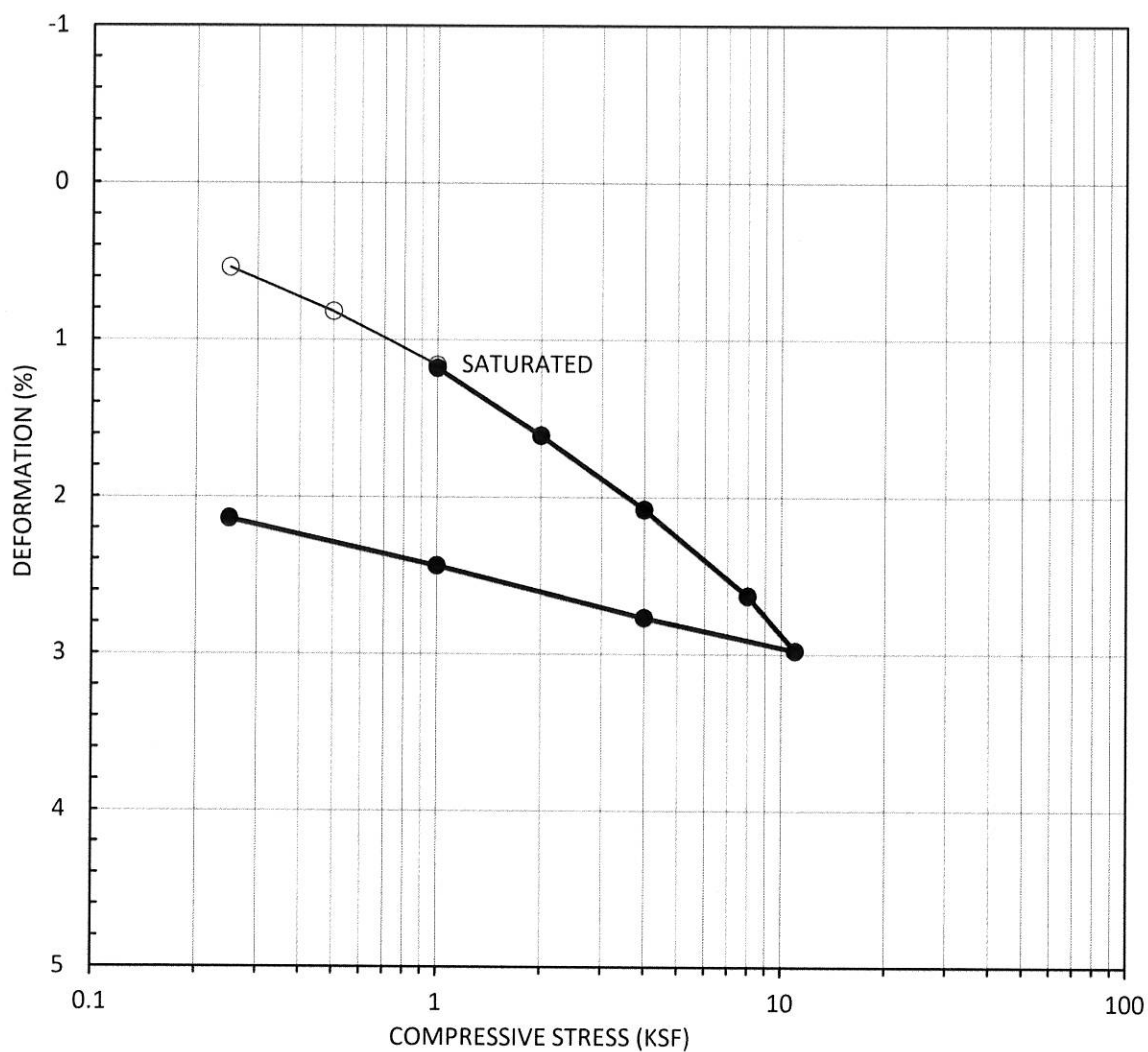
EGLAB Project No.: 20-059-010

CONSOLIDATION

09/20

(ASTM D2435)

Figure



Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-3	2	5.0	SM	14.6	111.7	0.509

EGLAB, INC.

Project Name:

FF / West LA

Client: LGC Valley, Inc.

Job No.: 203022-01

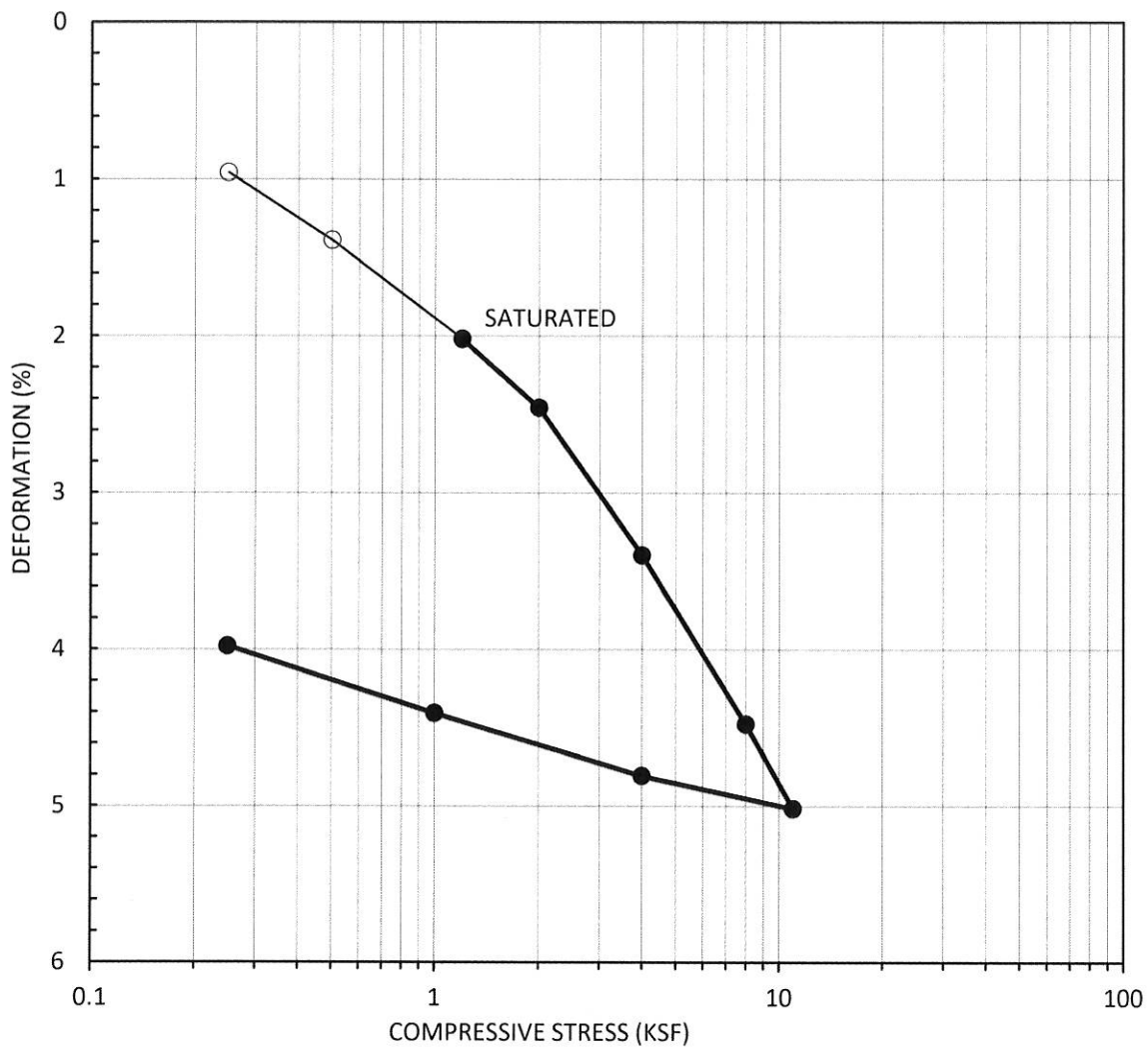
EGLAB Project No.: 20-059-010

CONSOLIDATION

09/20

(ASTM D2435)

Figure



Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-3	3	7.5	SC	12.0	129.3	0.303

EGLAB, INC.

Project Name:

FF / West LA

Client:

LGC Valley, Inc.

Job No.:

203022-01

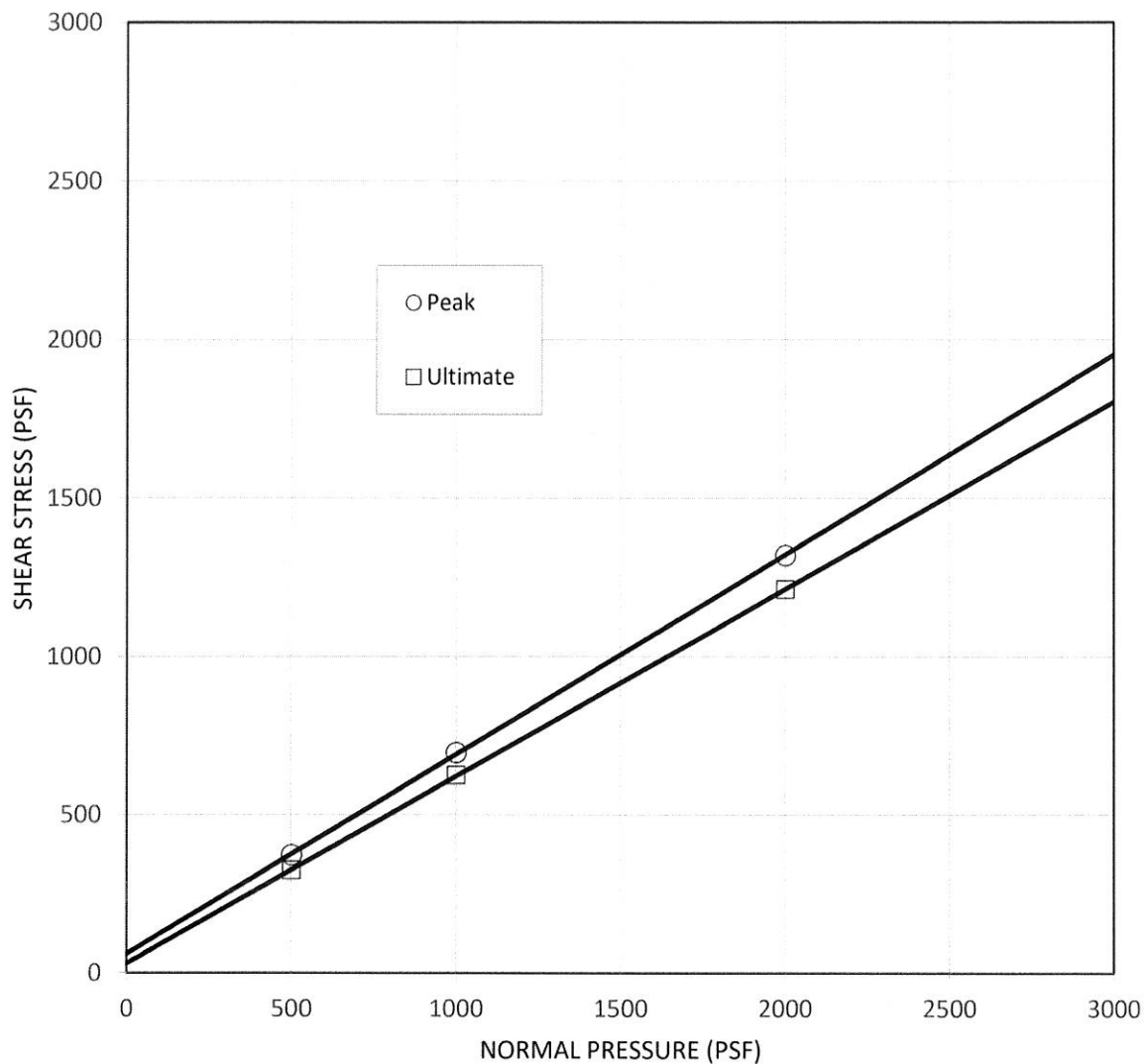
EGLAB Project No.: 20-059-010

CONSOLIDATION

09/20

(ASTM D2435)

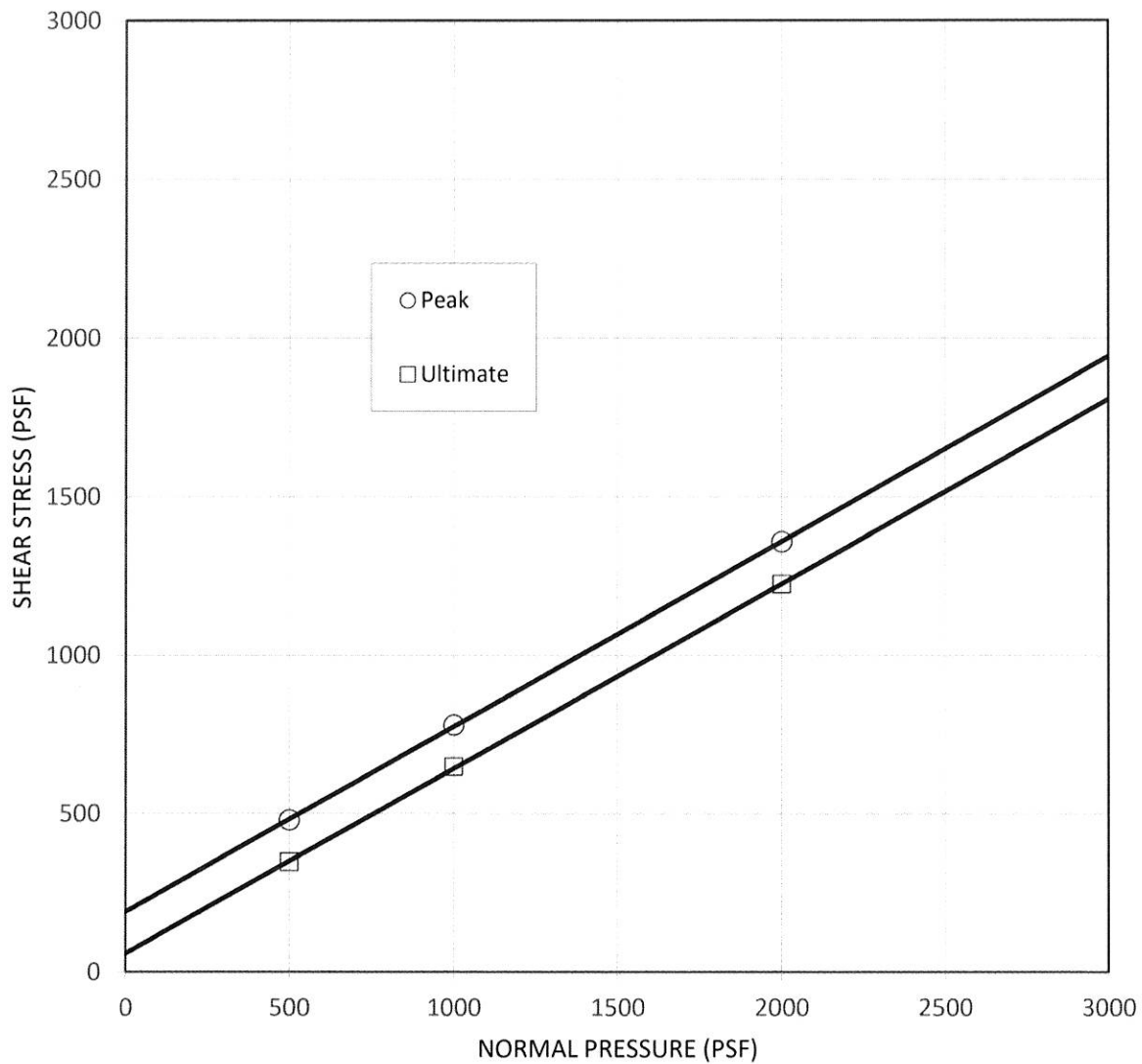
Figure



Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Type	Symbol	Cohesion (PSF)	Friction Angle
B-1	1	2.5	Ring	SM	○	60	32
					□	30	31

Normal Stress (psf)	Initial Moisture (%)	Final Moisture (%)
500	5.1	22.0
1000	5.1	21.7
2000	5.1	21.1

EGLAB, INC.	Project Name: FF / West LA	
	Client:	LGC Valley, Inc.
	Project No.:	203022-01
	EGLAB Project No.:	20-059-010
DIRECT SHEAR		
09/20	(ASTM D3080)	Figure



Boring No.	Sample No.	Depth (ft)	Sample Type	Soil Type	Symbol	Cohesion (PSF)	Friction Angle
B-1	Bag-1	0-5.0	Bulk	CL	○	191	30
					□	60	30

Note: Sample was remolded to **90 %** maximum relative density and optimum moisture

Maximum Dry Density: **130.0 pcf**

Optimum Moisture: **8.5 %**

Normal Stress (psf)	Initial Moisture (%)	Final Moisture (%)
500	8.5	16.7
1000	8.5	16.3
2000	8.5	15.9

EGLAB, INC.

Project Address:
FF / West LA

Client: LGC Valley, Inc.

Project No.: 203022-01

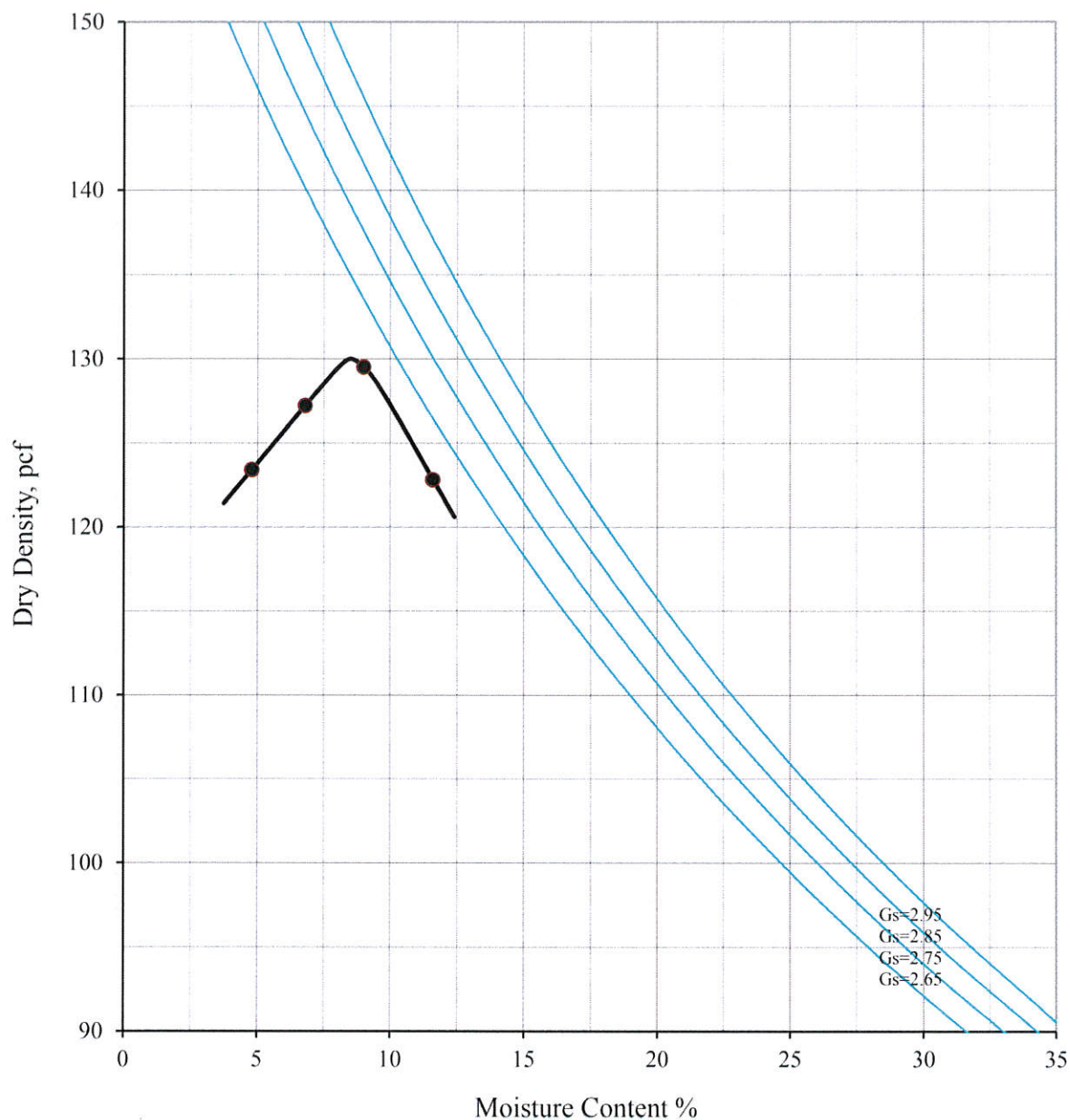
EGLAB Project No.: 20-059-010

DIRECT SHEAR

09/20

(ASTM D3080)

Figure



Maximum Dry Density = 130.0 pcf

Optimum Moisture Content = 8.5 %

EGLAB, INC.

Modified Proctor
(ASTM D1557)

Boring No: B-3

Sample: A

Depth : 3.0-4.0 feet

Description : Silty sand (SM), dark brown,
trace of gravel

Project Name:

FF / West LA

Client Name:

LGC Valley, Inc.

Job No.:

203022-01

EGLAB Project No.:

20-059-010

Date :

Sep-20

Figure

APPENDIX D

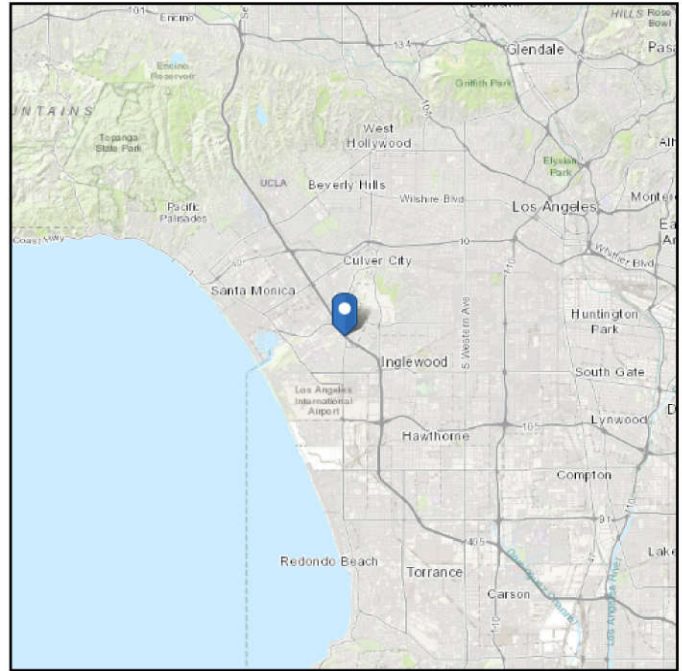
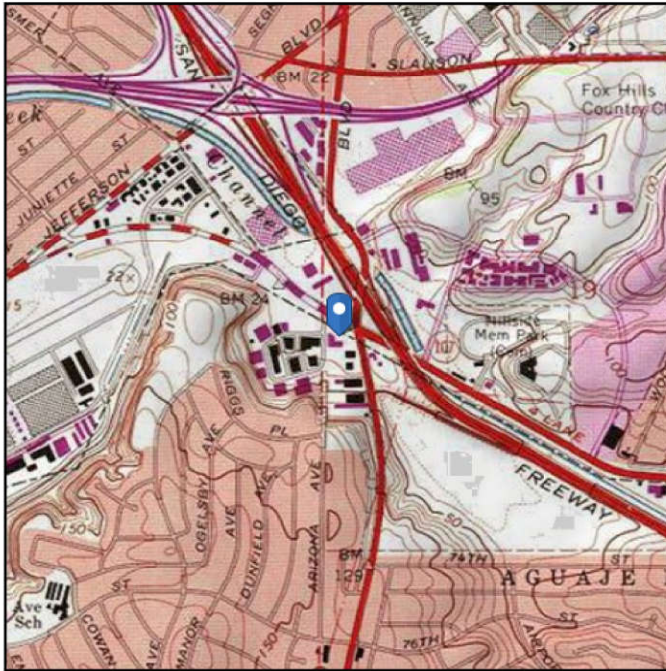
ASCE 7 Hazards Report and Liquefaction and Dry Sand Settlement Analysis

ASCE 7 Hazards Report

Address:
No Address at This
Location

Standard: ASCE/SEI 7-16
Risk Category: III
Soil Class: D - Stiff Soil

Elevation: 29.54 ft (NAVD 88)
Latitude: 33.980668
Longitude: -118.39535



Site Soil Class: D - Stiff Soil

Results:

S_S :	1.884	S_{D1} :	N/A
S_1 :	0.663	T_L :	8
F_a :	1	PGA :	0.806
F_v :	N/A	PGA _M :	0.887
S_{MS} :	1.884	F_{PGA} :	1.1
S_{M1} :	N/A	I_e :	1.25
S_{DS} :	1.256	C_v :	1.477

Ground motion hazard analysis may be required. See ASCE/SEI 7-16 Section 11.4.8.

Data Accessed: Thu Oct 01 2020

Date Source: [USGS Seismic Design Maps](#)

The ASCE 7 Hazard Tool is provided for your convenience, for informational purposes only, and is provided “as is” and without warranties of any kind. The location data included herein has been obtained from information developed, produced, and maintained by third party providers; or has been extrapolated from maps incorporated in the ASCE 7 standard. While ASCE has made every effort to use data obtained from reliable sources or methodologies, ASCE does not make any representations or warranties as to the accuracy, completeness, reliability, currency, or quality of any data provided herein. Any third-party links provided by this Tool should not be construed as an endorsement, affiliation, relationship, or sponsorship of such third-party content by or from ASCE.

ASCE does not intend, nor should anyone interpret, the results provided by this Tool to replace the sound judgment of a competent professional, having knowledge and experience in the appropriate field(s) of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the contents of this Tool or the ASCE 7 standard.

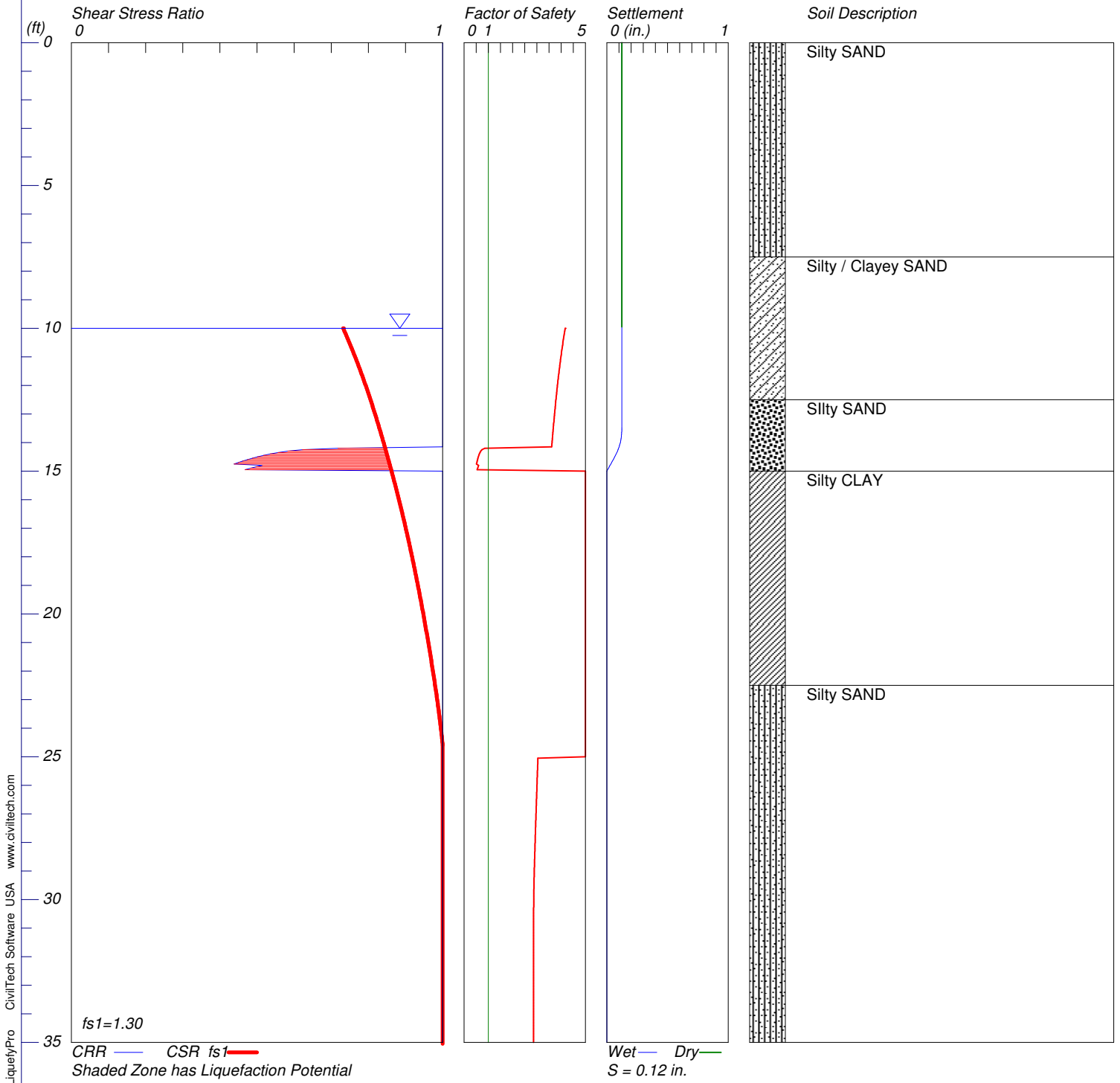
In using this Tool, you expressly assume all risks associated with your use. Under no circumstances shall ASCE or its officers, directors, employees, members, affiliates, or agents be liable to you or any other person for any direct, indirect, special, incidental, or consequential damages arising from or related to your use of, or reliance on, the Tool or any information obtained therein. To the fullest extent permitted by law, you agree to release and hold harmless ASCE from any and all liability of any nature arising out of or resulting from any use of data provided by the ASCE 7 Hazard Tool.

LIQUEFACTION ANALYSIS

FF West La

Hole No.=B-1 Water Depth=10 ft

Magnitude=6.36
Acceleration=0.887g



LIQUEFACTION ANALYSIS CALCULATION SHEET

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Input File Name: C:\Users\ARich\Desktop\FF West La\Project No. 203022-01

B-1.liq

Title: FF West La
Subtitle: Project No. 203022-01

Surface Elev.=
Hole No.=B-1
Depth of Hole= 39.0 ft
Water Table during Earthquake= 10.0 ft
Water Table during In-Situ Testing= 29.0 ft
Max. Acceleration= 0.89 g
Earthquake Magnitude= 6.4

Input Data:

Surface Elev.=
Hole No.=B-1
Depth of Hole=39.0 ft
Water Table during Earthquake= 10.0 ft
Water Table during In-Situ Testing= 29.0 ft
Max. Acceleration=0.89 g
Earthquake Magnitude=6.4

Earthquake Magnitude=6.4

2. Settlement Analysis Method: Tokimatsu / Seed

3. Fines Correction for Liquefaction: Idriss/Seed (SPT only)

4. Fine Correction for Settlement: During Liquefaction*

5. Settlement Calculation in: All zones*

6. Hammer Energy Ratio, Ce = 1.3

7. Borehole Diameter, Cb= 1

8. Sampling Method, Cs= 1.2

9. User request factor of safety (apply to CSR) , User= 1.3

Plot one CSR curve (fs1=User)

10. Use Curve Smoothing: Yes*

* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.0	17.0	117.9	NoLiq
5.0	13.0	117.9	NoLiq
7.5	25.0	143.3	NoLiq
10.0	29.0	138.0	6.8
12.5	31.0	138.0	6.8
15.0	17.0	120.0	NoLiq
17.5	17.0	120.0	NoLiq
20.0	11.0	120.0	NoLiq
22.5	37.0	120.0	NoLiq
25.0	33.0	138.0	24.0
27.5	65.0	138.0	24.0
30.0	53.0	138.0	13.0
32.5	74.0	138.0	13.0
35.0	50.0	138.0	13.0
37.5	50.0	138.0	13.0

Output Results:

Settlement of saturated sands=0.12 in.
Settlement of dry sands=0.00 in.
Total settlement of saturated and dry sands=0.12 in.
Differential Settlement=0.062 to 0.082 in.

Depth ft	CRRm	CSRs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.00	0.75	5.00	0.12	0.00	0.12
0.05	2.00	0.75	5.00	0.12	0.00	0.12
0.10	2.00	0.75	5.00	0.12	0.00	0.12
0.15	2.00	0.75	5.00	0.12	0.00	0.12
0.20	2.00	0.75	5.00	0.12	0.00	0.12
0.25	2.00	0.75	5.00	0.12	0.00	0.12
0.30	2.00	0.75	5.00	0.12	0.00	0.12
0.35	2.00	0.75	5.00	0.12	0.00	0.12
0.40	2.00	0.75	5.00	0.12	0.00	0.12
0.45	2.00	0.75	5.00	0.12	0.00	0.12
0.50	2.00	0.75	5.00	0.12	0.00	0.12
0.55	2.00	0.75	5.00	0.12	0.00	0.12
0.60	2.00	0.75	5.00	0.12	0.00	0.12
0.65	2.00	0.75	5.00	0.12	0.00	0.12
0.70	2.00	0.75	5.00	0.12	0.00	0.12
0.75	2.00	0.75	5.00	0.12	0.00	0.12
0.80	2.00	0.75	5.00	0.12	0.00	0.12
0.85	2.00	0.75	5.00	0.12	0.00	0.12
0.90	2.00	0.75	5.00	0.12	0.00	0.12
0.95	2.00	0.75	5.00	0.12	0.00	0.12
1.00	2.00	0.75	5.00	0.12	0.00	0.12

11.05	3.05	0.77	3.98	0.12	0.00	0.12
11.10	3.05	0.77	3.97	0.12	0.00	0.12
11.15	3.05	0.77	3.97	0.12	0.00	0.12
11.20	3.05	0.77	3.96	0.12	0.00	0.12
11.25	3.05	0.77	3.95	0.12	0.00	0.12
11.30	3.05	0.77	3.94	0.12	0.00	0.12
11.35	3.05	0.77	3.94	0.12	0.00	0.12
11.40	3.05	0.78	3.93	0.12	0.00	0.12
11.45	3.05	0.78	3.92	0.12	0.00	0.12
11.50	3.05	0.78	3.91	0.12	0.00	0.12
11.55	3.05	0.78	3.91	0.12	0.00	0.12
11.60	3.05	0.78	3.90	0.12	0.00	0.12
11.65	3.05	0.78	3.89	0.12	0.00	0.12
11.70	3.05	0.78	3.89	0.12	0.00	0.12
11.75	3.05	0.79	3.88	0.12	0.00	0.12
11.80	3.05	0.79	3.87	0.12	0.00	0.12
11.85	3.05	0.79	3.87	0.12	0.00	0.12
11.90	3.05	0.79	3.86	0.12	0.00	0.12
11.95	3.05	0.79	3.85	0.12	0.00	0.12
12.00	3.05	0.79	3.85	0.12	0.00	0.12
12.05	3.05	0.79	3.84	0.12	0.00	0.12
12.10	3.05	0.80	3.83	0.12	0.00	0.12
12.15	3.05	0.80	3.83	0.12	0.00	0.12
12.20	3.05	0.80	3.82	0.12	0.00	0.12
12.25	3.05	0.80	3.82	0.12	0.00	0.12
12.30	3.05	0.80	3.81	0.12	0.00	0.12
12.35	3.05	0.80	3.80	0.12	0.00	0.12
12.40	3.05	0.80	3.80	0.12	0.00	0.12
12.45	3.05	0.80	3.79	0.12	0.00	0.12
12.50	3.05	0.81	3.78	0.12	0.00	0.12
12.55	3.05	0.81	3.78	0.12	0.00	0.12
12.60	3.05	0.81	3.77	0.12	0.00	0.12
12.65	3.05	0.81	3.77	0.12	0.00	0.12
12.70	3.05	0.81	3.76	0.12	0.00	0.12
12.75	3.05	0.81	3.76	0.12	0.00	0.12
12.80	3.05	0.81	3.75	0.12	0.00	0.12
12.85	3.05	0.81	3.74	0.12	0.00	0.12
12.90	3.05	0.82	3.74	0.12	0.00	0.12
12.95	3.05	0.82	3.73	0.12	0.00	0.12
13.00	3.05	0.82	3.73	0.12	0.00	0.12
13.05	3.05	0.82	3.72	0.12	0.00	0.12
13.10	3.05	0.82	3.72	0.12	0.00	0.12
13.15	3.05	0.82	3.71	0.12	0.00	0.12
13.20	3.05	0.82	3.71	0.12	0.00	0.12
13.25	3.05	0.82	3.70	0.12	0.00	0.12
13.30	3.05	0.83	3.70	0.12	0.00	0.12
13.35	3.05	0.83	3.69	0.12	0.00	0.12
13.40	3.05	0.83	3.69	0.12	0.00	0.12
13.45	3.05	0.83	3.68	0.12	0.00	0.12
13.50	3.05	0.83	3.68	0.12	0.00	0.12

13.55	3.05	0.83	3.67	0.12	0.00	0.12
13.60	3.05	0.83	3.67	0.12	0.00	0.12
13.65	3.05	0.83	3.66	0.12	0.00	0.12
13.70	3.05	0.83	3.66	0.12	0.00	0.12
13.75	3.05	0.84	3.65	0.12	0.00	0.12
13.80	3.05	0.84	3.65	0.12	0.00	0.12
13.85	3.05	0.84	3.64	0.12	0.00	0.12
13.90	3.05	0.84	3.64	0.12	0.00	0.12
13.95	3.05	0.84	3.63	0.11	0.00	0.11
14.00	3.05	0.84	3.63	0.11	0.00	0.11
14.05	3.05	0.84	3.62	0.11	0.00	0.11
14.10	3.05	0.84	3.62	0.10	0.00	0.10
14.15	3.05	0.84	3.61	0.10	0.00	0.10
14.20	0.72	0.85	0.85*	0.10	0.00	0.10
14.25	0.62	0.85	0.73*	0.09	0.00	0.09
14.30	0.58	0.85	0.69*	0.09	0.00	0.09
14.35	0.56	0.85	0.66*	0.08	0.00	0.08
14.40	0.54	0.85	0.63*	0.08	0.00	0.08
14.45	0.52	0.85	0.61*	0.07	0.00	0.07
14.50	0.50	0.85	0.59*	0.07	0.00	0.07
14.55	0.49	0.85	0.57*	0.06	0.00	0.06
14.60	0.47	0.85	0.55*	0.05	0.00	0.05
14.65	0.46	0.85	0.54*	0.05	0.00	0.05
14.70	0.45	0.86	0.52*	0.04	0.00	0.04
14.75	0.44	0.86	0.51*	0.03	0.00	0.03
14.80	0.51	0.86	0.60*	0.03	0.00	0.03
14.85	0.50	0.86	0.58*	0.02	0.00	0.02
14.90	0.48	0.86	0.56*	0.01	0.00	0.01
14.95	0.47	0.86	0.54*	0.01	0.00	0.01
15.00	2.00	0.86	5.00	0.00	0.00	0.00
15.05	2.00	0.86	5.00	0.00	0.00	0.00
15.10	2.00	0.86	5.00	0.00	0.00	0.00
15.15	2.00	0.87	5.00	0.00	0.00	0.00
15.20	2.00	0.87	5.00	0.00	0.00	0.00
15.25	2.00	0.87	5.00	0.00	0.00	0.00
15.30	2.00	0.87	5.00	0.00	0.00	0.00
15.35	2.00	0.87	5.00	0.00	0.00	0.00
15.40	2.00	0.87	5.00	0.00	0.00	0.00
15.45	2.00	0.87	5.00	0.00	0.00	0.00
15.50	2.00	0.87	5.00	0.00	0.00	0.00
15.55	2.00	0.87	5.00	0.00	0.00	0.00
15.60	2.00	0.87	5.00	0.00	0.00	0.00
15.65	2.00	0.88	5.00	0.00	0.00	0.00
15.70	2.00	0.88	5.00	0.00	0.00	0.00
15.75	2.00	0.88	5.00	0.00	0.00	0.00
15.80	2.00	0.88	5.00	0.00	0.00	0.00
15.85	2.00	0.88	5.00	0.00	0.00	0.00
15.90	2.00	0.88	5.00	0.00	0.00	0.00
15.95	2.00	0.88	5.00	0.00	0.00	0.00
16.00	2.00	0.88	5.00	0.00	0.00	0.00

36.05	2.89	1.01	2.86	0.00	0.00	0.00
36.10	2.89	1.01	2.86	0.00	0.00	0.00
36.15	2.89	1.01	2.86	0.00	0.00	0.00
36.20	2.89	1.01	2.86	0.00	0.00	0.00
36.25	2.89	1.01	2.86	0.00	0.00	0.00
36.30	2.89	1.01	2.86	0.00	0.00	0.00
36.35	2.89	1.01	2.86	0.00	0.00	0.00
36.40	2.89	1.01	2.86	0.00	0.00	0.00
36.45	2.89	1.01	2.86	0.00	0.00	0.00
36.50	2.89	1.01	2.86	0.00	0.00	0.00
36.55	2.89	1.01	2.86	0.00	0.00	0.00
36.60	2.89	1.01	2.86	0.00	0.00	0.00
36.65	2.89	1.01	2.86	0.00	0.00	0.00
36.70	2.89	1.01	2.86	0.00	0.00	0.00
36.75	2.89	1.01	2.86	0.00	0.00	0.00
36.80	2.89	1.01	2.86	0.00	0.00	0.00
36.85	2.89	1.01	2.87	0.00	0.00	0.00
36.90	2.89	1.01	2.87	0.00	0.00	0.00
36.95	2.89	1.01	2.87	0.00	0.00	0.00
37.00	2.88	1.01	2.87	0.00	0.00	0.00
37.05	2.88	1.01	2.87	0.00	0.00	0.00
37.10	2.88	1.01	2.87	0.00	0.00	0.00
37.15	2.88	1.01	2.87	0.00	0.00	0.00
37.20	2.88	1.01	2.87	0.00	0.00	0.00
37.25	2.88	1.01	2.87	0.00	0.00	0.00
37.30	2.88	1.01	2.87	0.00	0.00	0.00
37.35	2.88	1.00	2.87	0.00	0.00	0.00
37.40	2.88	1.00	2.87	0.00	0.00	0.00
37.45	2.88	1.00	2.87	0.00	0.00	0.00
37.50	2.88	1.00	2.87	0.00	0.00	0.00
37.55	2.88	1.00	2.87	0.00	0.00	0.00
37.60	2.88	1.00	2.87	0.00	0.00	0.00
37.65	2.88	1.00	2.87	0.00	0.00	0.00
37.70	2.88	1.00	2.87	0.00	0.00	0.00
37.75	2.88	1.00	2.87	0.00	0.00	0.00
37.80	2.88	1.00	2.87	0.00	0.00	0.00
37.85	2.88	1.00	2.87	0.00	0.00	0.00
37.90	2.88	1.00	2.87	0.00	0.00	0.00
37.95	2.87	1.00	2.87	0.00	0.00	0.00
38.00	2.87	1.00	2.87	0.00	0.00	0.00
38.05	2.87	1.00	2.87	0.00	0.00	0.00
38.10	2.87	1.00	2.87	0.00	0.00	0.00
38.15	2.87	1.00	2.87	0.00	0.00	0.00
38.20	2.87	1.00	2.87	0.00	0.00	0.00
38.25	2.87	1.00	2.87	0.00	0.00	0.00
38.30	2.87	1.00	2.87	0.00	0.00	0.00
38.35	2.87	1.00	2.87	0.00	0.00	0.00
38.40	2.87	1.00	2.87	0.00	0.00	0.00
38.45	2.87	1.00	2.87	0.00	0.00	0.00
38.50	2.87	1.00	2.87	0.00	0.00	0.00

38.55	2.87	1.00	2.87	0.00	0.00	0.00
38.60	2.87	1.00	2.87	0.00	0.00	0.00
38.65	2.87	1.00	2.87	0.00	0.00	0.00
38.70	2.87	1.00	2.87	0.00	0.00	0.00
38.75	2.87	1.00	2.87	0.00	0.00	0.00
38.80	2.87	1.00	2.87	0.00	0.00	0.00
38.85	2.87	1.00	2.87	0.00	0.00	0.00
38.90	2.87	1.00	2.87	0.00	0.00	0.00
38.95	2.86	1.00	2.87	0.00	0.00	0.00
39.00	2.86	1.00	2.87	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone

(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units Depth = ft, Stress or Pressure = tsf (atm), Unit Weight =
pcf, Settlement = in.

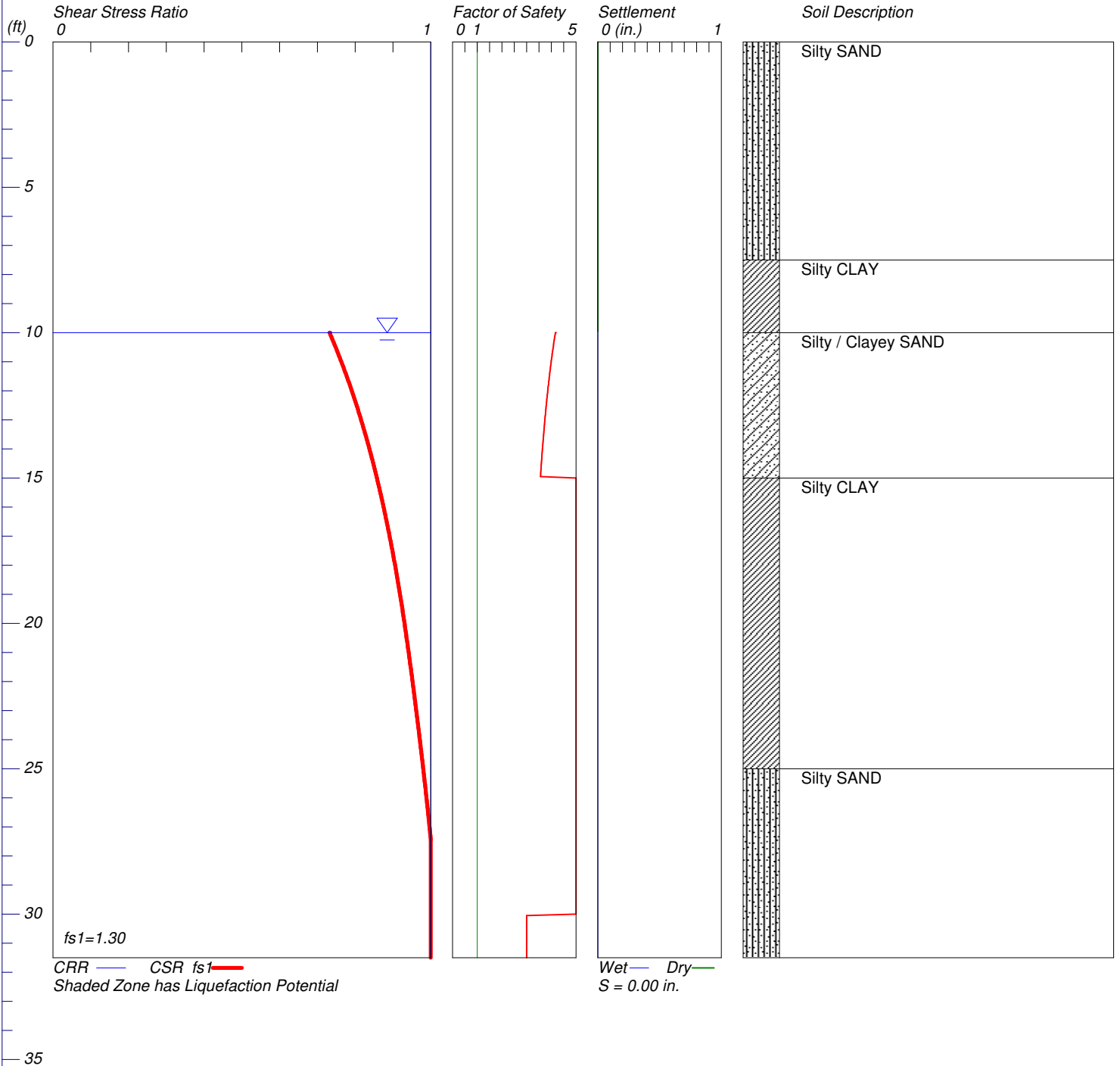
CRRm	Cyclic resistance ratio from soils
CSRfs	Cyclic stress ratio induced by a given earthquake (with user
request factor of safety)	
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRfs
S_sat	Settlement from saturated sands
S_dry	Settlement from dry sands
S_all	Total settlement from saturated and dry sands
NoLiq	No-Liquefy Soils

LIQUEFACTION ANALYSIS

FF West La

Hole No.=B-2

Magnitude=6.36
Acceleration=0.887g



LIQUEFACTION ANALYSIS CALCULATION SHEET

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Input File Name: C:\Users\ARich\Desktop\FF West La\Project No. 203022-01

B-2.liq

Title: FF West La
Subtitle: Project No. 203022-01

Surface Elev.=
Hole No.=B-2
Depth of Hole= 31.5 ft
Water Table during Earthquake= 10.0 ft
Water Table during In-Situ Testing= 10.0 ft
Max. Acceleration= 0.89 g
Earthquake Magnitude= 6.4

Input Data:

Surface Elev.=
Hole No.=B-2
Depth of Hole=31.5 ft
Water Table during Earthquake= 10.0 ft
Water Table during In-Situ Testing= 10.0 ft
Max. Acceleration=0.89 g
Earthquake Magnitude=6.4

Earthquake Magnitude=6.4

2. Settlement Analysis Method: Tokimatsu / Seed

3. Fines Correction for Liquefaction: Idriss/Seed (SPT only)

4. Fine Correction for Settlement: During Liquefaction*

5. Settlement Calculation in: All zones*

6. Hammer Energy Ratio, Ce = 1.3

7. Borehole Diameter, Cb= 1

8. Sampling Method, Cs= 1.2

9. User request factor of safety (apply to CSR) , User= 1.3

Plot one CSR curve (fs1=User)

10. Use Curve Smoothing: Yes*

* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.0	25.0	125.2	NoLiq
5.0	9.0	127.2	NoLiq
7.5	35.0	138.1	NoLiq
10.0	25.0	136.7	6.8
15.0	50.0	138.8	NoLiq
20.0	25.0	138.8	NoLiq
25.0	50.0	123.4	NoLiq
30.0	53.0	123.4	11.0

Output Results:

Settlement of saturated sands=0.00 in.
Settlement of dry sands=0.00 in.
Total settlement of saturated and dry sands=0.00 in.
Differential Settlement=0.000 to 0.000 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.00	0.75	5.00	0.00	0.00	0.00
0.05	2.00	0.75	5.00	0.00	0.00	0.00
0.10	2.00	0.75	5.00	0.00	0.00	0.00
0.15	2.00	0.75	5.00	0.00	0.00	0.00
0.20	2.00	0.75	5.00	0.00	0.00	0.00
0.25	2.00	0.75	5.00	0.00	0.00	0.00
0.30	2.00	0.75	5.00	0.00	0.00	0.00
0.35	2.00	0.75	5.00	0.00	0.00	0.00
0.40	2.00	0.75	5.00	0.00	0.00	0.00
0.45	2.00	0.75	5.00	0.00	0.00	0.00
0.50	2.00	0.75	5.00	0.00	0.00	0.00
0.55	2.00	0.75	5.00	0.00	0.00	0.00
0.60	2.00	0.75	5.00	0.00	0.00	0.00
0.65	2.00	0.75	5.00	0.00	0.00	0.00
0.70	2.00	0.75	5.00	0.00	0.00	0.00
0.75	2.00	0.75	5.00	0.00	0.00	0.00
0.80	2.00	0.75	5.00	0.00	0.00	0.00
0.85	2.00	0.75	5.00	0.00	0.00	0.00
0.90	2.00	0.75	5.00	0.00	0.00	0.00
0.95	2.00	0.75	5.00	0.00	0.00	0.00
1.00	2.00	0.75	5.00	0.00	0.00	0.00
1.05	2.00	0.75	5.00	0.00	0.00	0.00
1.10	2.00	0.75	5.00	0.00	0.00	0.00
1.15	2.00	0.75	5.00	0.00	0.00	0.00
1.20	2.00	0.75	5.00	0.00	0.00	0.00
1.25	2.00	0.75	5.00	0.00	0.00	0.00
1.30	2.00	0.75	5.00	0.00	0.00	0.00
1.35	2.00	0.75	5.00	0.00	0.00	0.00

31.40	3.05	1.02	3.00	0.00	0.00	0.00
31.45	3.05	1.02	3.00	0.00	0.00	0.00
31.50	3.05	1.02	3.00	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units Depth = ft, Stress or Pressure = tsf (atm), Unit Weight =
pcf, Settlement = in.

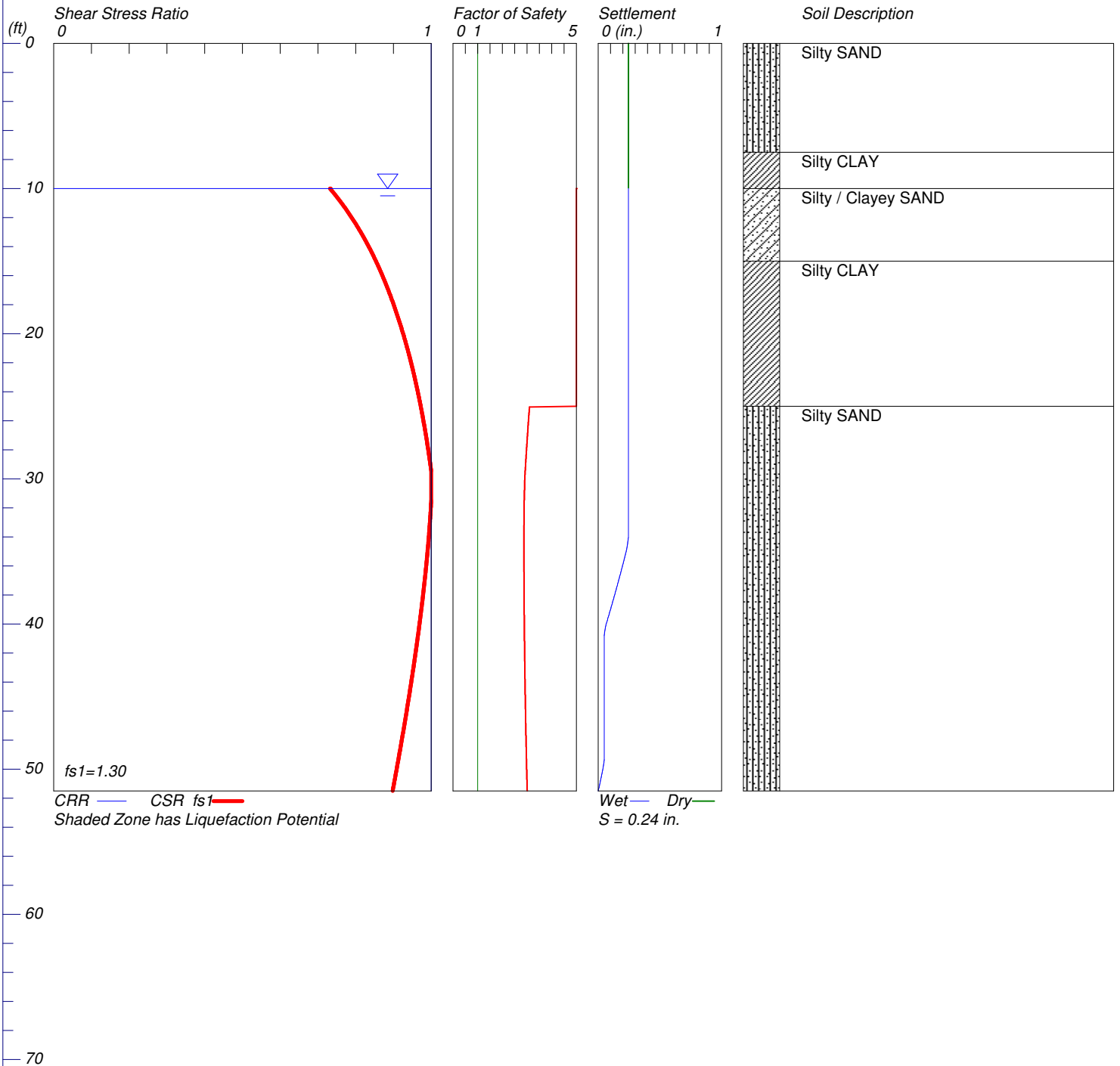
CRRm	Cyclic resistance ratio from soils
CSRfs	Cyclic stress ratio induced by a given earthquake (with user
request factor of safety)	
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRfs
S_sat	Settlement from saturated sands
S_dry	Settlement from dry sands
S_all	Total settlement from saturated and dry sands
NoLiq	No-Liquefy Soils

LIQUEFACTION ANALYSIS

FF West La

Hole No.=B-3 Water Depth=10 ft

Magnitude=6.36
Acceleration=0.887g



LIQUEFACTION ANALYSIS CALCULATION SHEET

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Input File Name: C:\Users\ARich\Desktop\FF West La\Project No. 203022-01

B-3.liq

Title: FF West La
Subtitle: Project No. 203022-01

Surface Elev.=
Hole No.=B-3
Depth of Hole= 51.5 ft
Water Table during Earthquake= 10.0 ft
Water Table during In-Situ Testing= 32.0 ft
Max. Acceleration= 0.89 g
Earthquake Magnitude= 6.4

Input Data:

Surface Elev.=
Hole No.=B-3
Depth of Hole=51.5 ft
Water Table during Earthquake= 10.0 ft
Water Table during In-Situ Testing= 32.0 ft
Max. Acceleration=0.89 g
Earthquake Magnitude=6.4

Earthquake Magnitude=6.4

2. Settlement Analysis Method: Tokimatsu / Seed

3. Fines Correction for Liquefaction: Idriss/Seed (SPT only)

4. Fine Correction for Settlement: During Liquefaction*

5. Settlement Calculation in: All zones*

6. Hammer Energy Ratio, Ce = 1.3

7. Borehole Diameter, Cb= 1

8. Sampling Method, Cs= 1.2

9. User request factor of safety (apply to CSR) , User= 1.3

Plot one CSR curve (fs1=User)

10. Use Curve Smoothing: Yes*

* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.0	11.0	128.0	NoLiq
5.0	6.0	128.0	NoLiq
7.5	26.0	144.8	NoLiq
10.0	25.0	141.2	NoLiq
15.0	17.0	141.2	NoLiq
20.0	37.0	135.8	NoLiq
25.0	58.0	135.8	6.6
30.0	47.0	135.3	6.6
35.0	32.0	135.3	6.6
40.0	33.0	135.3	6.6
45.0	58.0	135.3	6.6
50.0	36.0	135.3	6.6

Output Results:

Settlement of saturated sands=0.24 in.
Settlement of dry sands=0.00 in.
Total settlement of saturated and dry sands=0.24 in.
Differential Settlement=0.122 to 0.161 in.

Depth ft	CRRm	CSRs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	2.00	0.75	5.00	0.24	0.00	0.24
0.05	2.00	0.75	5.00	0.24	0.00	0.24
0.10	2.00	0.75	5.00	0.24	0.00	0.24
0.15	2.00	0.75	5.00	0.24	0.00	0.24
0.20	2.00	0.75	5.00	0.24	0.00	0.24
0.25	2.00	0.75	5.00	0.24	0.00	0.24
0.30	2.00	0.75	5.00	0.24	0.00	0.24
0.35	2.00	0.75	5.00	0.24	0.00	0.24
0.40	2.00	0.75	5.00	0.24	0.00	0.24
0.45	2.00	0.75	5.00	0.24	0.00	0.24
0.50	2.00	0.75	5.00	0.24	0.00	0.24
0.55	2.00	0.75	5.00	0.24	0.00	0.24
0.60	2.00	0.75	5.00	0.24	0.00	0.24
0.65	2.00	0.75	5.00	0.24	0.00	0.24
0.70	2.00	0.75	5.00	0.24	0.00	0.24
0.75	2.00	0.75	5.00	0.24	0.00	0.24
0.80	2.00	0.75	5.00	0.24	0.00	0.24
0.85	2.00	0.75	5.00	0.24	0.00	0.24
0.90	2.00	0.75	5.00	0.24	0.00	0.24
0.95	2.00	0.75	5.00	0.24	0.00	0.24
1.00	2.00	0.75	5.00	0.24	0.00	0.24
1.05	2.00	0.75	5.00	0.24	0.00	0.24
1.10	2.00	0.75	5.00	0.24	0.00	0.24
1.15	2.00	0.75	5.00	0.24	0.00	0.24

51.20	2.70	0.90	3.00	0.01	0.00	0.01
51.25	2.70	0.90	3.00	0.01	0.00	0.01
51.30	2.70	0.90	3.00	0.01	0.00	0.01
51.35	2.70	0.90	3.00	0.00	0.00	0.00
51.40	2.70	0.90	3.00	0.00	0.00	0.00
51.45	2.70	0.90	3.01	0.00	0.00	0.00
51.50	2.70	0.90	3.01	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units Depth = ft, Stress or Pressure = tsf (atm), Unit Weight =
pcf, Settlement = in.

CRRm	Cyclic resistance ratio from soils
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request factor of safety)	
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRfs
S_sat	Settlement from saturated sands
S_dry	Settlement from dry sands
S_all	Total settlement from saturated and dry sands
NoLiq	No-Liquefy Soils

APPENDIX E

LGC VALLEY, INC.

General Earthwork and Grading Specifications For Rough Grading

1.0 General

1.1 Intent: These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record: Prior to commencement of work, the owner shall employ a qualified Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to confirm that the attained level of compaction is being accomplished as specified. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

- 1.3 The Earthwork Contractor:** The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the project plans and specifications. The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of “equipment” of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate personnel will be available for observation and testing. . The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified. It is the contractor’s sole responsibility to provide proper fill compaction.

2.0 Preparation of Areas to be Filled

- 2.1 Clearing and Grubbing:** Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 10 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed. The contractor is responsible for all hazardous waste relating to his work. The Geotechnical Consultant does not have expertise in this area. If hazardous waste is a concern, then the Client should acquire the services of a qualified environmental assessor.

- 2.2 Processing:** Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free from oversize material and the working surface is reasonably uniform, flat, and free from uneven features that would inhibit uniform compaction.
- 2.3 Overexcavation:** In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.
- 2.4 Benching:** Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.
- 2.5 Evaluation/Acceptance of Fill Areas:** All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

- 3.1 General:** Material to be used as fill shall be essentially free from organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

- 3.2 **Oversize:** Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.
- 3.3 **Import:** If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 **Fill Placement and Compaction**

- 4.1 **Fill Layers:** Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.
- 4.2 **Fill Moisture Conditioning:** Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557-91).
- 4.3 **Compaction of Fill:** After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557-91). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.
- 4.4 **Compaction of Fill Slopes:** In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557-91.

4.5 Compaction Testing: Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing: Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations: The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

- 7.1** The Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.
- 7.2** All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of maximum from 1 foot above the top of the conduit to the surface.
- 7.3** The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.
- 7.4** The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.
- 7.5** Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.