Appendix D:
Preliminary Geotechnical Report

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LGC Valley, Inc.

Geotechnical Consulting

PRELIMINARY GEOTECHNICAL REPORT FOR THE PROPOSED RESIDENTIAL DEVELOPMENT 676 MOSS STREET, CHULA VISTA, CALIFORNIA

Dated: July 13, 2018

Project No. 184013-00

Prepared For:

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July 13, 2018 Project No. 184013-00

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Subject: ***Preliminary Geotechnical Investigation for the Proposed Residential Development of 676 Moss Street, Chula Vista, California***

In accordance with your request and authorization, LGC Valley, Inc. (LGC) has performed a preliminary geotechnical investigation of the proposed multi-family development located at 676 Moss Street in the City of Chula Vista, California. The purpose of our investigation and analysis was to evaluate the existing geologic and geotechnical conditions, and provide preliminary geotechnical design criteria. This report presents the results of our background review, field investigation, and laboratory testing. It also summarizes our geotechnical analysis of the collected data, and provides our conclusions, opinions, and recommendations relative to the proposed development of the site.

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

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

Respectfully submitted,

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

In accordance with your request and authorization, LGC Valley, Inc. (LGC) has performed a preliminary geotechnical investigation of the proposed multi-family development located at 676 Moss Street, Chula Vista, California (Figure 1 - Site Location Map). The purpose of our preliminary geotechnical investigation was to identify and evaluate the existing geologic and geotechnical conditions at the site as they pertain to the site configurations presented The Conceptual Site Plan-Ideas 1 and 2, prepared by WHA, dated June 8, 2018, and to provide preliminary geotechnical design criteria per the 2016 California Building Code requirements. Recommendations for grading, construction, preliminary foundation design for the proposed structures, and other relevant aspects of the proposed development are included herein to address the on-site geotechnical conditions. This report includes the results of our background review, site exploration, laboratory testing, and engineering evaluation of the site, and provides our conclusions, opinions, and recommendations with respect to site development.

Our scope of services for preparation of this document included:

- Review of pertinent available geotechnical literature/publications, geologic maps, and aerial photographs (Appendix A).

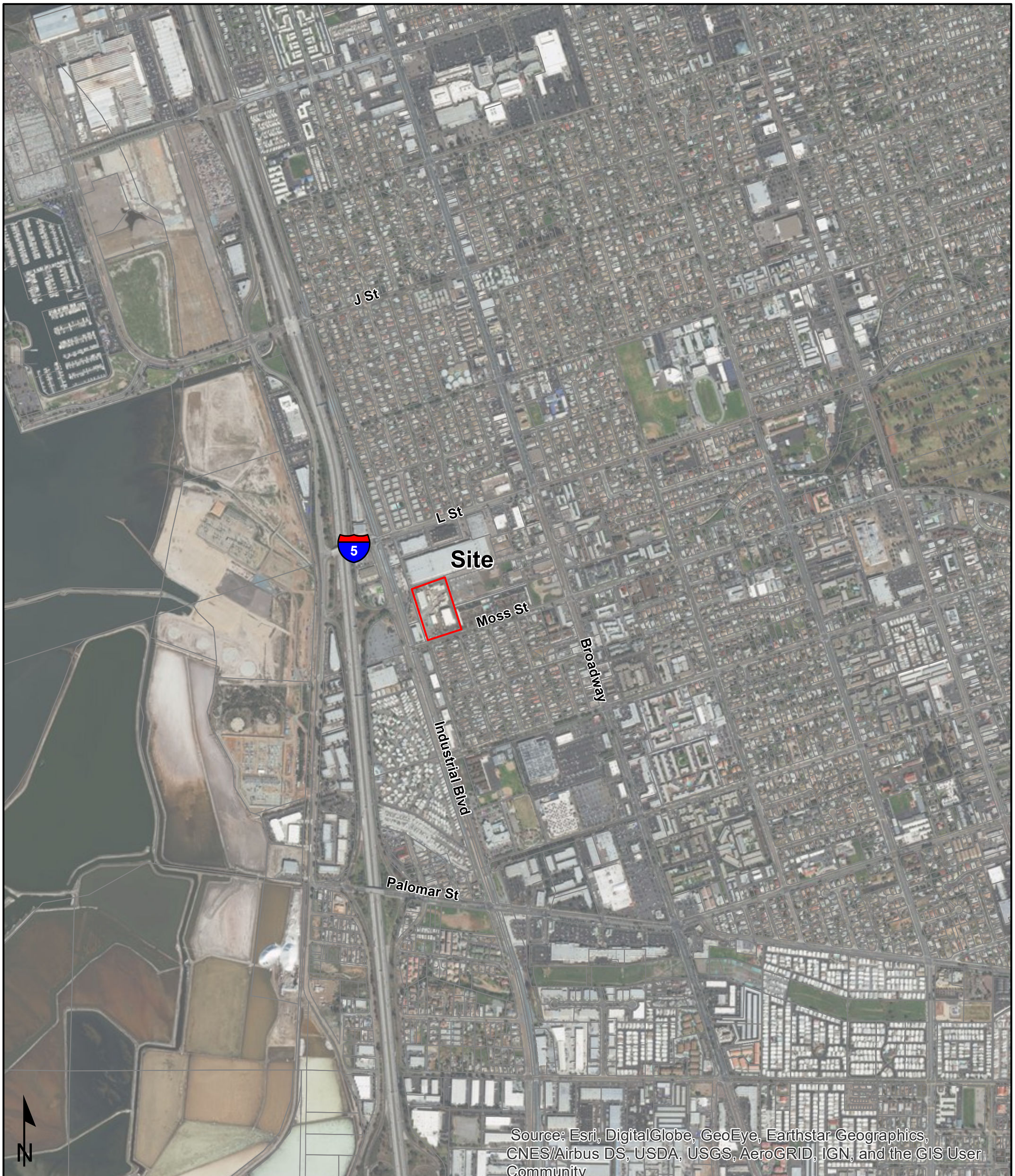
- Site reconnaissance and geologic mapping of the site.

- A subsurface investigation consisting of the excavation, sampling, and logging of six small-diameter exploratory borings that are labeled Borings B-1 through B-6. Logs of the borings are presented in Appendix B, and their approximate locations are depicted on the Geotechnical Map (Figure 2) and the Proposed Plan (Figure 3). The excavations were sampled and logged under the supervision of a licensed engineering geologist from our firm. The excavations were performed to evaluate the general characteristics of the subsurface conditions on the site including classification of site soils, determination of depth to competent soil and groundwater, and to obtain representative soil samples.

- Laboratory testing of representative soil samples obtained during our geotechnical investigation. Results of these tests are presented in Appendix C.

- Geotechnical evaluation of the data obtained during this study, including the potential for liquefaction, and seismic and static settlement.

- Preparation of this report presenting our findings, conclusions, opinions, and recommendations (including the General Earthwork and Grading Specifications for Rough Grading presented in Appendix D) with respect to the geologic and geotechnical conditions at the site.



LGC

Figure 1

Site Location Map

676 Moss Street
Chula Vista, California

Project Name	Shopoff/Moss St
Project Number	184013-00
Eng./Geo.	BIH/RKW
Date	June 13, 2018

0 0.4 Miles

1.2 Site Description and Proposed Development

The project site is located at 676 Moss Street, near the northeast corner of the intersection of Moss St and Industrial Blvd in the City of Chula Vista, California. 676 Moss Street consists of two parcels (APN 618-010-26-01 and 618-010-31-00) that are essentially rectangular-shaped, totaling approximately 6.91 acres/301,000 square-feet in size. The property is currently zoned for industrial use, but may be rezoned for residential use by the City of Chula Vista.

The property is bounded by a light railway/trolley system parallel and adjacent to Industrial Boulevard on the west; a large industrial building and light railway spur on the north; an industrial parking lot to the northeast, a multi-family residential development on the southeast; and Moss Street on the south. The site is generally flat, with elevations ranging from approximately 29 feet above mean sea level (msl) on the west edge of the site to 34 feet, msl along the east side of the site.

The site is currently being leased by three companies: a construction equipment rental company, scaffolding and storage bin suppliers, and a sand-blasting service company. Existing improvements on the property include three (3) existing buildings totaling approximately 37,100 square feet, paved asphalt driveways and parking lots, and an unimproved portion of the site located in the north west corner that is being used for container storage. The ALTA survey indicates the presence of many underground utilities including a large box culvert that transects the center of the site from east to west, and a storm drain system located in the southwest portion of the site, which terminates in the box culvert near the western edge of the site.

Based on the Conceptual Site Plans (WHA, 2018) we anticipate the site will consist of three-story rowhomes/townhomes with associated improvements including driveways, parking areas, concrete flatwork, underground utilities, pocket parks, and other landscaping. We also anticipate that some type of storm-water biofiltration system will be required for the site; however, this study does not include any field infiltration system since the locations of the basin(s) are unknown at this time.

1.3 Site History

The ALTA survey for the property indicates that there is an existing underground storm drain box culvert approximately 30 feet in width within a 45-foot wide easement with an unknown depth crossing the site. Based on our aerial photo review, it appears that for many years leading up to development, the site was transected by an open channel drainage system running in a generally northeast to southwest direction from the northeast corner of the property to the middle of the site on the west side. Photos from 1975 indicate that a smaller open drainage channel ran from the middle of the southern site boundary along the southwest portion of the site, and connected to the afore-mentioned larger channel along the west side of the project site.

By 1980, all of the current buildings on the site were built but the smaller drainage appears to have been filled in and likely replaced with underground storm drain pipes. By 1989, the large channel was filled in and replaced with the current box culvert. The box culvert was constructed in a more east-west direction, and located south of the prior open channel drainage. The location of the box culvert is presented on Figures 2 and 3, while aerial photographs of the site from 1953 and 1975 are presented on Figure 4.

1.4 Subsurface Investigation and Laboratory Testing

Our subsurface investigation was performed on June 21, 2018 and consisted of advancing six hollow-stem 8-inch diameter hollow-stem auger borings. Three borings were drilled to a depth of 51 feet below ground surface, two borings were drilled to a depth of approximately 16 feet below ground surface, and one boring hit refusal at 1.5 feet below ground surface when the box culvert was encountered. The approximate locations of the borings are shown on the Geotechnical Map (Figure 2).

During the subsurface investigation, representative bulk and relatively undisturbed samples were collected for laboratory testing, and samples were forwarded to EGLAB, Inc. (EGL) and to LGC Valley, Inc. for classification testing. Laboratory testing included moisture and density tests, maximum density and optimum moisture content, consolidation testing, sieve and hydrometer analysis, Atterberg Limits, remolded direct shear, expansion index and corrosion suite testing. A summary of the test procedures and laboratory test results are presented in Appendix C. The moisture and density test results are presented on the boring logs included in Appendix B.

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional and Local Geology

The site is located on the southeast side of San Diego Bay near the mouth of the Otay River within the Coastal Plain Region of San Diego County. The Coastal Plain Region is the westernmost territory of three distinct regions of San Diego County and is characterized by Mesozoic-age basement rocks overlain by a thick sequence of Cenozoic and Quaternary-aged marine and non-marine sedimentary rocks. The general area of the site is composed of Quaternary-aged young alluvial deposits. Subsequent to the deposition of this unit, erosion and regional tectonic uplift created the valleys and ridges of the area. Recent weathering and erosional processes have produced sedimentary undifferentiated Holocene-aged sediments (i.e. the young alluvial flood-plain deposits) while human influences have created the undocumented fill soils that mantle portions of the site.

2.2 Site-Specific Geology

The majority of the subject site is underlain by a thin layer of undocumented fills (undifferentiated) over Quaternary-aged young alluvial flood-plain deposits (map symbol Qya), with localized areas of deeper undocumented fill (map symbol Afu) corresponding to areas of the existing structures and old drainage courses. Although not encountered during our exploration, it is anticipated that the Quaternary-aged old paralic deposits (i.e. the Bay Point Formation) exist at a depth below the Quaternary-aged young alluvial flood-plain deposits.

The undocumented fill soils encountered on the site are dark brown silty clay, sandy clay, and clayey sand derived of onsite alluvial flood-plain deposits. Fills below the existing buildings are expected to be relatively shallow, while the fills above the old drainage channels are expected to possibly exceed 12 feet in depth. These soils are considered to be compressible and unsuitable in its present state, and should be completely removed to competent soil. The material removed may be placed as engineered fill per Section 4.1.5 of this report.

The Quaternary-aged young alluvial flood-plain deposits encountered on the site are predominantly massively bedded, reddish-brown, dark brown, and gray-brown, weakly cemented, stiff to hard, silty to sandy clay, clayey silt, and silty to clayey sand. This unit was encountered in our borings to the depth of our deepest borings (51 feet, bgs). The top 3 to 5 feet of this unit (in addition to any overlying fill) should be considered compressible and should be removed and may be placed as engineered fill.

2.3 Geologic Structure

Based on our subsurface investigation, review of the geologic maps of the general vicinity (Appendix A) and our professional experience, the Quaternary alluvial flood-plain deposits are generally massive to thickly bedded. Bedding within the unit is flat lying and thus not considered significant from a geotechnical perspective.

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 in the three deep geotechnical borings at elevations of approximately 32 feet below ground surface (bgs), or approximately at sea level. Given these conditions, groundwater is not anticipated to be encountered during grading, and should not provide any constraint to development. However; in general, groundwater levels will fluctuate with seasonal variations and local zones of perched groundwater may occur within the near-surface deposits when precipitation is high.

2.6 ***Surface Water and Flooding***

Based on our review of local topographic maps, sheet flow is to the west/southwest. Surface water runoff relative to project design is the purview of the project civil engineer and should be directed away from the planned structures.

LGC reviewed the applicable Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FIRM) relative to the site and general vicinity. Based on our review of FIRM Map Number 06073C2154H, Panel 2152 of 2375 (USFEMA, 2016), the site is located within Flood Zone X, defined as areas of 500-year flood; areas of 100-year flood with average depths less than 1-foot; and areas protected by levees from 100-year flood. The box culvert transecting the site is classified as Zone A: 100-year flood contained in channel.

2.7 ***Faulting, Seismicity, and Related Effects***

2.7.1 ***Faulting***

The southern California region has long been recognized as being seismically active. The seismic activity results from a number of active faults that cross the region, all of which are related to the San Andreas transform system, a broad zone of right lateral faults that extend from Baja California to Cape Mendocino. The numerous faults in Southern California include Holocene-active and pre-Holocene faults. The definitions of fault activity terms used here are based on those developed for the Alquist-Priolo (AP) Special Studies Zone Act of 1972 that was recently updated in January 2018 (CGS, 2018).

Holocene-active faults are faults that have had surface displacement within Holocene time (i.e. the last 11,700 years before the present [BP]). Faults are considered pre-Holocene if the past movement is older than 11,700 years BP. A third category, age-undetermined faults, are faults where the recency of faulting has not been determined (i.e. “a fault can be considered age-undetermined if the fault in question has simply not been studied in order to determine its recency of movement [CGS, 2018]).

The site is not within a currently established Alquist-Priolo Earthquake Fault Zone for fault rupture hazard (formerly Special Studies Zones for fault rupture hazard). Based on a review of geologic literature, no Holocene-active faults are known to occur beneath the site. Accordingly, it appears that there is little probability of surface rupture due to faulting beneath the site. There are, however, several faults located in sufficiently close proximity that movement associated with them could cause significant ground motion at the site. Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the southern California region include soil liquefaction and dynamic settlement. Other secondary seismic effects include shallow ground rupture, 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.

Regional active faults that occur within the Chula Vista area include the on-shore and off-shore Rose Canyon-Newport Inglewood fault zone to the west, and the Coronado Bank and San Diego Trough faults offshore to the west. The closest known active fault splay within the Rose Canyon fault zone is located approximately 3.8 miles (6.1 kilometers) to the west of the site, and the closest known active fault splay within the Coronado Bank fault zone with is located approximately 15.0 miles (24.1 kilometers) to the west.

2.7.2 Seismicity

The main seismic parameters to be considered when discussing the potential for earthquake-induced damage 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.

2.7.3 Seismic Design Criteria

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2016 California Building Code (CBC). Representative site coordinates of latitude 33.61325º N and longitude -117.08790º 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 Table 1.

Table 1 **California Building Code Site Seismic Characteristics**	
Selected Parameters from 2016 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)*	1.015g
Risk-Targeted Spectral Accelerations for 1-Second Periods (S_1) *	0.384g
Site Coefficient F_a per Table 1613.3.3(1)	1.1
Site Coefficient F_v per Table 1613.3.3(2)	1.6
Site Modified Spectral Acceleration for Short Periods (S_{MS}) for Site Class D [Note: $S_{MS} = F_a S_S$]	1.110g
Site Modified Spectral Acceleration for 1-Second Periods (S_{M1}) for Site Class D [Note: $S_{M1} = F_v S_1$]	0.627g
Design Spectral Acceleration for Short Periods (S_{DS}) for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	0.740g
Design Spectral Acceleration for 1-Second Periods (S_{D1}) for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.418g
Mapped Risk Coefficient at 0.2 sec Spectral Response Period, C_{RS} (per ASCE 7)	0.889
Mapped Risk Coefficient at 1 sec Spectral Response Period, C_{R1} (per ASCE 7)	0.933

* From USGS, 2013

Section 1803.5.12 of the 2016 California Building Code (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 0.46g (USGS, 2013). A deaggregation of the PGA based on a 2,475-year average return period indicates that an earthquake magnitude of 6.94 at a distance of approximately 15.02 km (9.33 mi) from the site would contribute the most to this ground motion (USGS, 2014).

2.7.4 Lurching and Shallow Ground Rupture

Soil lurching refers to the rolling motion on the ground surface by the passage of seismic surface waves. Effects of this nature are not likely to be significant where the thickness of soft sediments do not vary appreciably under the structure. Although there are several nearby active and potentially active faults, the native soils are generally dense. Based on this data, it is our opinion that the potential for lurching or shallow rupture at the site is low.

2.7.5 Liquefaction and Dynamic 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.

The Environmental Element within the City of Chula Vista General Plan indicates this site is located in an area where liquefaction analysis is encouraged (City of Chula Vista, 2015). However, based on our geotechnical subsurface investigation, the standard penetration test (SPT) blow counts were observed to be relatively high in shallow depths. Water table depth varies between 30 to 33 feet, bgs at this site. Subsequently, considering the type of the soil, relatively high SPT blow counts, and existing deep ground water table, the potential for liquefaction is not a source of concern on this site.

2.7.6 Seismically Induced Settlement

Based on our liquefaction evaluation discussed in Section 2.7.5 of this report, the subject site is not prone to liquefaction; and therefore, seismically induced settlements from liquefaction are not a concern for the site.

During a strong seismic event, seismically induced settlement can occur within loose to moderately dense, dry or saturated granular soil. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement. Based on in-situ densities (dense sands or medium stiff to stiff fine-grained soils), and soil types, dry sand settlement and induced surface manifestations are not considered an issue at the site.

2.7.7 Tsunamis and Seiches

Due to the elevation of the site with respect to sea level and its distance from large open bodies of water (i.e. the San Diego Bay located approximately 0.7 miles to the west), the potential of seiches and/or tsunami is considered to be low (CEMA, 2009 and Chula Vista, 2015).

2.8 *Slope Stability*

Given the generally flat configuration of the proposed site, and the relatively flat bedding within the underlying soils, the site/site slopes are considered stable.

2.9 *Laboratory Testing*

Laboratory testing of the onsite soils was performed as a part of LGC's investigation on representative samples obtained from the borings, and included moisture and density tests, maximum density and optimum moisture content, consolidation testing, remolded shear testing, sieve and hydrometer analyses, Atterberg limit, expansion index, and corrosion suite testing. Laboratory testing was performed by EGLAB, Inc. (EGL) and LGC Valley, Inc. LGC has reviewed the laboratory test data, procedures, and results 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.

Based on the results of our laboratory testing, the near-surface soils were found to have an expansion index of 37 (low expansion potential); however, medium expansive soils are also anticipated to be encountered onsite. Corrosion test results indicate a soluble sulfate content of 0.016 percent, a minimum resistivity of 670 ohm-centimeters, a pH of 7.77, and a chloride content of 180 ppm. The soils should be considered corrosive to buried metal based on the resistivity results. The corrosive effects on concrete are considered negligible (per ACI 318R-11 Table 4.3.1). These results/assumptions should be confirmed at the completion of site grading.

Consolidation testing was performed on the near-surface in-situ soil samples. Results indicate that the upper 5 to 7 feet of material are subject to consolidation and therefore we recommend remedial removals of the top 5 to 7 feet of material across the site, while some areas within the zones of artificial fill are expected to be deeper. See Figure 2- Geotechnical Map for the anticipated removal depths across the site. During grading, the remedial removal depth should be verified in the field by a representative from this office.

2.10 *Excavation Characteristics*

It is anticipated that the on-site materials can be excavated with conventional heavy-duty construction equipment.

3.0 CONCLUSIONS

Based on the results of our geotechnical investigation, evaluation, and review, it is our professional 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:

- Based on our field exploration, localized areas of the site are underlain by undocumented fill soils. Undocumented fill should be anticipated in the areas of the existing buildings, existing utilities, and the location of the old drainage channels. We estimate these fills range in depth across the site between 1 to approximately 12 feet, with the deeper areas in the locations of the old drainage channels. The onsite fill soils consist of clay, silty clay, and sandy clay. These soils should be considered compressible and completely removed during remedial grading operations.

- Based on our observations during the current subsurface field investigation, and our review of readily available geologic maps of the area (Appendix A), the site is underlain by Quaternary-aged alluvial flood-plain deposits. These deposits were encountered from ground surface in some borings to the total depth explored of 51 feet, bgs. The onsite alluvial soils consist of clay, silty to sandy clay, and clayey sand.

- The alluvial materials are considered suitable to support the proposed structures with the exception of the upper 3 to 5 feet across the site, which laboratory testing indicates, is subject to consolidation. The potentially compressible alluvial soils must be removed to competent alluvium as determined during future grading by the geotechnical consultant. Removals in limited areas may be deeper based on field observations during grading.

- From a geotechnical perspective, the existing onsite soils are suitable for use as fill, provided they are relatively free from rocks greater than 8-inches in diameter, construction debris, and organic material.

- The ALTA survey indicates a number of underground utilities exist across the site. Active utility lines must be relocated, being careful to locate them outside of the zone of influence of any proposed structures and improvements prior to beginning any grading operations. The existing utility line trench backfill and utility lines should be completely removed or abandoned in place.

- If the box culvert and storm drain system located within the easement on the southwest corner of the site, will not be relocated, the site configuration shown on the Conceptual Site Plans (WHA, 2018) must be revised to avoid having buildings on top of, or within the structural zone of influence of these drainage structures.

- Groundwater was encountered in the geotechnical borings at an elevation of approximately mean sea level, or 30 to 32.5 feet below existing ground surface. Therefore, we do not anticipate encountering groundwater during grading.

- Active or potentially active faults are not known to exist on the site.

- The site is not located within an Alquist-Priolo Earthquake Fault Zone (CGS, 2018). The closest Holocene-active fault is a splay of the Rose Canyon fault zone which is located approximately 3.8 miles (6.1 kilometers) west of the site.

- The main seismic hazard that may affect the site is ground shaking from one of the active regional faults.

- Based on our analysis, due to the fine-grained nature and/or relatively dense nature of the on-site soils, the potential for liquefaction and dynamic settlement of the site is negligible and not a concern for the site.

- The expansion potential testing indicates that onsite soils have a low expansion potential, but given the predominance of clayey soils on the site, we recommend that the foundations be designed for soils with low to medium expansive potentials.

- We recommend that any import materials should be limited to very low to low expansion potential.

- Laboratory test results indicate that the onsite soils should be considered corrosive to buried metal, based on the resistivity results. The corrosive effects on concrete are considered negligible. LGC is not a corrosion engineer, and recommends that upon completion of grading, you enlist the expertise of a corrosion engineer to sample the soils at finish grade and design measures to mitigate potential effects of the onsite corrosive soils.

- The proposed building structure may be designed to be supported by a conventional, post-tension, or mat slab foundation system designed to account for the low or medium expansion potential and anticipated static settlements.

4.0 RECOMMENDATIONS

4.1 Site Earthwork

Site preparation will include demolition of the existing buildings, and removal of onsite trash and surface pavement. We anticipate that earthwork at the site will consist of removal and realignment of the onsite sewer and storm drain lines, with the exception of the box culvert in the center of the site, which we anticipate will remain in place. Earthwork will also include remedial removals of undocumented fill below the existing buildings and in the areas of the old drainage channel, as well as removing the top compressible layers of the young alluvial flood-plain deposits. Site grading, will include construction of slab-on-grade type foundations, installation of utilities, and placement of the driveways, parking spaces, and concrete flatwork around the proposed building.

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

4.1.1 Site Preparation

Prior to grading of the area to receive structural fill, the ground surface should be cleared of trash and stripped of vegetation. The debris should be removed and properly disposed of offsite. 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 to 12-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 Recompaction

As discussed in Section 2.2, the proposed site is underlain by relatively shallow layers of unsuitable soils, which may settle under the addition of water, surcharge of fill and/or foundation loads. Therefore, compressible materials, within areas planned to support the proposed building structure, should be excavated to competent material and replaced with compacted fill soils.

We anticipate removals to be on the order of 5 to 12 feet below existing ground surface along the old drainage channels, and approximately 5 to 7 feet across the remainder of the site; 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 6 to 12 inches, brought to at least optimum-moisture content, and recompacted. The fill prism beneath the building footings should extend downward at a 1:1 (horizontal to vertical) slope from the outside edge of the footing bottoms. The removals should extend a minimum of 5 feet beyond the building perimeters. In general, the intent of the remedial removals is to remove all undocumented fills, and unsuitable alluvium.

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 recompacted prior to additional fill placement or construction.

4.1.3 <u>Shrinkage/Bulking</u>

Based on the site soils, bulking is not anticipated at the site. The preliminary estimated shrinkage factors of 5 to 15 percent for the upper site soils (undocumented fills 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 <u>Temporary Stability of Removal Excavations</u>

Due to the recommended depth of remedial removals below existing grades (approximately 5 to 12 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 Fill Placement and Compaction

From a geotechnical perspective, the on-site soils are suitable for use as compacted fill, provided they are screened of rocks greater than 8 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 recompacted to at least 90 percent relative compaction (based on ASTM Test Method D1557). 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.

If possible, import soils to be used as fill shall be: 1) essentially free from organic matter and other deleterious substances; 2) should contain no materials over 6 inches in maximum dimension; 3) have a very low to low expansion potential (i.e. an Expansion Index ranging from 0 to 50); and 4) possess a negligible sulfate content. Representative samples of the desired import source shall be given to the geotechnical consultant at least 48 hours (2 working days) before importing grading begins so that its suitability can be determined, and appropriate tests performed.

4.1.6 Trench Backfill and Compaction

The on-site 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 Foundation Selection

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 three foundations recommended for the proposed structure are: 1) conventional for very low to low expansion potentials; 2) post-tension foundation; or 3) mat slab.

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 ***Bearing Capacity***

Shallow foundations may be designed for a maximum allowable bearing capacity of 2,000 lb/ft^2 (gross), for continuous footings a minimum of 12 inches wide and 12 inches deep, and spread footings 24 inches wide and 12 inches deep, into certified compacted fill. A factor of safety greater than 3 was used in evaluating the above bearing capacity value. This value maybe increased by 300 psf for each additional foot in depth and 150 psf for each additional foot of width to a maximum value of 3,000 psf.

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/ft^2/ft. The passive earth pressure incorporates a factor of safety of greater than 1.5.

All footing excavations should be cut square and level as much as possible, and should be free of sloughed materials including sand, rocks and gravel, and trash debris. Subgrade soils should be pre-moistened for the assumed low expansion potential (to be confirmed at the end of grading). These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only.

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.

4.2.3 ***Conventional Foundations/Slabs***

The proposed three story structures underlain by very low to low expansive soils may be designed for shallow conventional foundations. Proposed continuous footings should be a minimum of 15 inches wide and embedded a minimum of 24 inches deep, and spread footings a minimum of 24 inches wide and embedded a minimum of 24 inches deep, into certified compacted fill. The proposed footings should be designed per the bearing capacity recommendations provided in Section 4.2.1. A preliminary effective plasticity index of 15, for very low to low expansive soils may be used in the foundation design.

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 $lb/ft^2/ft$. The passive earth pressure incorporates a factor of safety of greater than 1.5. The passive resistance value may be increased by one-third when considering loads of short duration such as wind or seismic loads.

All footing excavations should be cut square and level as much as possible, and should be free of sloughed materials including sand, rocks and gravel, and trash debris. Subgrade soils should be pre-moistened for the assumed very low to 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 very low to low expansion potential. The minimum thickness of the floor slabs should be at least 4.5 inches, and joints should be provided per usual practice.

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.

4.2.4 Post-Tension Foundations

Based on the site geotechnical conditions (i.e. expansion potential and static and seismically induced settlements) and provided the remedial recommendations provided herein are implemented, the site may be considered suitable for the support of the anticipated structures using a post-tensioned slab-on-grade foundation system, for the anticipated low to medium expansive soils. The following section summaries our recommendations for the foundation system. The parameters provided below are for the expansion potential only and do not include the estimated static settlements.

Table 2 contains the geotechnical recommendations for the construction of PT slab on grade foundations. The structural engineer should design the foundation system based on these parameters including the foundation settlement as indicated in the following section to the allowable deflection criteria determined by the structural engineer/architect.

Table 2

Preliminary Geotechnical Parameters for Post-Tensioned Foundation Design

Parameter	Value	
Expansion Classification (Assumed to be confirmed at the completion of grading):	Low and Medium Expansion	
Thornthwaite Moisture Index (From Figure 3.3):	-20	
Constant Soil Suction (From Figure 3.4):	PF 3.6	
Center Lift Edge moisture variation distance (from Figure 3.6), e_m: Center lift, y_m:	**Low** 9.0 feet 0.3 inches	**Medium** 9.0 feet 0.50 inches
Edge Lift Edge moisture variation distance (from Figure 3.6), e_m: Edge lift, y_m:	**Low** 5.1 feet 0.61 inches	**Medium** 5.1 feet 1.1 inches
Soluble Sulfate Content for Design of Concrete Mix in Contact with Site Soils in Accordance with American Concrete Institute standard 318, Section 4.3:	Negligible Exposure	
Corrosivity of Earth Materials to Ferrous Metals:	Corrosive	
Modulus of Subgrade Reaction, k (assuming presaturation as indicated below):	100 pci (low) 85 pci (medium)	

Additional Recommendations:

1. Presaturate slab subgrade to at least 1.2 times optimum moisture, to minimum depths of 12 and 18 inches below ground surface, respectively for low and medium expansion potentials.
2. Install a 15-mil moisture/vapor barrier (or equivalent) moisture/vapor barrier in direct contact with the concrete (unless superseded by the Structural/Post-tension engineer*) with 1 to 2 inches of sand below the moisture/vapor barrier.
3. Minimum perimeter foundation embedment below finish grade for moisture cut off should be 12 and 18 inches, respectively for low and medium expansion potentials.
4. Minimum slab thickness should be 5 inches.

* The above sand and moisture/vapor barrier recommendations are traditionally included with geotechnical foundation recommendations although they are generally not a major factor influencing the geotechnical performance of the foundation. The sand and moisture/vapor barrier requirements are the purview of the foundation engineer/corrosion engineer (in accordance with ACI Publication 302 "Guide for Concrete Floor and Slab Construction") and the homebuilder to ensure that the concrete cures more evenly than it would otherwise, is protected from corrosive environments, and moisture penetration of through the floor is acceptable to future homeowners. Therefore, the recommendations provided herein may be superseded by the requirements of the previously mentioned parties.

As indicated above, the underslab vapor/moisture retarder (i.e. an equivalent capillary break method) may consist of a minimum 15-mil vapor barrier in conformance with ASTM E 1745 Class A material, placed in general conformance with ASTM E1643, 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 concrete mix design, which will address bleeding, shrinkage, and curling are the purview of the foundation engineer, in consideration of the project requirements provided by the architect and developer. The underslab vapor/moisture retarder described above is considered a suitable alternative in accordance with the Capillary Break Section 4.505.2.1 of the CALGreen code.

4.2.4 ***Mat Foundations***

A mat foundation can be used for support of the proposed building. An allowable soil bearing pressure of 1,000 psf may be used for the design of the mat at the surface under the slab area.

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 85 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. Based on assumed structural loads, we estimate that total static settlement will be on the order of an inch at the center of the mat foundation. Post construction differential settlement can be taken as one-half of the maximum estimated settlement

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. Foundations may be designed for a coefficient of friction of 0.35. Minimum perimeter footing embedment provided in the previous sections maybe reduced for the mat slab design.

Coordination with the structural engineer will be required in order to ensure structural loads are adequately distributed throughout the mat foundation to avoid localized stress concentrations resulting in potential settlement. The foundation plan should be reviewed by LGC to confirm preliminary estimated total and differential static settlements.

4.2.5 ***Foundation Settlement***

Based on our current understanding of the project, the results of our site investigation and the recommended remedial grading with shallow foundations embedded into compacted fills, we estimate the post-construction static settlement of the site to be up to 1-inch with a differential settlement of approximately of 0.5-inches in 30 feet. The settlement values provided herein are with the assumption that the proposed structures at this site will consist of lightly loaded structures. If larger/heavier loaded structures are designed , the settlement values provided in this section should be reevaluated by LGC.

4.3 Lateral Earth Pressures for Retaining Walls

The following lateral earth pressures may be used for the design of any future site retaining walls. Due to the variable nature of the onsite soils, we recommend site retaining walls be backfilled with select soils or clean sand having a sand equivalence of greater than 30. Select soils should consist of clean, granular soils (less than 15 percent passing the No. 200 sieve) of very low to low expansion potential (expansion index 30 or less based on UBC. 18-2). The recommended lateral pressures for clean sand or approved select soils for level or sloping backfill are presented in Table 3.

Table 3

Lateral Earth Pressures for Retaining Walls

Conditions	**Equivalent Fluid Weight (pcf)**		
	Level Backfill	**2:1 Backfill Sloping Upwards**	**Seismic Earth Pressure (pcf) ***
	Approved Select Material	**Approved Select Material**	
Active	35	55	15
At Rest	55	80	--
Passive	250	150 (Sloping Down)	--

* For walls with greater than 6-feet in backfill height, the above seismic earth pressure should be added to the static pressures given in the table above. The seismic earth pressure should be 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 for determining earth pressures as a result of seismic events.

Embedded structural walls should be designed for lateral earth pressures exerted on them. The magnitude of these pressures depends on the amount of deformation that the wall can yield under load. If the wall can yield enough 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 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.

For design purposes, the recommended equivalent fluid pressure for each case for walls founded above the static groundwater and backfilled with very low to low expansive onsite or import soils is provided in the table above. The equivalent fluid pressure values assume free-draining conditions. The backfill soils should be compacted to at least 90 percent relative compaction. The walls should be constructed and backfilled as soon as possible after back-cut excavation. Prolonged exposure of back-cut slopes may result in some localized slope instability. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer.

Surcharge loading effects from any adjacent structures should be evaluated by the geotechnical and structural engineers. Surcharge loading on retaining walls should be considered when any loads are located within a 1:1 (horizontal to vertical) projection from the base of the retaining wall and should be added to the applicable lateral earth pressures. Where applicable, a minimum uniform lateral pressure of 100 psf should be added to the appropriate lateral earth pressures to account for typical vehicle traffic loading.

All retaining wall structures should be provided with appropriate drainage and appropriately waterproofed. The outlet pipe should be sloped to drain to a suitable outlet. It should be noted that the subdrain located behind the wall does not provide protection against seepage through the face of the wall and/or efflorescence. Efflorescence is generally a white crystalline powder (discoloration) that results when water, which contains soluble salts, migrates over a period of time through the face of a retaining wall and evaporates. If such seepage or efflorescence is undesirable, retaining walls should be waterproofed to reduce this potential.

For sliding resistance, the friction coefficient of 0.35 may be used at the concrete and soil interface. Wall footings should be designed in accordance with structural considerations. The passive resistance value may be increased by one-third when considering loads of short duration such as wind or seismic loads. For short term loading (i.e. seismic) the allowable bearing capacity may be increased by one-third for seismic loading.

Foundations for retaining walls in properly compacted fill should be embedded at least 18 inches below lowest adjacent grade. At this depth and a minimum of 12 inches in width, an allowable bearing capacity of 2,000 psf may be assumed. A factor of safety greater than 3 was used in evaluating the above bearing capacity value. This value maybe increased by 300 psf for each additional foot in depth and 150 psf for each additional foot of width to a maximum value of 3,000 psf. All excavations should be made in accordance with Cal OSHA. Excavation safety is the sole responsibility of the contractor.

4.4 ***Preliminary Pavement Design Recommendations***

Based on an R-value of 10, we recommend the following preliminary minimum pavement sections for Traffic Indices of 5 and 6 (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				
Location	**Traffic Index**	**Design R-Value**	**Asphalt Concrete Thickness (inches)**	**Aggregate Base Thickness (inches)**
Parking Spaces	5	10	3.0	9.0
Alleys/Driveways	6	10	3.5	12.0

The aggregate base material should conform to the specifications for Class 2 Aggregate Base (Caltrans) or Crushed Aggregate Base (Standard Specifications for Public Works Construction). The base material should be compacted to achieve a minimum relative compaction of 95 percent. The subgrade should achieve a minimum relative compaction of 90 percent through the upper 12 inches. Base and subgrade materials should be moisture-conditioned to relatively uniform moisture content at or slightly over optimum.

Portland Cement Concrete Pavement (PCCP) may be designed using a minimum of 8-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 10-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.

The following recommendations are for vehicular concrete pavers designed for vehicular traffic and are underlain by 1-inch of sand. Based on ASCE 58-10 for interlocking pavers, considering a Traffic Index (TI) of 6.0 and an R-value of 10 for the subgrade soils, we recommend the following minimum base section underlying the proposed pavers. The proposed pavers and sand should be underlain by a minimum 12-inches of crushed aggregate base. The aggregate base material should conform to the specifications for Crushed Aggregate Base (Standard Specifications for Public Works Construction) and be place and compacted thin lifts. The base material should be compacted to achieve a minimum relative compaction of 95 percent. The subgrade should achieve a minimum relative compaction of 90 percent through the upper 12 inches. Base and subgrade materials should be moisture-conditioned to a relatively uniform moisture content near optimum moisture.

4.5 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-11 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.

Laboratory testing of representative on-site soils indicated that most the site soils tested are classified as having a negligible sulfate exposure condition in accordance with ACI 318R-11 Table 4.3.1. As a preliminary recommendation, concrete in contact with onsite soils should be designed in accordance with ACI 318R-11 Table 4.3.1 for the negligible category. It is also our opinion that onsite soils should be considered corrosive to buried metals. Site grading will redistribute the materials, which may result in soils with different corrosion potentials. Therefore, the as-graded soil conditions should be verified with confirmatory sampling and testing during the grading phase of the project.

Despite the minimum recommendation above, LGC is not a corrosion-engineering firm. Therefore, we recommend that after site grading, consultation with a competent corrosion engineer be initiated to evaluate the actual corrosion potential of the site and to provide recommendations to reduce the corrosion potential with respect to the proposed improvements, as necessary. The recommendations of the corrosion engineer may supersede the above requirements.

4.6 Nonstructural Concrete Flatwork

Concrete flatwork has 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 dynamics as foundation elements. To reduce the potential for excessive cracking and lifting, concrete should be designed in accordance with the minimum guidelines outlined in Table 5. 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 (inches)	4
Presaturation	Presoak to 12 inches
Reinforcement	No. 3 at 24 inches on centers
Maximum Joint Spacing	5 feet or per usual practice

4.7 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 the building. The 2016 California Building Code, Section 1804.3 states that the ground immediately adjacent to the foundation should be sloped a minimum of 5-percent away from the building for a minimum distance of 10 feet, and should further be diverted into a swale with a slope of at least 2-percent. If there is an impervious surface immediately adjacent to the foundation, the slope may be reduced to a 2-percent gradient. However, based on site soils, positive drainage may be accomplished by providing drainage away from the building 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.

4.8 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.

Geotechnical observation and testing should be performed by the geotechnical consultant during site excavations, subgrade for slab/foundation, backfill of utility trenches, preparation of any subgrade and placement of aggregate base, or when any unusual soil conditions are 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.

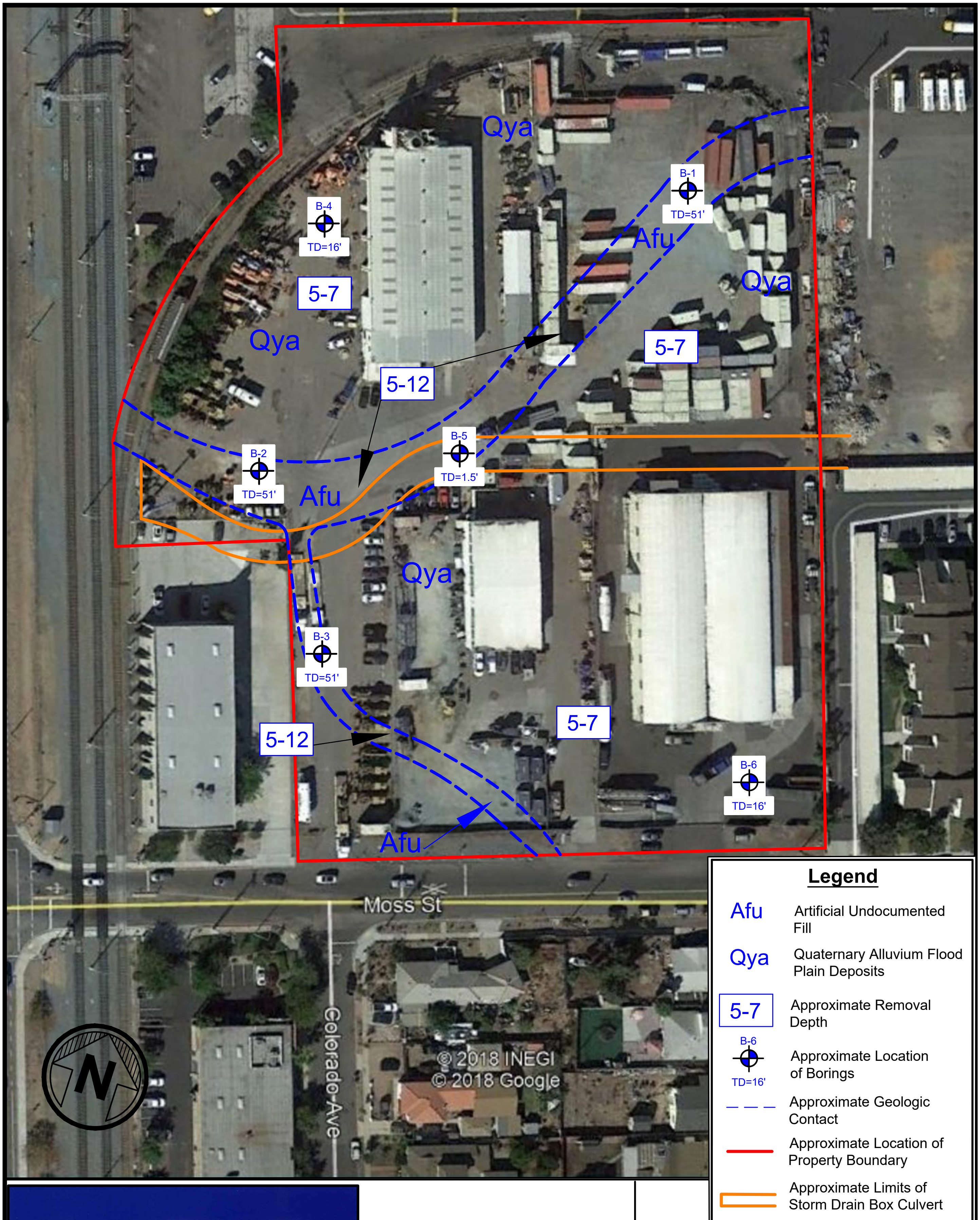


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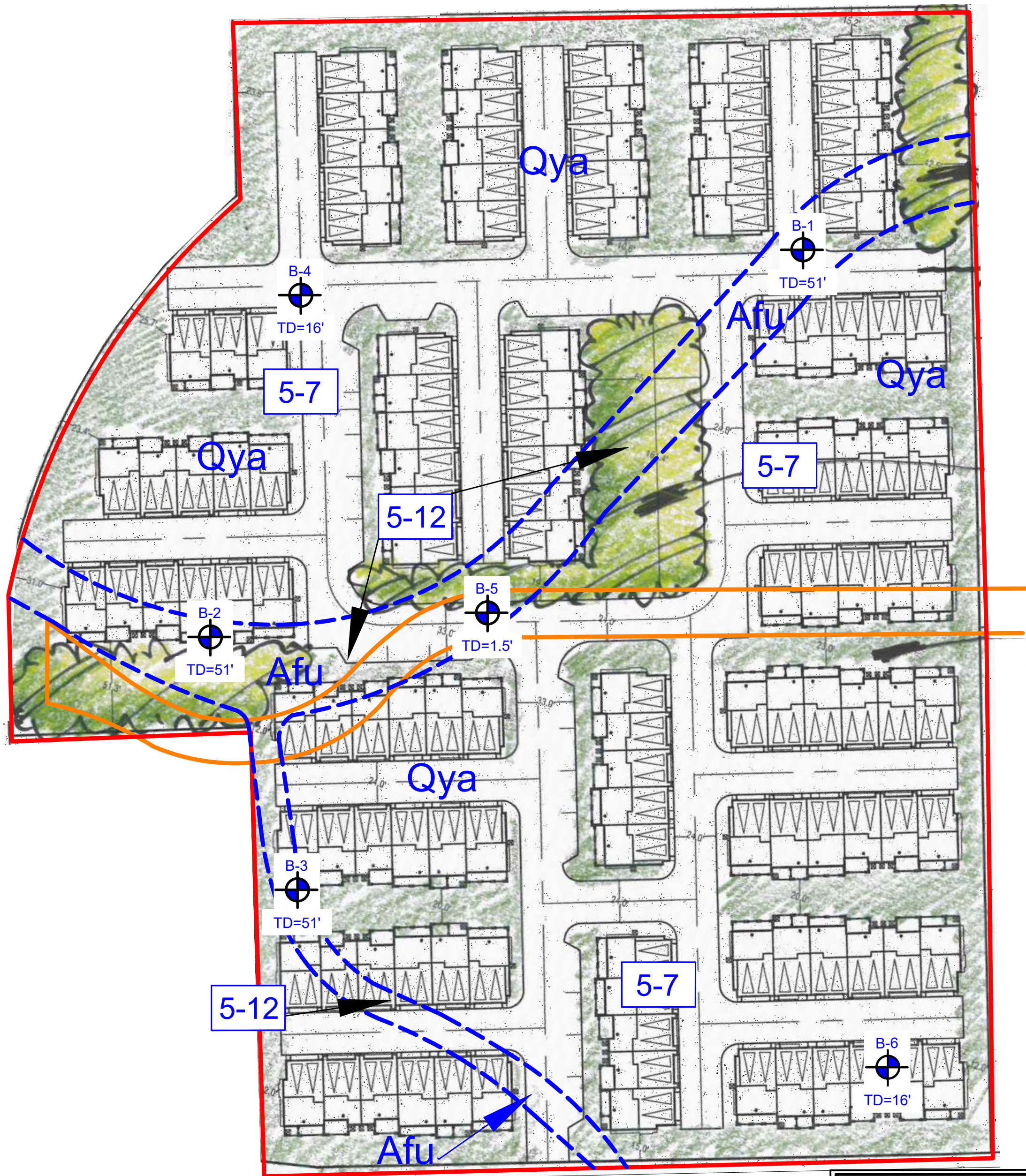
Figure 2
Geotechnical Map
676 Moss Street
Chula Vista, California

Project Name	Shopoff/Moss Street
Project Number	184013-00
Eng./Geo.	BIH/RKW
Date	June 13, 2018
Scale	1"=200'



Legend

Afu	Artificial Undocumented Fill
Qya	Quaternary Alluvium Flood Plain Deposits
5-7	Approximate Removal Depth
B-6 TD=16'	Approximate Location of Borings
	Approximate Geologic Contact
	Approximate Location of Property Boundary
	Approximate Limits of Storm Drain Box Culvert

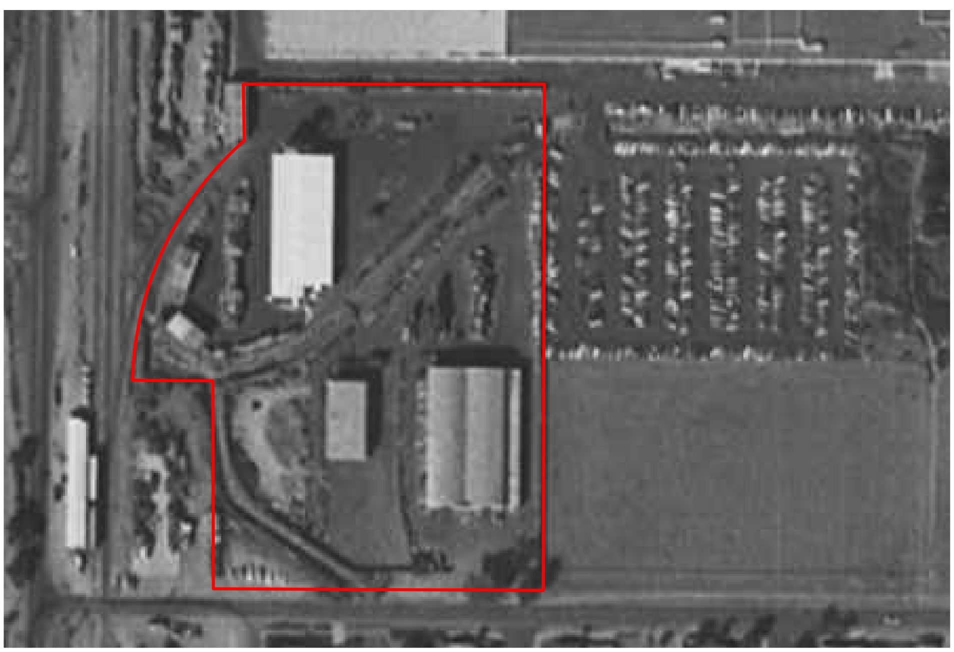
WHA, 2018, Conceptual Site Plan - Idea 1

LGC

Figure 3
Proposed Plan
676 Moss Street
Chula Vista, California

Project Name	Shopoff/Moss Street
Project Number	184013-00
Eng./Geo.	BIH/RKW
Date	June 13, 2018
Scale	1"=200'

1953 Aerial Photo



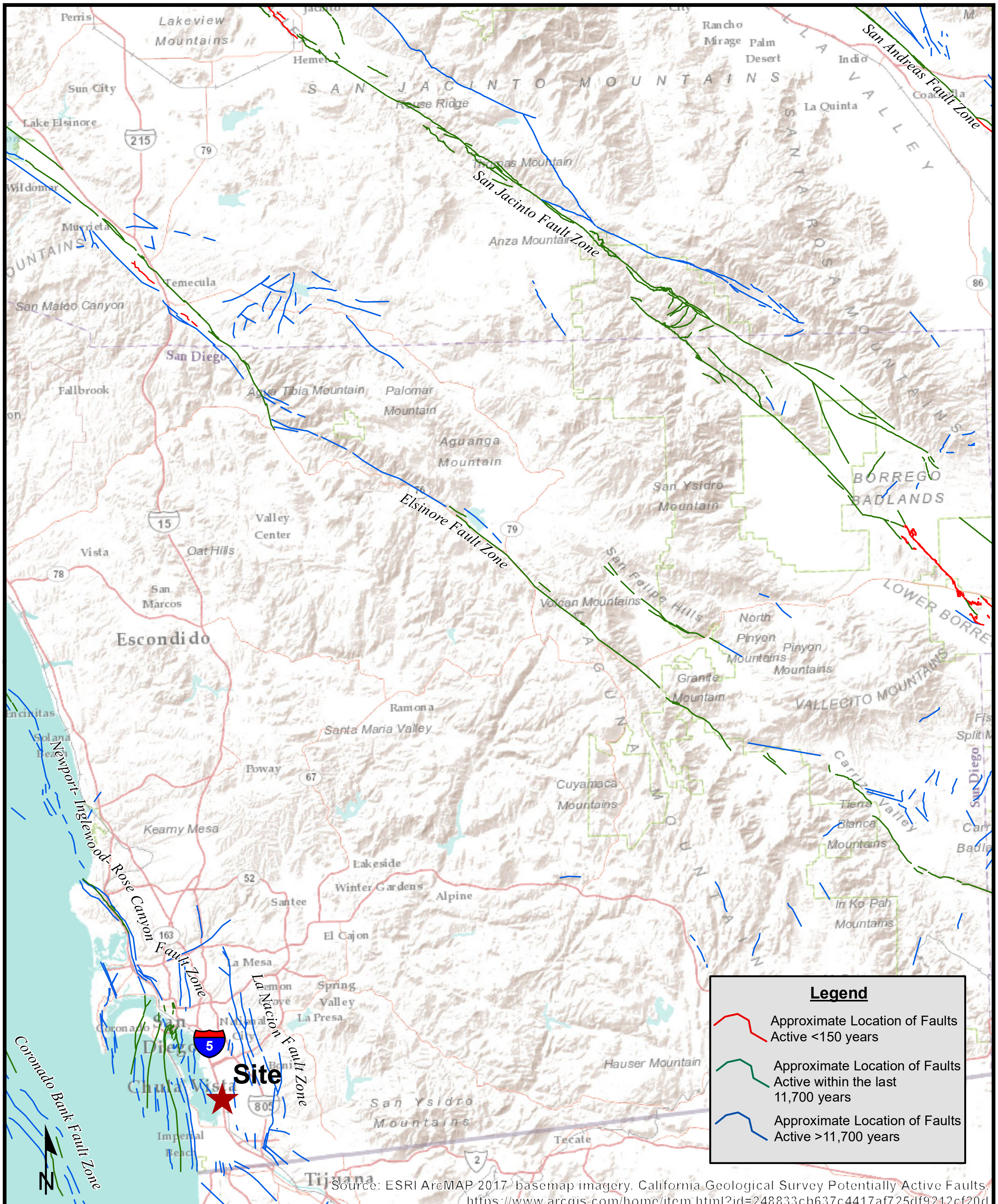
1975 Aerial Photo

LGC

Figure 4

Historical Site Photos
676 Moss Street
Chula Vista, California

Project Name	Moss Street
Project Number	184013-00
Eng./Geo.	BIH/RKW
Date	June 13, 2018
Client	Shopoff



LGC

Figure 5
Regional Fault Map
676 Moss Street
Chula Vista, California

Project Name	Shopoff/Moss Street
Project Number	184013-00
Eng./Geo.	BIH/RKW
Date	June 13, 2018

0 4.5 9 Miles

APPENDIX A

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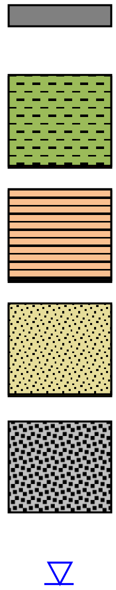
Aerial Photographs Review			
Date	**Source**	**Flight**	**Photo No(s)**
1953-03-31	UCSB Library	AXN-3M	79 and 80
1953-05-02	UCSB Library	AXN-14M	108, 109 and 110
1958-03-04	UCSB Library	C23023	12-75 and 12-76
1964-04-07	UCSB Library	SD5	168
1975-06-30	UCSB Library	TG-7500	1A-5

APPENDIX B

Geotechnical Boring Logs



ASPHALT CONCRETE

CLAY

SILT

SAND

GRAVEL/COBBLES

APPROXIMATE GROUNDWATER LEVEL

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

Geotechnical Boring Log LGC B-1

Date:June 21, 2018	Page: 1 of 2
Project Name: 676 Moss St	Project Number: 184013-00
Drilling Company: Baja Exploration	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 32 Feet (mean sea level)	Hole Location: See Map

N:32.613957 E:-117.087642

Logged By: LF
Sampled By: LF

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
32	0						CL	**Artifical Fill - Undocumented (Afu):** @0' silty CLAY; dark brown, moist, stiff	
			1	16			SC	@2.5' clayey silty fine SAND; dark brown, moist, loose	
			A						
27	5		2	29	106.0	12.8		@5' becomes medium dense	EI, COR, RDS, MAX
			3	16				@7.5' loose, 1" rounded gravel present	SHA, CN
22	10		4	23	118.0	9.2		@10' medium dense, no gravel	-200
			5	24			CL	**Quaternary Young Alluvial Flood-Plain Deposits (Qya):** @12.5' fine sandy CLAY; dark red brown, moist, very stiff	
17	15		6	62				@15' becomes hard	
			7	37	112.7	15.8		@17.5' silty CLAY; medium orange brown, moist, very stiff	
12	20		8	84				@20' fine sandy CLAY; medium orange brown, moist, hard	SHA
			9	42	115.3	8.3			
7	25		10	43	112.1	18.0	SC	@25'clayey SAND; medium orange brown, moist, medium dense	
			11	14					CN
2	30								

LGC

■ = Ring sample
☒ = SPT sample
BULK = Bulk sample

LGC VALLEY, INC.
THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED

Geotechnical Boring Log LGC B-1

Date: June 21, 2018	Page: 2 of 2
Project Name: 676 Moss St	Project Number: 184013-00
Drilling Company: Baja Exploration	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 32 Feet (mean sea level)	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION N:32.613957 E:-117.087642 Logged By: LF Sampled By: LF	Type of Test
2	30		12	29	110.7	20.5	SC	**Quaternary Young Alluvial Flood-Plain Deposits (Qya) (cont.):** @30' silty to clayey fine SAND; medium orange/red brown, wet, medium dense @32.5' encountered groundwater	
			13	28					SHA
-3	35		14	36	106.3	22.2	CL	@35' silty CLAY; medium orange brown, saturated, very stiff	
			15	16					AL
-8	40		16	53	111.0	19.7	SM	@40' silty fine SAND; dark brown, saturated, dense	-200
			17	32				@42.5' becomes fine to medium grained, dense	SHA
-13	45		18	50	109.3	18.0		@45' clayey fine SAND; light gray brown, saturated, medium dense	
			19	40			ML	@47.5' fine sandy SILT; light gray brown, saturated, hard	SHA
-18	50		20	61					
								Total Depth = 51 Feet **Groundwater Encountered @32.5 Feet** **Backfilled with Bentonite on June 21, 2018**	
-23	55								
-28	60								

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BULK = Bulk sample

LGC VALLEY, INC.
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Geotechnical Boring Log LGC B-2

Date: June 21, 2018	Page: 1 of 2
Project Name: 676 Moss St	Project Number: 184013-00
Drilling Company: Baja Exploration	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 31 Feet (mean sea level)	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION N:32.613098 E:-117.088567 Logged By: LF Sampled By: LF	Type of Test
31	0							@0' 3" of asphalt underlain by 8" of aggregate base material	
							CL	**Artifical Fill - Undocumented (Afu):**	
			1					@1' silty CLAY; dark brown, moist, medium stiff @2.5' no recovery of sample	
26	5		2	13					CN
			3	49	119.3	14.4	CL	**Quaternary Young Alluvial Flood-Plain Deposits (Qya):** @7.5' fine to medium sandy CLAY; medium orange brown, moist, medium hard	SHA
21	10		4	50				@10' fine sandy CLAY; dark brown, moist, hard	AL
			5	55	121.8	11.0	SC	@12.5' clayey SAND; light brown and medium gray, moist, dense	
16	15		6	54				@15' contains stringers of white calcium carbonate	
			7	54	103.5	19.9			
11	20		8	53			CL	@20' fine sandy CLAY; dark brown, moist, hard; 2" gravel encountered	SHA
			9	55	121.3	13.2		@22.5' sandy CLAY; dark brown, moist, hard	
6	25		10	32				@25' fine sandy CLAY; dark orange brown, moist, very stiff	
			11	56	121.0	14.6	SC	@27.5' clayey fine to medium SAND; dark orange brown, wet, dense	
1	30								

LGC

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Geotechnical Boring Log LGC B-2

Date: June 21, 2018	Page: 2 of 2
Project Name: 676 Moss St	Project Number: 184013-00
Drilling Company: Baja Exploration	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 31 Feet (mean sea level)	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION N:32.613098 E:-117.088567 Logged By: LF Sampled By: LF	Type of Test
1	30		12	41			SM	**Quaternary Young Alluvial Flood-Plain Deposits (Qya) (cont.):** @30' silty fine to medium SAND; dark orange brown, wet, dense	
			13	53	119.2	16.2		@32.5' encountered groundwater; 3" cobble encountered	SHA
-4	35		14	51			SM- CL	@35' silty fine to medium SAND and fine to medium sandy CLAY; dark orange brown, saturated, very dense/hard	
			15	67	115.6	13.0	CL	@37.5 fine to coarse sandy CLAY; dark orange brown, saturated, hard	
-9	40		16	29					
			17	63	113.0	15.8	SM	@42.5'silty fine to medium SAND; dark orange brown, saturated, dense	
-14	45		18	56					SHA
			19	59	108.4	16.4	CL	@47.5' fine sandy CLAY; dark gray, saturated, hard	
-19	50		20	42					
								Total Depth = 51 Feet **Groundwater Encountered @32.5 Feet** **Backfilled with Bentonite on June 21, 2018**	
-24	55								
-29	60								

LGC

■ = Ring sample
☒ = SPT sample
BULK = Bulk sample

LGC VALLEY, INC.
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Geotechnical Boring Log LGC B-3

Date: June 21, 2018	Page: 1 of 2
Project Name: 676 Moss St	Project Number: 184013-00
Drilling Company: Baja Exploration	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 30 Feet (mean sea level)	Hole Location: See Map

DESCRIPTION

N:32.612726 E:-117.088216

Logged By: LF

Sampled By: LF

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	Description	Type of Test
30	0							@0' 3" of asphalt underlain by 6" of aggregate base material	
			A				CL	**Artifical Fill - Undocumented (Afu):**	
			1	14	105.5	17.5		@1' silty CLAY; dark brown, moist, stiff	
25	5		2	26			CL	@5' sandy silty CLAY; dark brown, moist, very stiff	-200
			3	33	117.2	13			
20	10		4	24					CN
			5	30	118.6	10	SM	**Quaternary Young Alluvial Flood-Plain Deposits (Qya):**	SHA
15	15		6	25				@12.5' silty fine SAND; dark red brown, moist, medium dense	
			7	44	114.8	17.4		@17.5' becomes medium to coarse grained, some gravel found	
10	20		8	61			CL	@20' silty CLAY; dark red brown, moist, hard	
			9	30	113.7	18.1		@22.5' becomes very stiff	
5	25		10	43			SC	@25' fine to medium clayey SAND; dark red brown, wet, dense	
0	30	▽							

LGC

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BULK = Bulk sample

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Geotechnical Boring Log LGC B-3

Date: June 21, 2018	Page: 2 of 2
Project Name: 676 Moss St	Project Number: 184013-00
Drilling Company: Baja Exploration	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 30 Feet (mean sea level)	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION N:32.612726 E:-117.088216 Logged By: LF Sampled By: LF	Type of Test
0	30		11	43			SW	**Quaternary Young Alluvial Flood-Plain Deposits (Qya)(cont.):** @30' fine to coarse SAND; light brown, saturated, dense; groundwater encountered	
-5	35		12	64			SM	@35'silty SAND; light brown, very dense; contains scattered coarse gravels	SHA
-10	40		13	56			SC	@40' clayey fine SAND; light brown, very dense	
-15	45		14	58				@45' dark gray to brown, saturated, very dense	
-20	50		15	55			CL	@50' fine sandy CLAY; medium gray to light brown, saturated, very stiff	
-25	55							**Total Depth = 51 Feet** **Groundwater Encountered @30 Feet** **Backfilled with Bentonite on June 21, 2018**	
-30	60								

LGC

■ = Ring sample
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BULK = Bulk sample

LGC VALLEY, INC.
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Geotechnical Boring Log LGC B-4

Date: June 21, 2018	Page: 1 of 1
Project Name: 676 Moss St	Project Number: 184013-00
Drilling Company: Baja Exploration	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 33 Feet (mean sea level)	Hole Location: See Map

DESCRIPTION

N:32.613657 E:-117.088544

Logged By: LF

Sampled By: LF

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
33	0							@0' 6" of asphalt underlain by 3" of aggregate base material	
							CL	**Quaternary Young Alluvial Flood-Plain Deposits (Qya):**	
			1	26	105.4	20.6		@0.9' silty CLAY; dark brown, moist, very stiff	
28	5		2	31				@5' sandy silty CLAY; dark brown, moist, very stiff	
23	10		3	55	113	10.3	SC	@10' clayey fine to medium SAND; medium orange brown, moist, dense	
18	15		4	55				@15' clayey fine SAND; light gray and very pale gray, moist, dense	
13	20							**Total Depth = 16 Feet** **No Groundwater Encountered** **Backfilled with Bentonite on June 21, 2018**	
8	25								
3	30								

LGC

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BULK = Bulk sample

LGC VALLEY, INC.
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Geotechnical Boring Log LGC B-5

Date: June 21, 2018	Page: 1 of 1
Project Name: 676 Moss St	Project Number: 184013-00
Drilling Company: Baja Exploration	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 33 Feet (mean sea level)	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION N:32.613262 E:-117.088034 Logged By: LF Sampled By: LF	Type of Test
33	0							@0' 3" of asphalt underlain by 8" of aggregate base material	
								Artifical Fill - Undocumented (Afu): @1' silty CLAY; dark brown, moist @1.5' Hit top of box culvert	
28	5							**Total Depth = 1.5 Feet** **No Groundwater Encountered** **Backfilled with Native Soil on June 21, 2018**	
23	10								
18	15								
13	20								
8	25								
3	30								

LGC

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Geotechnical Boring Log LGC B-6

Date: June 21, 2018	Page: 1 of 1
Project Name: 676 Moss St	Project Number: 184013-00
Drilling Company: Baja Exploration	Type of Rig: Hollow Stem Auger
Drive Weight: 140 lbs.	Drop: 30" Hole Dia: 8"
Elevation of Top of Hole: 34 Feet (mean sea level)	Hole Location: See Map

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION N:32.612774 E:-117.087016 Logged By: LF Sampled By: LF	Type of Test
34	0							@0' 3" of asphalt underlain by 10" of aggregate base material	
							CL	**Quaternary Young Alluvial Flood-Plain Deposits (Qya):**	
			1	21				@1.1' silty CLAY ; dark redish brown, moist, stiff	
29	5		2	71	120.2	13.8	CL-SC	@5' silty CLAY and clayey SAND; dark brown, moist, hard/dense	
24	10		3	56			CL	@10' silty CLAY; dark brown, moist, hard	
19	15		4	53	123.5	11.9	SC	@15'clayey SAND; light brown to gray, moist, dense	
14	20								
9	25								
4	30								

Total Depth = 16 Feet
No Groundwater Encountered
Backfilled with Bentonite on June 21, 2018

LGC

■ = Ring sample
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BULK = Bulk sample

LGC VALLEY, INC.
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APPENDIX C

Laboratory Testing Procedures and Test Results

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 with respect to the subject site, concurs with, and accepts responsibility as geotechnical engineer of record for their work (laboratory testing).

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 excavation logs.

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 and presented on the following table:

Sample Location	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
B-1 #15 @37.5'	29	16	13
B-2 #4 @10'	43	16	27

Chloride Content: Chloride content was tested in accordance with CTM 422. The results are presented below:

Sample Location	Sample Description	Chloride Content (ppm)	Potential Degree of Chloride Attack*
B-1 #A @5'- 6'	Dark brown fine sandy silty CLAY	180	Negligible

* Extrapolation from California Test Method 532, Method for Estimating the Time to Corrosion of Reinforced Concrete Substructures and previous experience.

Laboratory Testing Procedures and Test Results (cont.)

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). Where an appreciable amount of fines were encountered (greater than 20 percent passing the No. 200 sieve) a hydrometer analysis was done to determine the distribution of soil particles passing the No. 200 sieve. The sieve and hydrometer curves are presented on the attached figures at the end of this appendix. The percent passing the #200 sieve is presented on the following table:

Sample Location	Sample Description	Percent Passing #200 Sieve
B-1 #3 @7.5’	Dark brown clayey fine SAND	43
B-1 #8 @20’	Medium orange brown fine sandy clayey SILT	65
B-1 #10 @25’	Medium orange brown clayey SAND	36
B-1 #13 @32.5’	Medium orange-red brown silty to clayey fine SAND	48
B-1 #16 @40’	Dark brown silty fine SAND	15
B-1 #17 @42.5’	Dark brown silty fine to medium grained SAND	12
B-1 #19 @47.5’	Light gray brown fine sandy SILT	61
B-2 #3 @7.5’	Medium orange brown fine to medium sandy CLAY	60
B-2 #8 @20’	Light brown and medium gray clayey SAND to sandy CLAY	50
B-2 #13 @32.5’	Dark orange brown, clayey fine to medium SAND	21
B-2 #18 @45’	Dark orange brown silty fine to medium SAND	27
B-3 #2 @5’	Dark brown silty CLAY	70
B-3 #5 @12.5’	Dark red brown silty fine SAND	32
B-3 #12 @35’	Light brown silty fine to coarse SAND	11

Laboratory Testing Procedures and Test Results (cont.)

Consolidation: Consolidation tests were performed on selected, relatively undisturbed ring samples (per 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. The consolidation pressure curves are presented on the attached figures at the end of this appendix.

Direct Shear (Remolded or Undisturbed): Direct shear tests were performed 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). The test results are presented on the following table and/or on the attached figures at the end of this appendix.

Sample Location	Sample Description	Peak/Ultimate Friction Angle (degrees)	Peak/Ultimate Apparent Cohesion (psf)
B-1 #A @5'-6'	Dark brown fine sandy silty CLAY (Remolded)	216/72	30/30

Expansion Index Tests: The expansion potential of selected materials was evaluated by the Expansion Index Test, UBC Standard No. 18-I-B 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. The results of these tests are presented in the table below:

Sample Location	Sample Description	Expansion Index	Expansion Potential
B-1 #A @5'-6'	Dark brown fine sandy silty CLAY	37	Low

Laboratory Testing Procedures and Test Results (cont.)

Maximum Dry Density Tests: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM Test Method D1557. The results of these tests are presented in the table below:

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-1 #A @5'-6'	Dark brown fine sandy silty CLAY	129.0	9.0

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. The results of these tests are presented on the boring logs. Where applicable, only moisture content was determined from undisturbed or disturbed samples.

Minimum Resistivity and pH Tests: 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 results of soil's resistivity decreases corrosivity increases. The results are presented in the table below:

Sample Location	Sample Description	pH	Minimum Resistivity (ohms-cm)	Potential Degree of Corrosivity*
B-1 #A @5'-6'	Dark brown fine sandy silty CLAY	7.77	670	Corrosive

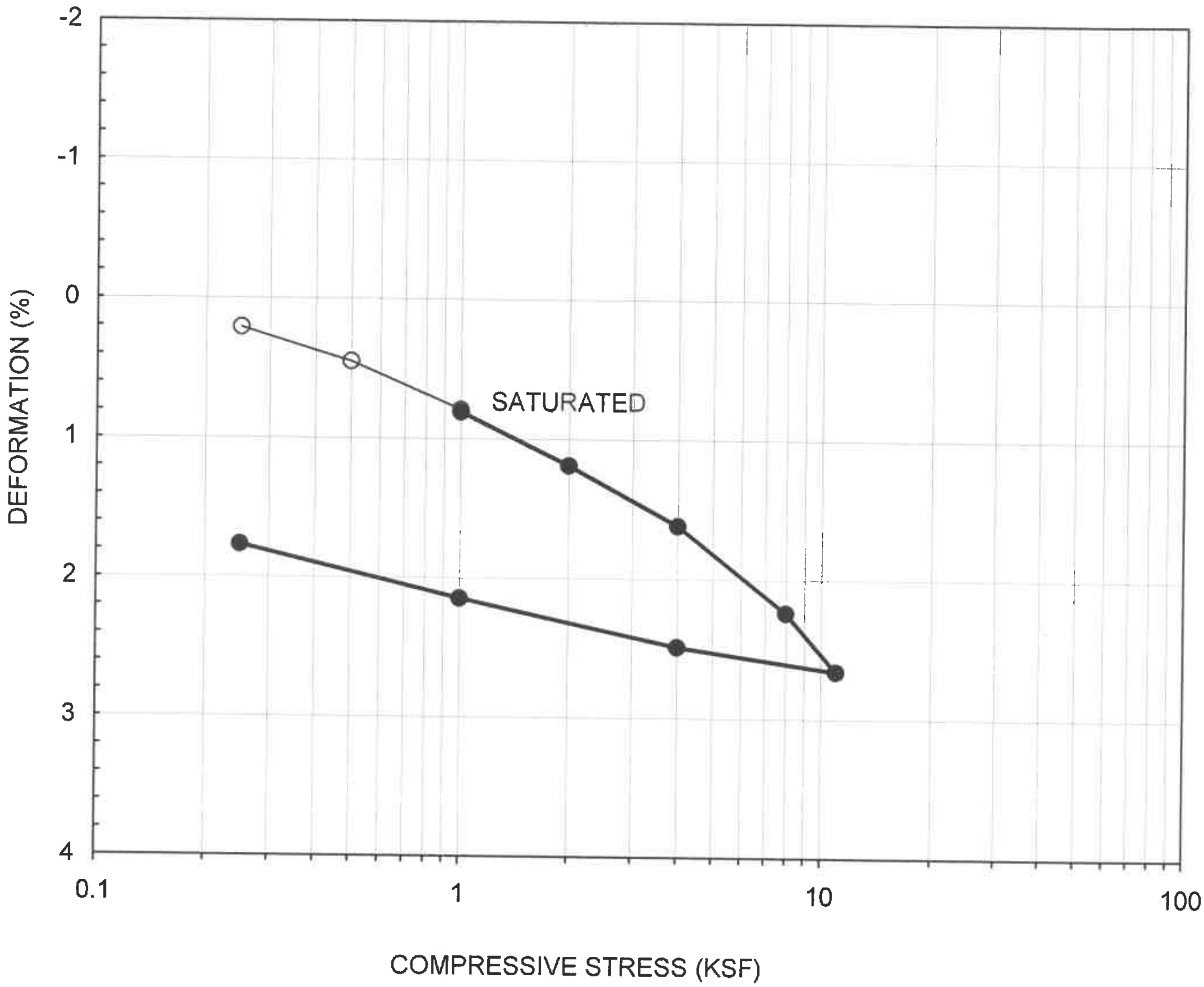
* NACE Corrosion Basics

Laboratory Testing Procedures and Test Results (cont.)

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods (CTM417). 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:

Sample Location	**Sample Description**	**Sulfate Content (% by weight)**	**Potential Degree of Sulfate Attack***
B-1 #A @5'-6'	Dark brown fine sandy silty CLAY	0.016	Negligible

* Per ACI 318R-08 Table 4.3.1 (ACI, 2008).



Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-1	3	7.5	SC	11.1	116.3	0.449

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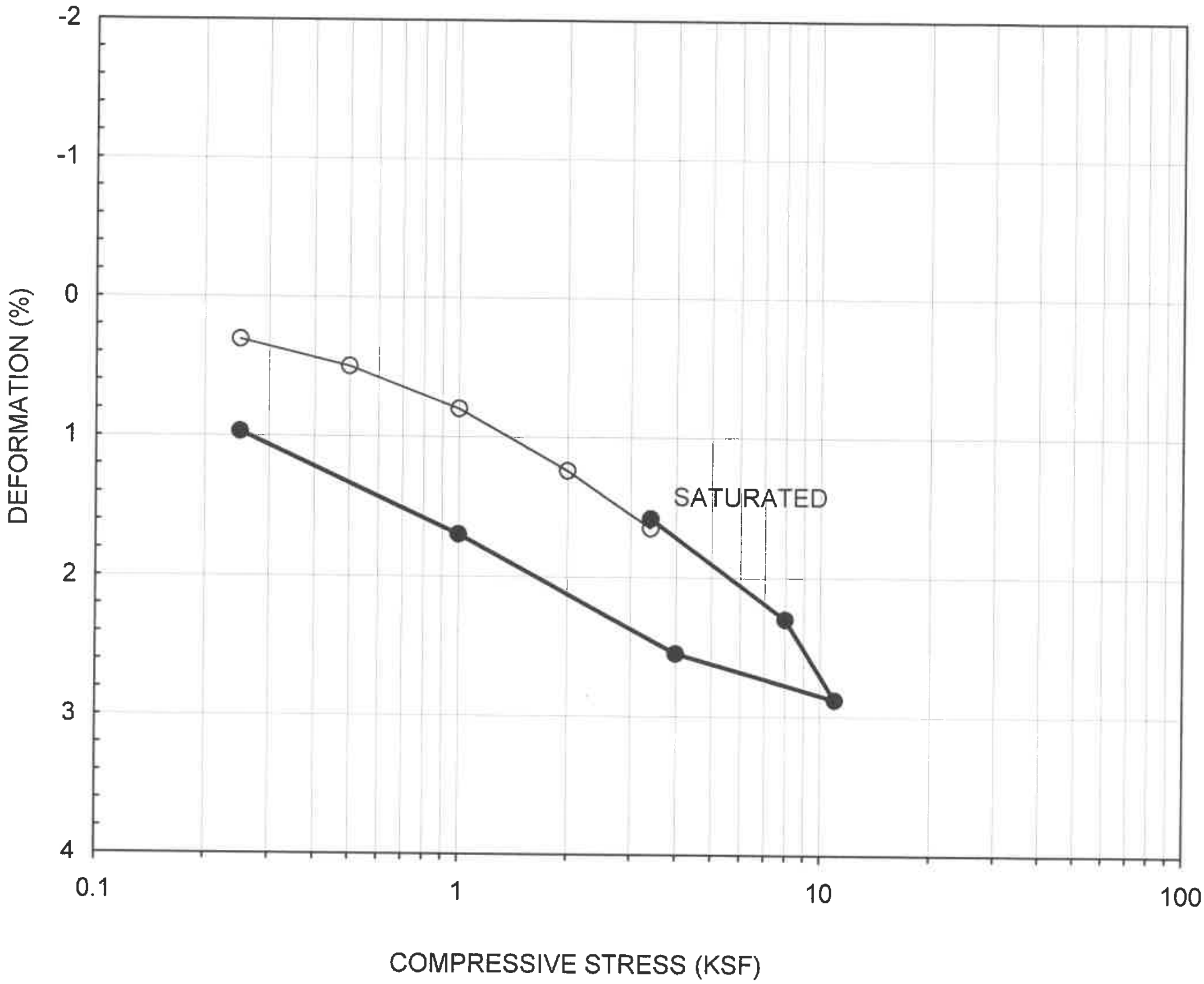
Figure



Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-1	10	25.0	CL	18.0	109.7	0.536

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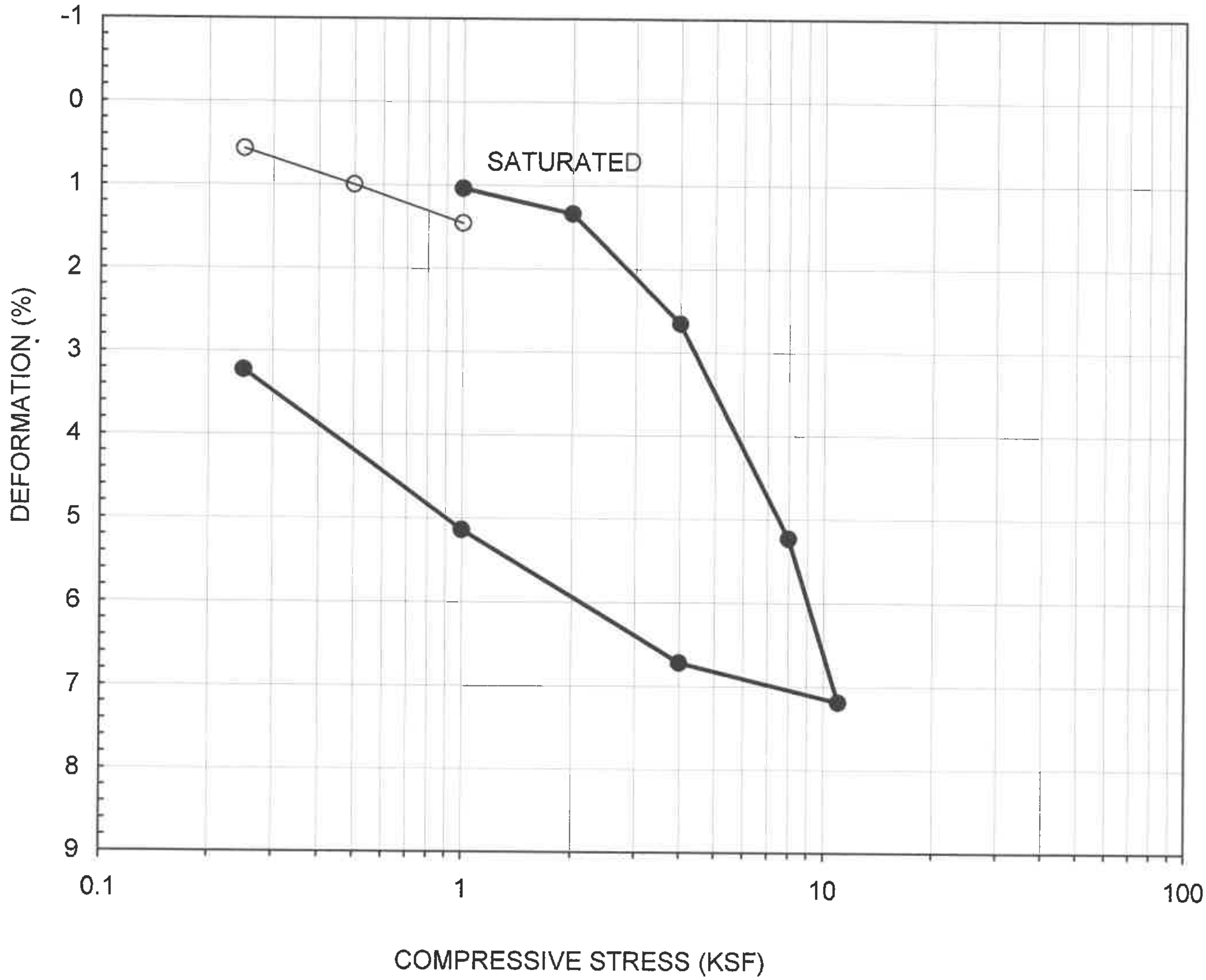

Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-2	2	5.0	CL	33.5	83.7	1.012

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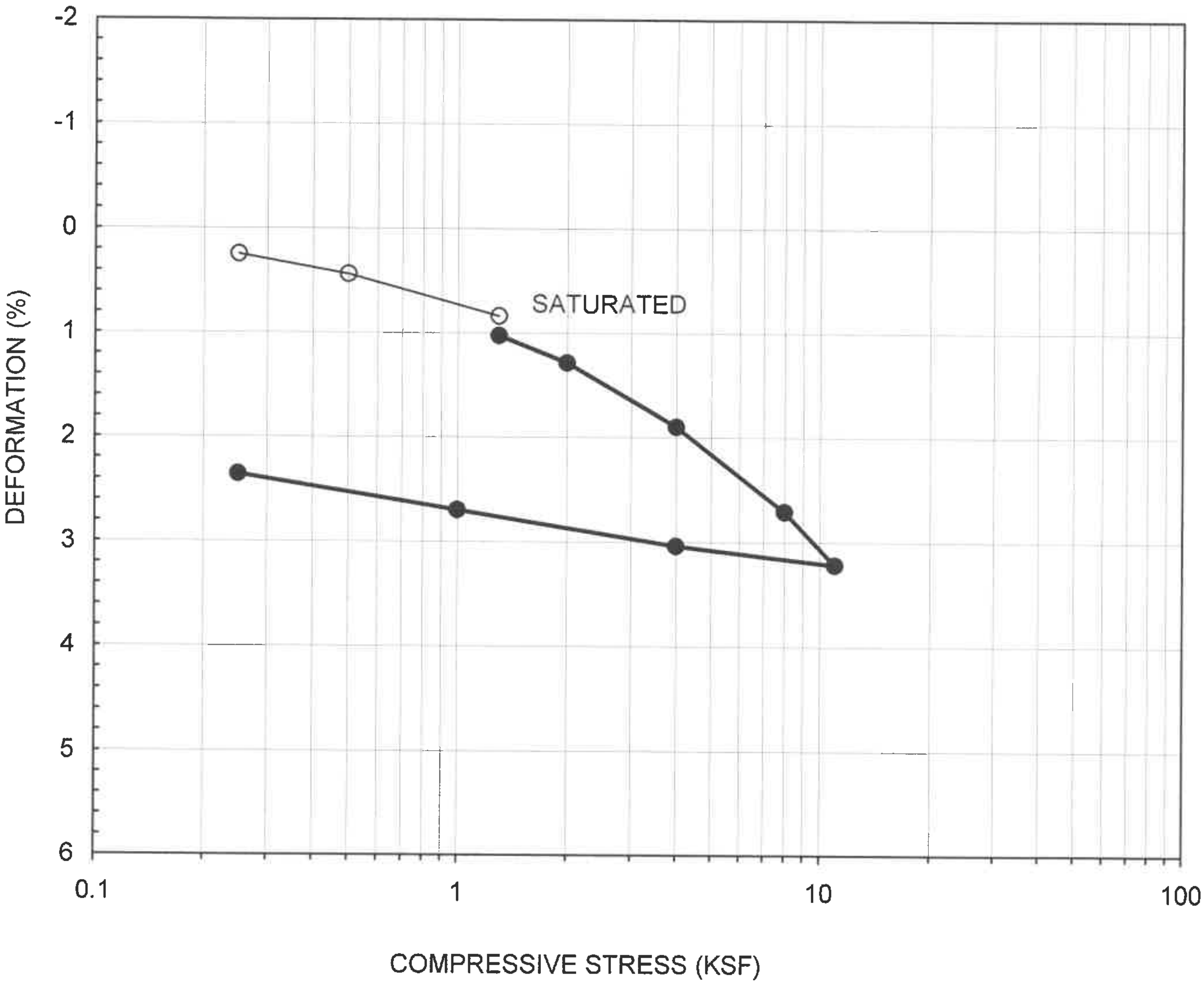

Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density (PCF)	Init. Void Ratio
○	B-3	4	10.0	SC-SM	10.2	111.2	0.515

EGLAB, INC.	Project Name: 676 Moss Street Client: LGC Valley, Inc. Job No.: 184013-00 EGLAB Project No.: 18-059-017

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07/18 (ASTM D2435) Figure

APPENDIX D

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 sheeps-foot 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.