June 20, 2022 Project No. 22070-01

Mr. Jeff Alvarez
EPD Solutions, Inc.
2355 Main Street, Suite 100
Irvine, CA 92614

Subject: ***Preliminary Geotechnical Evaluation, Proposed Industrial Development, 2771 West 205th Street, Torrance, California***

In accordance with your request, LGC Geotechnical, Inc. has performed a geotechnical evaluation for the proposed industrial development located at 2771 West 205th Street, in the City of Torrance, California. This report summarizes the results of our background review, subsurface exploration, and geotechnical analyses of the data collected, and presents our findings, conclusions, and preliminary recommendations for the proposed development.

If you should have any questions regarding this report, please do not hesitate to contact our office. We appreciate this opportunity to be of service.

Respectfully,

LGC Geotechnical, Inc.

REGISTERED PROFESSIONAL ENGINEER
RYAN L. DOUGLAS
No. 3147
GEOTECHNICAL
STATE OF CALIFORNIA

Ryan Douglas, PE, GE 3147
Project Engineer

RLD/BPP/klr

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This report presents the results of our geotechnical evaluation for the proposed industrial development located at 2271 West 205th Street in the City of Torrance, California. (see Site Location Map, Figure 1). The purpose of our work was to collect subsurface data in order to prepare a geotechnical report providing preliminary recommendations for design and construction of the proposed project. Our scope of services included:

- Review of pertinent readily available geotechnical information and geologic maps (Appendix A).
- Subsurface investigation including excavation, sampling, and logging of 7 small-diameter hollow stem borings.
- Performed 3 infiltration tests within the hollow stem borings.
- Laboratory testing of representative samples obtained during our subsurface investigation (Appendix C).
- Geotechnical analysis and evaluation of the data obtained.
- Preparation of this report presenting our preliminary findings, conclusions and recommendations with respect to the proposed site development.

1.2 Background and Project Description

The approximately 6.0-acre site is bound to the north by residential developments, to the south by West 205th Street and to the east and west by existing office buildings. Review of historic aerial photographs suggests the following:

<u>1952 through 1980 Aerial Photos</u>: At this time, the subject site contained a parking lot and undeveloped land.

<u>1985 Aerial Photos</u>: Construction of the current office building had begun with 5 out of the 6 buildings being complete.

<u>1991 through 2018 Aerial Photos:</u> By 1991, all 6 buildings had been built throughout the site. The site has remained essentially the same since this time except for some minor landscape improvements.

Based on the preliminary conceptual site plan (RGA, 2021), one approximately 126,000 square foot industrial warehouse structure with on grade parking areas is proposed. The proposed industrial building is anticipated to be a concrete tilt-up structure with estimated maximum column and wall loads of approximately 150 kips and 10 kips per linear foot, respectively. Please note no structural loads or preliminary grading plans were provided to us at the time of this report.

The recommendations provided herein are based upon the estimated structural loading and layout information above. We understand that the project plans are currently being developed at this time; LGC Geotechnical should be provided with updated project plans and any changes to the assumed structural loads when they become available, in order to either confirm or modify the recommendations provided herein. Additional field work and/or laboratory testing may be necessary.

1.3 Subsurface Evaluation

LGC Geotechnical performed a recent subsurface geotechnical evaluation of the site consisting of the excavation of seven hollow-stem auger borings (three of which were used for infiltration testing).

The four hollow-stem borings (HS-1 through HS-4) and three hollow-stem borings used for infiltration testing (I-1 through I-4) were drilled to a depth ranging from approximately 15 to 50 feet below existing grade and approximately 10 to 15 feet below existing grade, respectively. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. The borings were excavated using a truck-mounted drill rig equipped with an 8-inch-diameter hollow-stem auger. Driven soil samples were collected by means of the Standard Penetration Test (SPT) and Modified California Drive (MCD) sampler generally obtained at 2.5 to 5-foot vertical increments. The MCD is a split-barrel sampler with a tapered cutting tip and lined with a series of 1-inch-tall brass rings. The SPT sampler and MCD sampler were driven using a 140-pound automatic hammer falling 30 inches to advance the sampler a total depth of 18 inches. The raw blow counts for each 6-inch increment of penetration were recorded on the boring logs. Bulk samples were also collected and logged at select depths for laboratory testing. At the completion of drilling, the borings were backfilled and tamped. Some settlement of the backfill soils may occur over time.

Infiltration testing was performed within three of the borings (I-1 through I-3) at a depths ranging from approximately 10 to 15 feet below existing grade, per the direction of the civil engineer. An LGC Geotechnical staff engineer installed standpipes, backfilled the boring annulus with crushed rock, and pre-soaked the infiltration wells prior to testing. Infiltration testing was performed in accordance with the County of Los Angeles testing guidelines. The infiltration test wells were subsequently backfilled with native soils and tapped at the completion of testing. Some settlement of the backfill soils may occur over time.

The approximate locations of borings are shown on the Boring Location Map (Figure 2). Boring logs are presented in Appendix B.

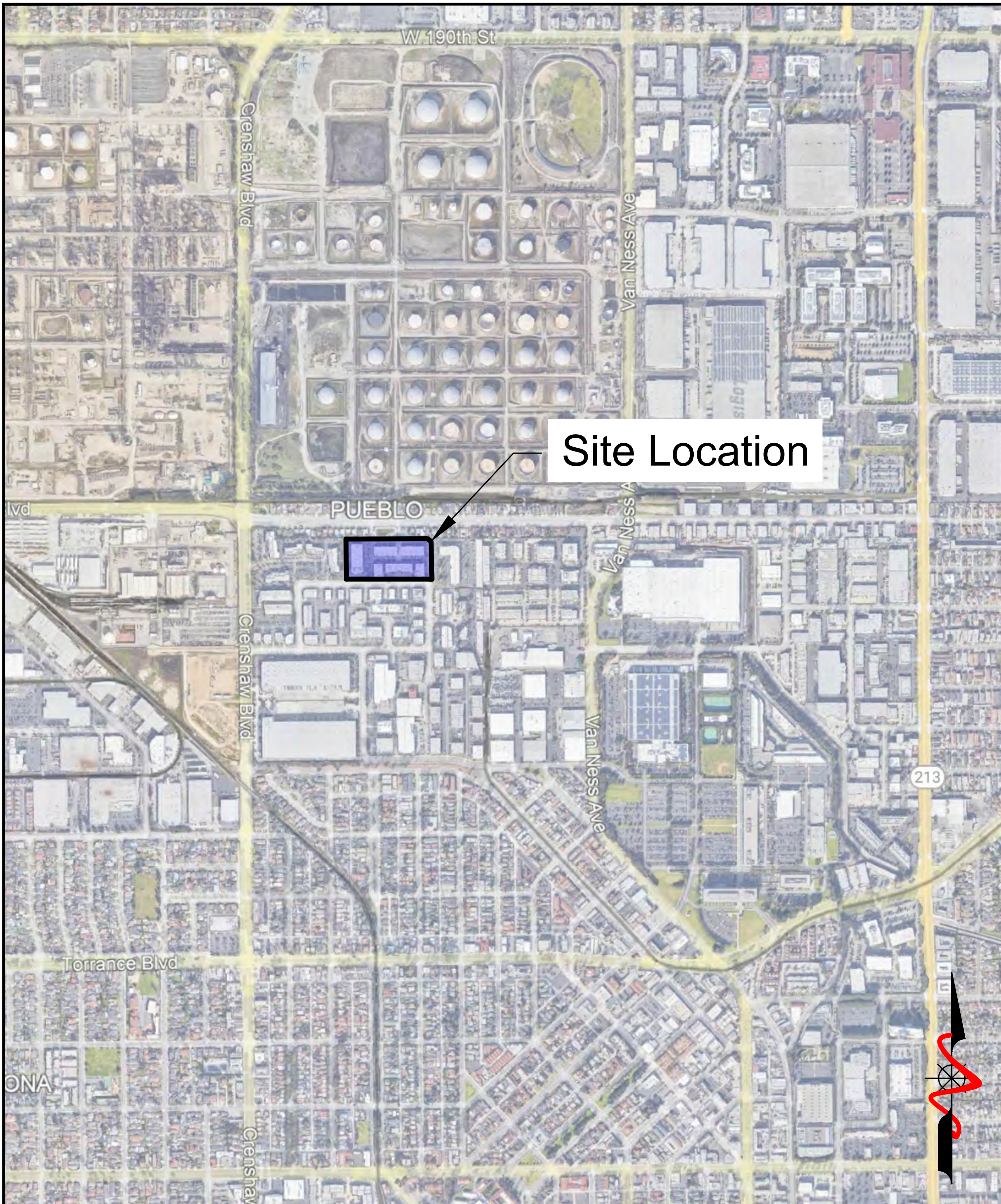
1.4 Laboratory Testing

Laboratory testing was performed on representative soil samples obtained from our subsurface evaluation. Laboratory testing included in-situ moisture and density tests, fines content, Atterberg Limits, expansion index, maximum density, direct shear, consolidation and corrosion (sulfate, chloride content, pH, and minimum resistivity).

The following is a summary of the recent laboratory test results.

- Dry density of the samples collected ranged from approximately 83 pounds per cubic foot (pcf) to 118 pcf, with an average of approximately 104 pcf. Field moisture contents ranged from approximately 3 percent to 38 percent, with an average of 14 percent.
- Five samples tested for fines content indicated a fines content (passing No. 200 sieve) of approximately 13 percent to 50 percent. According to the Unified Soils Classification System (USCS), the tested samples are classified as “coarse-grained” soil.
- One Atterberg Limit (liquid limit and plastic limit) test was performed. Results indicate a Plasticity Index value of 18. The plot is provided in Appendix C.
- One direct shear test was performed. The plot is provided in Appendix C.
- One consolidation test was performed. The stress vs. deformation plot is provided in Appendix C.
- One Expansion Index (EI) tests was performed. Results were an EI value of 37, corresponding to “Low” expansion potential.
- Laboratory compaction of a near-surface bulk sample resulted in a maximum dry density of 121.0 pcf at an optimum moisture content of 11.0 percent.
- Corrosion testing indicated soluble sulfate contents less than approximately 0.01 percent, chloride content of 185 parts per million (ppm), pH value of 8.13, and minimum resistivity value of 1,490 ohm-cm.

A summary of the results is presented in Appendix C. The moisture and dry density test results are presented on the boring logs in Appendix B.



LGC Geotechnical, Inc.

FIGURE 1
Site Location Map

PROJECT NAME	EPD - Torrance
PROJECT NO.	22070-01
ENG. / GEOL.	RLD
SCALE	Not to Scale
DATE	June 2022

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

The subject site is generally located within the Peninsular Ranges Geomorphic Province, specifically within the coastal plain that forms the gently sloping flatlands to the north of the uplifted Palos Verdes Peninsula. The coastal plain consists of Quaternary older alluvium interpreted to be middle to late Pleistocene in age (Saucedo et al, 2016).

No known faults cross the site, and the only complex regional geologic feature near the site is an inferred anticline, as shown on the regional geologic map to pass about two miles to the south (Saucedo et al, 2016). The Newport Inglewood right lateral strike slip fault passes more than 5 miles east of the site.

2.2 Site Geology and Generalized Subsurface Conditions

Based on review of available geologic maps (Saucedo et al, 2016), the primary geologic unit underlying the site is Quaternary old alluvium. As encountered at the subject site, native alluvial soils generally consisted of medium dense to very dense sands and silty sands and stiff to hard silts and clays below the recommended removal and recompaction bottoms to the maximum explored depth of approximately 50 feet below existing grade (see Appendix B for Boring Logs). For the purposes of this report, the thin veneer of artificial fill present across the site has not been differentiated on the boring logs.

It should be noted that borings are only representative of the location and time where/when they are performed, and varying subsurface conditions may exist outside of the performed location. In addition, subsurface conditions can change over time. The soil descriptions provided above should not be construed to mean that the subsurface profile is uniform, and that soil is homogeneous within the project area. For details on the stratigraphy at the exploration locations, refer to Appendix B.

2.3 Groundwater

Groundwater was not encountered to the maximum explored depth of approximately 50 feet below existing grade during this recent evaluation. Historic high groundwater is mapped at greater than approximately 50 feet below current grade based on the seismic hazard zone report for the Ontario quadrangle (CDMG, 1998). Groundwater levels recorded nearby the subject site by the California Department of Water Resources were measured at depths approximately 85 feet below the ground surface (CDWR, 1999).

In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present within the near-surface deposits due to local seepage or during rainy seasons. Groundwater conditions below the site may be variable, depending on numerous factors including seasonal rainfall, local irrigation and groundwater pumping, among others.

2.4 *Field Infiltration Testing*

Three shallow infiltration tests were performed in Borings I-1 through I-3 ranging from depths of approximately 10 to 15 feet below existing grade. The approximate locations are shown on the Boring Location Map (Figure 2). The borings for the infiltration tests were excavated using a drill rig equipped with an 8-inch diameter hollow-stem auger. Estimation of infiltration rates was accomplished in general accordance with the guidelines set forth by the County of Los Angeles (2021). A 3-inch diameter perforated PVC pipe was placed in the borehole above a thin layer of gravel and the annulus was backfilled with gravel. The infiltration wells were pre-soaked 1 hour prior to testing. Initially the procedure for 30-minute reading intervals was followed for the borings (I-1 through I-3). During the 30-minute period, water remained in the boring after 30 minutes. Therefore, the test procedure utilizing a thirty-minute reading interval was performed. Readings were taken for a minimum of 3 hours or until a "stabilized rate" was established. A "stabilized rate" is when the highest and lowest readings are within 10 percent of each other over three consecutive readings. At the completion of infiltration testing, the pipe was removed, backfilled with cuttings, tamped, and the asphalt was patched in the necessary areas. Some settlement of the backfill should be expected.

Based on the County of Los Angeles testing guidelines, the measured infiltration is calculated by dividing the volume of water discharged by the surface area of the test section (including sidewalls plus the bottom of the boring), in a given amount of time. The measured infiltration rates are provided in Table 1 below. Please note that the values provided in Table 1 do not include reduction factors for the test procedure, site variability, and long-term siltation plugging that are required for the design infiltration rate, refer to Table 8 in Section 4.8. Infiltration tests were performed using relatively clean water free of particulates, silt, etc. Refer to the infiltration test data provided in Appendix D. Refer to Section 4.8 for infiltration recommendations.

TABLE 1

Summary of Field Infiltration Testing

Infiltration Test Location	Infiltration Test Depth (ft)	Measured Infiltration Rate* (inch/hr.)
I-1	10	0.0
I-2	15	1.1
I-3	10	0.0

*Does Not Include Required Reduction Factors, refer to Table 8, Section 4.8.

2.5 *Faulting and Seismic Hazards*

Prompted by damaging earthquakes in Northern and Southern California, State legislation and policies concerning the classification and land-use criteria associated with faults have been developed. The Alquist-Priolo Earthquake Fault Zoning Act was implemented in 1972 to prevent the construction of urban developments across the trace of active faults. California Geologic Survey Special Publication 42 was created to provide guidance for following and implementing the law requirements. Special Publication 42 was most recently revised in 2018 (CDMG, 2018). According to the State Geologist, an “active” fault is defined as one which has had surface displacement within Holocene time (roughly the last 11,700 years). Regulatory Earthquake Fault Zones have been delineated to encompass traces of known, Holocene-active faults to address hazards associated with surface fault rupture within California. Where developments for human occupation are proposed within these zones, the state requires detailed fault evaluations be performed so that engineering-geologists can identify the locations of active faults and recommend setbacks from locations of possible surface fault rupture.

The subject site is not located within a State of California Earthquake Fault Zone (i.e., Alquist-Priolo Earthquake Fault Act Zone) and no active faults are known to cross the site (CDMG, 2015). The possibility of damage due to ground rupture is considered low since no active faults are known to cross the site.

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, which may affect the site, include ground lurching, shallow ground rupture, soil liquefaction and dynamic settlement. 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. Some of the major active nearby faults that could produce these secondary effects include the Palos Verdes, Newport-Inglewood, Whittier, Compton Blind Thrust, and San Andreas Faults, among others (CGS, 2015). A discussion of these secondary effects is provided in the following sections.

2.5.1 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 coexist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. Studies indicate that loose, saturated, near-surface, cohesionless soils exhibit the highest liquefaction potential, while dry, dense, cohesionless soils, and cohesive soils exhibit low to negligible liquefaction potential. In general, cohesive soils are not considered susceptible to liquefaction. Effects of liquefaction on level ground include settlement, sand boils, and bearing capacity failures below structures. Furthermore, dynamic settlement of dry sands can occur as the sand particles tend to settle and densify as a result of a seismic event.

Based on our review of the State of California Seismic Hazard Zone for liquefaction potential (CDMG, 1999), the site is not located within a liquefaction hazard zone. Based on our field evaluation, site soils are generally not susceptible to liquefaction due to a lack of groundwater and the medium dense to very dense and fine-grained alluvium soils in the upper 50 feet; therefore, liquefaction potential is considered very low.

2.5.2 Lateral Spreading

Lateral spreading is a type of liquefaction induced ground failure associated with the lateral displacement of surficial blocks of sediment resulting from liquefaction in a subsurface layer. Once liquefaction transforms the subsurface layer into a fluid mass, gravity plus the earthquake inertial forces may cause the mass to move downslope towards a free face (such as a river channel or an embankment). Lateral spreading may cause large horizontal displacements and such movement typically damages pipelines, utilities, bridges, and structures.

Due to the depth to groundwater, very low potential for liquefaction and lack of nearby "free face" conditions, the potential for lateral spreading is considered very low.

2.6 <u>Seismic Design Criteria</u>

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2019 California Building Code (CBC) and applicable portions of ASCE 7-16 which has been adopted by the CBC. **Please note that the following seismic parameters are only applicable for code-based acceleration response spectra and are not applicable for where site-specific ground motion procedures are required by ASCE 7-16.** Representative site coordinates of latitude 33.845424 degrees north and longitude -118.325832 degrees west 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 2 on the following page. Since site soils are Site Class D, additional adjustments are required to code acceleration response spectrums as outlined below and provided in ASCE 7-16. The structural designer should contact the geotechnical consultant if structural conditions (e.g., number of stories, seismically isolated structures, etc.) require site-specific ground motions.

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.84 at a distance of approximately 8.35 km from the site would contribute the most to this ground motion (USGS, 2014).

Section 1803.5.12 of the 2019 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean (MCE_G) Peak Ground Acceleration (PGA) should be used for liquefaction potential. The PGA_M for the site is equal to 0.848g (SEAOC, 2022).

TABLE 2

Seismic Design Parameters

Selected Parameters from 2019 CBC, Section 1613 - Earthquake Loads	Seismic Design Values	Notes/Exceptions
Distance to applicable faults classifies the site as a "Near-Fault" site.		Section 11.4.1 of ASCE 7
Site Class	D*	Chapter 20 of ASCE 7
Ss (Risk-Targeted Spectral Acceleration for Short Periods)	1.767g	From SEAOC, 2022
S_1 (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.633g	From SEAOC, 2022
F_a (per Table 1613.2.3(1))	1.0	For Simplified Design Procedure of Section 12.14 of ASCE 7, F_a shall be taken as 1.4 (Section 12.14.8.1)
F_v (per Table 1613.2.3(2))	1.7	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{MS} for Site Class D [Note: $S_{MS} = F_aS_S$]	1.767g	-
S_{M1} for Site Class D [Note: $S_{M1} = F_vS_1$]	1.076g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{DS} for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	1.178g	-
S_{D1} for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.717g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
C_{RS} (Mapped Risk Coefficient at 0.2 sec)	0.899	ASCE 7 Chapter 22
C_{R1} (Mapped Risk Coefficient at 1 sec)	0.895	ASCE 7 Chapter 22
*Since site soils are Site Class D and S_1 is greater than or equal to 0.2, the seismic response coefficient Cs is determined by Eq. 12.8-2 for values of $T \leq 1.5T_s$ and taken equal to 1.5 times the value calculated in accordance with either Eq. 12.8-3 for $T_L \geq T > T_s$, or Eq. 12.8-4 for $T > T_L$. Refer to ASCE 7-16.		

2.7 Expansion Potential

Based on the results of previous laboratory testing, site soils are anticipated to have a "Low" expansion potential. Final expansion potential of site soils should be determined at the completion of grading. Results of expansion testing at finish grades will be utilized to confirm final foundation design.

3.0 CONCLUSIONS

Based on the results of our subsurface geotechnical evaluation, it is our opinion that the proposed improvements are feasible from a geotechnical standpoint, provided that the recommendations contained in the following sections are incorporated during site grading and development. A summary of our geotechnical conclusions are as follows:

- As encountered at the subject site, native alluvial soils generally consisted of medium dense to very dense sands and silty sands and stiff to hard silts and clays below the recommended removal and recompaction bottoms to the maximum explored depth of approximately 50 feet below existing grade. The near-surface loose and compressible soils are not suitable for the planned improvements in their present condition (refer to Section 4.1).
- From a geotechnical perspective, onsite soils are anticipated to be suitable for use as general compacted fill, provided they are screened of construction debris and any oversized material (8 inches in greatest dimension).
- Groundwater was not encountered in our field evaluation to a maximum explored depth of 50 feet below existing grade. Historic high groundwater is mapped at greater than approximately 50 feet below current grade based on the seismic hazard zone report for the Ontario quadrangle (CDMG, 1998). Records indicate groundwater levels recorded in the area are at depths of approximately 85 feet below existing ground surface.
- The subject study area is not located within a mapped State of California Earthquake Fault Zone (i.e., Alquist-Priolo Earthquake Fault Act Zone), and based upon our review of published geologic mapping, no known active or potentially active faults are known to exist within or in the immediate vicinity of the site. Therefore, the potential for ground rupture as a result of faulting is considered very low.
- The main seismic hazard that may affect the site is ground shaking from one of the active regional faults. The subject site will likely experience strong seismic ground shaking during its design life.
- Site soils are generally not susceptible to liquefaction due to a lack of groundwater and medium dense to very dense as well as fine-grained alluvial soils in the upper 50 feet; therefore, liquefaction potential is considered very low.
- Based on the results of preliminary laboratory testing, site soils are anticipated to have “Low” expansion potential. Final design expansion potential must be determined at the completion of grading.
- Excavations into the existing site soils should be feasible with heavy construction equipment in good working order. We anticipate that the sandy and silty earth materials generated from the excavations will be generally suitable for re-use as compacted fill, provided they are relatively free of rocks larger than 8 inches in dimension, construction debris, and significant organic material.
- On-site soils will most likely not be suitable for backfill of site retaining walls. Import soils that will be used for retaining wall backfill should be tested and approved by the geotechnical consultant prior to the backfill of site walls.
- Field testing resulted in measured infiltration rates ranging from 0.0 to 1.1 inches per hour. The measured infiltration rates do not include a factor of safety. Discussion regarding infiltration is provided in Section 4.8.

4.0 RECOMMENDATIONS

The following recommendations are to be considered preliminary and should be confirmed upon completion of grading and earthwork operations. In addition, they should be considered minimal from a geotechnical viewpoint, as there may be more restrictive requirements from the architect, structural engineer, building codes, governing agencies, or the owner.

It should be noted that the following geotechnical recommendations are intended to provide sufficient information to develop the site in general accordance with the 2019 CBC requirements. With regard to the possible occurrence of potentially catastrophic geotechnical hazards such as fault rupture, earthquake-induced landslides, liquefaction, etc. the following geotechnical recommendations should provide adequate protection for the proposed development to the extent required to reduce seismic risk to an "acceptable level." The "acceptable level" of risk is defined by the California Code of Regulations as "that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project" [Section 3721(a)]. Therefore, repair and remedial work of the proposed improvement may be required after a significant seismic event. With regards to the potential for less significant geologic hazards to the proposed development, the recommendations contained herein are intended as a reasonable protection against the potential damaging effects of geotechnical phenomena such as expansive soils, fill settlement, groundwater seepage, etc. It should be understood, however, that our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions but cannot preclude the potential for some cosmetic distress or nuisance issues to develop as a result of the site geotechnical conditions.

The geotechnical recommendations contained herein must be confirmed to be suitable or modified based on the actual as-graded conditions.

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of required earthwork removals, precise grading and construction of the proposed new improvements, including the industrial structures, subsurface utilities, and vehicular pavement areas.

We recommend that earthwork onsite be performed in accordance with the following recommendations, future grading plan review report(s), the 2019 CBC/City of Torrance requirements, and the General Earthwork and Grading Specifications for Rough Grading included in Appendix E. In case of conflict, the following recommendations shall supersede those included in Appendix E. The following recommendations may be revised within future grading plan review reports or based on the actual conditions encountered during site grading.

4.1.1 Site Preparation

Prior to grading of areas to receive structural fill or engineered improvements, the areas should be cleared of existing asphalt, surface obstructions, structures, foundations and demolition debris. Vegetation and debris should be removed and properly disposed of off-site. Holes resulting from the removal of buried obstructions, which extend below proposed finish grades, should be replaced with suitable compacted fill material. Any abandoned sewer or storm drain lines should be completely removed and replaced with properly placed compacted fill. Deeper demolition may be required in order to remove existing foundations. We recommend the trenches associated with demolition which extend below the remedial grading depth be backfilled and properly compacted prior to the demolition contractor leaving the site.

If cesspools or septic systems are encountered, they should be removed in their entirety. The resulting excavation should be backfilled with properly compacted fill soils. As an alternative, cesspools can be backfilled with lean sand-cement slurry. Any encountered wells should be properly abandoned in accordance with regulatory requirements. At the conclusion of the clearing operations, a representative of LGC Geotechnical should observe and accept the site prior to further grading.

4.1.2 Removal and Recompaction Depths and Limits

In order to provide a relatively uniform bearing condition for the planned building structures, upper loose/compressible soils are to be temporarily removed and recompacted as properly compacted fills. Existing undocumented artificial fill within the influence of the proposed structural improvements should be removed to suitable, competent native materials prior to placement of artificial fill to design grades. For preliminary planning purposes, the depth of required removals and recompaction may be estimated as indicated below. Updated recommendations may be required based on additional field work, changes to building layouts and actual structural loads.

Buildings: Soils shall be temporarily removed and recompacted to a minimum depth of 6 feet below existing grade or 3 feet below the bottom of foundations, whichever is deeper. Additionally, existing undocumented fill and unsuitable topsoil encountered within the building footprints should be removed and recompacted for use as compacted fill. Where space is available, the envelope for removal and recompaction should extend laterally a minimum distance equal to the depth of removal and recompaction below finish grade or 5 feet beyond the edges of the proposed building improvements, whichever is larger.

Minor Site Structures: For minor site structures such as free-standing walls, retaining walls, etc., removal and recompaction should extend at least 3 feet below existing grade or 2 feet below the base of foundations, whichever is deeper. Where space is available, the envelope for removal and recompaction should extend laterally a minimum distance of 3 feet beyond the edges of the proposed minor site structure improvements.

Pavement and Hardscape: Within pavement and hardscape areas, removal and recompaction should extend to a depth of at least 2 feet below the existing grade or 1 foot below finished subgrade (i.e., below planned aggregate base/asphalt concrete), whichever is deeper. In general, the envelope for removal and recompaction should extend laterally a minimum distance of 2 feet beyond the edges of the proposed pavement and hardscape improvements.

Local conditions may be encountered during excavation that could require additional over-excavation beyond the above-noted minimum in order to obtain an acceptable subgrade. The actual depths and lateral extents of grading will be determined by the geotechnical consultant, based on subsurface conditions encountered during grading. Removal areas and areas to be over-excavated should be accurately staked in the field by the Project Surveyor.

4.1.3 Temporary Excavations

Temporary excavations should be performed in accordance with project plans, specifications, and applicable Occupational Safety and Health Administration (OSHA) requirements. Excavations should be laid back or shored in accordance with OSHA requirements before personnel or equipment are allowed to enter. Based on our field investigation, the majority of site soils are anticipated to be OSHA Type "B" soils (refer to the attached boring logs). Sandy soils are present and should be considered susceptible to caving. Soil conditions should be regularly evaluated during construction to verify conditions are as anticipated. The contractor shall be responsible for providing the "competent person" required by OSHA standards to evaluate soil conditions. Close coordination with the geotechnical consultant should be maintained to facilitate construction while providing safe excavations. Excavation safety is the sole responsibility of the contractor.

Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a minimum distance equivalent to a 1:1 projection from the bottom of the excavation or 5 feet, whichever is greater. Once an excavation has been initiated, it should be backfilled as soon as practical. Prolonged exposure of temporary excavations may result in some localized instability. Excavations should be planned so that they are not initiated without sufficient time to shore/fill them prior to weekends, holidays, or forecasted rain.

It should be noted that any excavation that extends below a 1:1 (horizontal to vertical) projection of an existing foundation will remove existing support of the structure foundation. If requested, temporary shoring parameters can be provided.

4.1.4 Subgrade Preparation

In general, areas to receive compacted fill should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition (generally within optimum and 2 percent above optimum moisture content), and re-compacted per project requirements. Removal bottoms and areas to receive fill should be observed and accepted by the geotechnical consultant prior to subsequent fill placement.

4.1.5 Material for Fill

From a geotechnical perspective, the onsite soils are generally considered suitable for use as general compacted fill, provided they are screened of organic materials, construction debris and any oversized material (8 inches in greatest dimension).

From a geotechnical viewpoint, import soils for general fill (i.e., non-retaining wall backfill) should consist of clean, granular soils of Low expansion potential (expansion index of 50 or less based on ASTM D4829). Import for retaining wall backfill should meet the criteria outlined in the paragraph below. Source samples should be provided to the geotechnical consultant for laboratory testing a minimum of three working days prior to any planned importation.

Retaining wall backfill should consist of sandy soils with a maximum of 35 percent fines (passing the No. 200 sieve) per American Society for Testing and Materials (ASTM) Test Method D1140 (or ASTM D6913/D422) and a Very Low expansion potential (EI of 20 or less per ASTM D4829). Soils should also be screened of organic materials, construction debris, and any material greater than 3 inches in maximum dimension. Most of the on-site soils do not appear to be suitable for retaining wall backfill due to their fines content (i.e., silt and clay content) and expansion potential; therefore, import of select sandy materials should be anticipated by the contractor. Samples of retaining wall backfill should be obtained prior to construction and provided to the geotechnical consultant for review to confirm the suitability.

Aggregate base (crushed aggregate base or crushed miscellaneous base) should conform to the requirements of Section 200-2 of the Standard Specifications for Public Works Construction ("Greenbook") for untreated base materials (except processed miscellaneous base), the City of Torrance or Caltrans Class 2 aggregate base.

The placement of demolition materials in compacted fill is acceptable from a geotechnical viewpoint provided the demolition material is broken up into pieces not larger than approximately 2 to 4 inches in maximum dimension and well blended into fill soils with essentially no resulting voids. Demolition material placed in fills must be free of construction debris (wood, organics, etc.) and reinforcing steel. If asphalt concrete fragments will be incorporated into the demolition materials, approval from an environmental viewpoint may be required and is not the purview of the geotechnical consultant. From our previous experience, we recommend that asphalt concrete fragments be limited to fill areas within planned street areas (i.e., not within building pad areas).

4.1.6 Placement and Compaction of Fills

Material to be placed as fill should be brought to near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content) and recompacted to at least 90 percent relative compaction (per ASTM D1557). Moisture conditioning of site soils will be required in order to achieve adequate compaction. Drying and/or mixing the very moist soils may be required prior to reusing the materials in compacted fills. Additionally, soils are present that will require additional moisture in order to achieve the recommended compaction criteria.

The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in compacted thickness. Each lift should be thoroughly compacted and accepted prior to subsequent lifts. Generally, placement and compaction of fill should be performed in accordance with local grading ordinances and with observation and testing by LGC Geotechnical. Oversized material as previously defined should be removed from site fills, if encountered.

During backfill of excavations, the fill should be properly benched into firm and competent soils of temporary backcut slopes as it is placed in lifts.

Aggregate base material should be compacted to a minimum of 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to a minimum of 90 percent relative compaction, or in accordance with the City of Torrance requirements, per ASTM D1557 at near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content), unless otherwise noted in the pavement recommendations section (see Sections 4.5 and 4.6).

If gap-graded ¾-inch rock is used for backfill (around storm drain storage chambers, retaining wall backfill, etc.) it will require compaction. Rock shall be placed in thin lifts (typically not exceeding 6 inches) and mechanically compacted with observation by geotechnical consultant. Backfill rock shall meet the requirements of ASTM D2321. Gap-graded rock is required to be wrapped in filter fabric (Mirafi 140N or approved alternative) to prevent the migration of fines into the rock backfill.

4.1.7 <u>Trench and Retaining Wall Backfill and Compaction</u>

If trenches are shallow or the use of conventional equipment may result in damage to the utilities, sand having a sand equivalent (SE) of 30 or greater (per Caltrans Test Method [CTM] 217) may be used to bed and shade the pipes within the pipe zone. Sand backfill within the pipe bedding zone may be densified by jetting or flooding and then tamped to ensure adequate compaction. The onsite soils may generally be considered suitable as trench backfill (zone defined as 12 inches above the pipe to subgrade), provided the soils are screened of rocks, construction debris, other material greater than 3 inches in diameter and significant organic matter. Trench backfill should be compacted in uniform lifts (as outlined above in Section "Material for Fill") by mechanical means to at least 90 percent relative compaction (per ASTM D1557). If gap-graded rock is used for trench backfill, refer to above Section 4.1.6.

Retaining wall backfill should consist of sandy soils as outlined in preceding Section 4.1.5. The limits of select sandy backfill should extend at minimum ½ the height of the retaining wall or the width of the heel (if applicable), whichever is greater, refer to Figure 3 (rear of text). Retaining wall backfill soils should be compacted in relatively uniform thin lifts to at least 90 percent relative compaction (per ASTM D1557). Jetting or flooding of retaining wall backfill materials should not be permitted.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, typically sand-cement slurry may be substituted for compacted backfill. The slurry should contain about one sack of cement per cubic yard. When set, such a mix typically has the consistency of compacted soil. Sand cement slurry placed near the surface within landscape areas should be evaluated for potential impacts on planned improvements.

A representative from LGC Geotechnical should observe, probe, and test the backfill to verify compliance with the project recommendations.

4.1.8 Shrinkage and Subsidence

Volumetric changes in earth quantities will occur when excavated onsite earth materials are replaced as properly compacted fill. The following is an estimate of shrinkage factors for the various soil types found onsite. These estimates are based on in-place densities of the various materials and on the estimated average degree of relative compaction that will be achieved during grading.

TABLE 3

Estimated Shrinkage

Soil Type	Allowance	Estimated Range
Artificial Fill/Alluvium	Shrinkage	0 to 10 %

Subsidence due to earthwork equipment is expected to be on the order of 0.1 feet. It should be stressed that these values are only estimates and that actual shrinkage factors are extremely difficult to predict. These values are estimates only and exclude losses due to removal of vegetation or debris. The effective change in volume of onsite soils will depend primarily on the type of compaction equipment, method of compaction used onsite by the contractor, and accuracy of the topographic survey. The above shrinkage estimates are intended as an aid for others in determining preliminary earthwork quantities. However, these estimates should be used with some caution since they are not absolute values.

4.2 Preliminary Foundation Recommendations

The proposed structures may be supported on spread or continuous footings and conventional slabs, provided earthwork is performed in accordance with the recommendations presented in this report. Since the site soils are anticipated to be "Low" expansion potential (EI of 50 or less per ASTM D4829), special design considerations from a geotechnical perspective are anticipated, to minimize the impacts of expansive soils. This must be verified based on as-graded conditions.

Footings should be supported on properly compacted fill. Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of project plans (i.e., foundation, grading and site layout plans) as well as completion of earthwork/grading.

Preliminary foundation recommendations are provided in the following sections. The foundation design must be performed by the structural engineer based on the following geotechnical parameters and minimum values provided.

4.2.1 Slab Design and Construction

From a geotechnical perspective, minimum slab thicknesses of 6.5 inches and 4.5 inches are recommended for new slabs in the warehouse areas and office areas, respectively. Slabs are to be supported on compacted fill soils properly prepared in accordance with the recommendations provided in this report. Alternative slab-on-grade recommendations can be provided for alternative building types upon request. The structural engineer should structurally connect the slab to the perimeter foundation/grade beam. The actual slab reinforcement, connections and thickness should be determined by the structural engineer based on the imposed loading and geotechnical conditions of the site.

The foundation designer may use a modulus of vertical subgrade reaction (k) of 150 pounds per cubic inch (pounds per square inch per inch of deflection). This value is for a 1-foot by 1-foot square loaded area and should be adjusted by the structural designer for the area of the proposed footing using the following formula:

$k = 100 \times [(B+1)/2B]^2$
k = modulus of vertical subgrade reaction, pounds per cubic inch (pci)
B = foundation width (feet)

It is recommended that moisture content of the subgrade soils below slabs be maintained up to the time of concrete placement. The recommended moisture content of the slab subgrade soils should be between optimum moisture content and approximately 4 percent above optimum moisture content to a minimum depth of 12 inches. The moisture content of the slab subgrade should be verified by the geotechnical consultant within 1 to 2 days prior to concrete placement. In addition, this moisture content should be maintained around the immediate perimeter of the slab during construction and up to occupancy of the building structures. Additional recommendations regarding the control of surface water and landscaping adjacent to the building are provided in Section 4.9.

The following recommendations are for informational purposes only, as they are unrelated to the geotechnical performance of the foundation. The following recommendations may be superseded by the foundation engineer and/or owner. Some post-construction moisture migration should be expected below the foundation. In general, interior floor slabs with moisture sensitive floor coverings should be underlain by a minimum 10 mil thick polyolefin material vapor retarder, which has a water vapor transmission rate (permeance) of less than 0.03 perms. The need for sand and/or the sand thickness (above and/or below the vapor retarder) should be specified by the structural engineer, architect or concrete contactor. The selection and thickness of sand is not a geotechnical engineering issue and is therefore outside our purview.

4.2.2 Foundation Design Parameters

For the proposed industrial warehouse structures, minimum continuous wall and column footing widths are to be 12 inches and 24 inches, respectively, minimum foundation embedment is to extend a minimum of 18 inches below the adjacent exterior grade, and interior column footings should be embedded a minimum of 12 inches beneath the adjacent subgrade. Footing reinforcement should be designed by the structural engineer based on the structural loading conditions.

The following allowable bearing pressures for both continuous and column spread footings presented in Table 4 are recommended for corresponding footing widths and embedments.

TABLE 4

Allowable Soil Bearing Pressures

Allowable Static Bearing Pressure (psf)	Minimum Footing Width (feet)	Minimum Footing Embedment* (feet)
3,000	4	2.5
2,500	3	2
2,000	2	1.5
1,500	1	1

* Refers to minimum depth measured below lowest adjacent grade.

Perimeter building foundations should be designed to be continuous across openings such as exterior doorways and flatwork should be connected to the building.

These allowable bearing values indicated above (exclusive of the weight of the footings) are for total dead loads and frequently applied live loads and may be increased by ⅓ for short duration loading (i.e., wind or seismic loads). The allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only.

In utilizing the above-mentioned allowable bearing capacity and provided our earthwork recommendations are implemented, foundation settlement due to structural loads is anticipated to be on the order of 1-inch or less. Differential static settlement may be taken as half of the static settlement (i.e., ½-inch over a horizontal span of 40 feet).

The foundation is to be excavated into competent compacted artificial fill placed during grading operations. It is recommended that the foundation subgrade soils be evaluated by the geotechnical engineer prior to steel and/or concrete placement.

4.2.3 Lateral Load Resistance

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.3 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 225 psf per foot of depth (or pcf) to a maximum of 2,250 psf may be used for the sides of footings poured against properly compacted fill. Allowable passive pressure may be increased to 300 pcf (maximum of 3,000 psf) for short duration seismic loading. This passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions. Frictional resistance and passive pressure may be used in combination without reduction. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt. The provided allowable passive pressures are based on a factor of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively.

4.3 Lateral Earth Pressures for Retaining Walls

The following preliminary lateral earth pressures may be used for site retaining walls. Lateral earth pressures are provided as equivalent fluid unit weights, in pound per square foot (psf) per foot of depth or pcf. These values do not contain an appreciable factor of safety, so the retaining wall designer should apply the applicable factors of safety and/or load factors during design.

The following lateral earth pressures are presented on Table 5 for approved on-site select or import granular soils with a maximum of 35 percent fines (passing the No. 200 sieve per ASTM D-421/422) and Very Low expansion potential (EI of 20 or less per ASTM D4829). Retaining wall backfill should also be limited to fill material not exceeding 3 inches in greatest dimension. The wall designer should clearly indicate on the retaining wall plans the required sandy soil backfill criteria. Most of the on-site soils do not appear to be suitable for retaining wall backfill due to their fines content (i.e., silt and clay content) and expansion potential; therefore, import of select sandy materials should be anticipated by the contractor.

TABLE 5

Lateral Earth Pressures – Import Sandy Backfill

Conditions	Equivalent Fluid Unit Weight (pcf)	Equivalent Fluid Unit Weight (pcf)
	Level Backfill	**2:1 Sloped Backfill**
	Approved Sandy Soils	**Approved Sandy Soils**
Active	35	55
At-Rest	55	70

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 earth pressure will be higher. The equivalent fluid pressure values assume free-draining conditions. Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed (Refer to Figure 3). Please note that waterproofing and outlet systems are not the purview of the geotechnical consultant. 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 consultant.

Surcharge loading effects from any adjacent structures should be evaluated by the retaining wall designer. In general, structural loads within a 1:1 (horizontal to vertical) upward projection from the bottom of the proposed retaining wall footing will surcharge the proposed retaining structure. In addition to the recommended earth pressure, retaining walls adjacent to streets should be designed to resist vehicular traffic if applicable. Uniform surcharges may be estimated using the applicable coefficient of lateral earth pressure using a rectangular distribution. A factor of 0.35 and 0.5 may be used for the active and at-rest conditions, respectively. The vertical traffic surcharge may be determined by the structural designer. The retaining wall designer should contact the geotechnical engineer for any required geotechnical input in estimating any applicable surcharge loads.

If required, the retaining wall designer may use a seismic lateral earth pressure increment of 5 pcf for level backfill conditions. This increment should be applied in addition to the provided static lateral earth pressure using a "normal" triangular distribution with the resultant acting at H/3 in relation to the base of the retaining structure (where H is the retained height). For the restrained, at-rest condition, the seismic increment may be added to the applicable active lateral earth pressure (in lieu of the at-rest lateral earth pressure) when analyzing short duration seismic loading. Per Section 1803.5.12 of the 2019 CBC, the seismic lateral earth pressure is applicable to structures assigned to Seismic Design Category D through F for retaining wall structures supporting more than 6 feet of backfill height. This seismic lateral earth pressure is estimated using the procedure outlined by the Structural Engineers Association of California (Lew, et al, 2010).

Soil bearing and lateral resistance (friction coefficient and passive resistance) are provided in Section 4.2. Earthwork considerations (temporary backcuts, backfill, compaction, etc.) for retaining walls are provided in Section 4.1 (Site Earthwork) and the subsequent earthwork related sub-sections.

4.4 Corrosivity to Concrete and Metal

Although not corrosion engineers (LGC Geotechnical is not a corrosion consultant), several governing agencies in Southern California require the geotechnical consultant to determine the corrosion potential of soils to buried concrete and metal facilities. We therefore present the results of our testing with regard to corrosion for the use of the client and other consultants, as they determine necessary.

Corrosion testing of near-surface bulk samples indicated soluble sulfate contents less than approximately 0.01 percent, chloride content of approximately 185 parts per million (ppm), pH value of approximately 8.13, and minimum resistivity value of 1,490 ohm-cm. Based on Caltrans Corrosion Guidelines (2021), soils are considered corrosive if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 1,500 ppm (0.15 percent) or greater. Based on the test results, soils are not considered corrosive using Caltrans criteria.

Based on laboratory sulfate test results, the near surface soils are designated to a class “S0” per ACI 318, Table 19.3.1.1 with respect to sulfates. Concrete in direct contact with the onsite soils can be designed according to ACI 318, Table 19.3.2.1 using the “S0” sulfate classification.

Laboratory testing may need to be performed at the completion of grading by the project corrosion engineer to further evaluate the as-graded soil corrosivity characteristics. Accordingly, revision of the corrosion potential may be needed, should future test results differ substantially from the conditions reported herein. The client and/or other members of the development team should consider this during the design and planning phase of the project and formulate an appropriate course of action.

4.5 Preliminary Asphalt Concrete Pavement Sections

For the purposes of these preliminary recommendations, we have assumed an R-value of 15 and calculated pavement sections for Traffic Indices of 5.0 (or less), 6.0, and 7.0. R-value testing of the drive aisles and parking subgrade will need to be performed to confirm our preliminary testing results/assumptions once the underground utilities have been backfilled, drive aisles and parking areas have been graded to finish subgrade elevations, and the final Traffic Index is determined by the Civil Engineer. Determination of the Traffic Index is not the purview of the geotechnical consultant. Final asphalt concrete pavement sections should be confirmed by the project civil engineer based upon the projected design Traffic Index. If requested, LGC Geotechnical will provide sections for alternate TI values.

TABLE 6

Preliminary Asphalt Concrete Pavement Sections

Assumed Traffic Index	5.0 (or less)	6.0	7.0
R -Value Subgrade	15	15	15
AC Thickness	4.0 inches	4.0 inches	5.0 inches
Aggregate Base Thickness	6.0 inches	9.5 inches	11.5 inches

Increasing the thickness of asphalt or adding additional base material will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur through the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement.

Aggregate base material (crushed aggregate base and crushed miscellaneous base) should be compacted to a minimum of 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to a minimum of 90 percent relative compaction, or the City of Torrance specifications, at or slightly above optimum moisture content per ASTM D1557. Earthwork recommendations are provided in Section 4.1 "Site Earthwork" and the related sub-sections of this report.

4.6 Preliminary Portland Cement Concrete Pavement Sections

For the purposes of these preliminary recommendations, we used an assumed R-value of 15. Preliminary minimum Portland Cement Concrete (PCC) pavement street sections are provided in Table 7 for Traffic Indices of 5.0 (or less), 6.0, and 7.0 and may be utilized in the design. These recommendations must be confirmed with R-value testing of representative near-surface soils at the completion of grading and after underground utilities have been installed and backfilled. Final PCC pavement sections should be confirmed by the project civil engineer based upon the projected design Traffic Index. If requested, LGC Geotechnical will provide sections for alternate TI values. The appropriate paving section must be selected by the project civil engineer/client based on design traffic indexes.

TABLE 7

Preliminary PCC Pavement Sections

Provided Traffic Index	5.0 (or less)	6.0	7.0
R -Value Subgrade	15	15	15
PCC Thickness	5.0 inches	6.0 inches	7.0 inches
Aggregate Base Thickness	4.0 inches	4.0 inches	4.0 inches

For preliminary planning purposes, the PCC pavement sections may consist of a minimum of concrete over aggregate base compacted to 95 percent relative compaction (see Table 7 for section thicknesses). The concrete should have a minimum compressive strength of 3,250 psi and a minimum flexural strength of 505 psi at the time the pavement is subjected to traffic. Steel reinforcement is not required (ACI, 2017). The provided pavement sections assume that edge restraints like a curb and gutter will be provided. To reduce the potential (but not eliminate) for cracking, paving should provide control joints at regular intervals in each direction. The maximum joint spacing within all PCC pavements is recommended to be equal to or less than 30 times the pavement thickness; however, we recommend joint spacing not exceed 15 feet in each direction. Joints should be a depth of 1/3 of the concrete thickness. Decreasing the spacing of these joints will further reduce, but not eliminate the potential for unsightly cracking.

If semi-trailers are to be disconnected from the tractors from dolly jacks the design should consider concentrated loads imposed on the concrete pavement. These loads typically exceed the axle loads of the semi-trailer combination and are applied to smaller contact areas, especially if applied near joint locations. If these irregular loadings are confined to specific areas of the site, the pavement section required thickness can be economized. These and other factors (e.g., traffic patterns, irregular loading, doweled vs un-doweled joints, etc.) outlined in ACI, 2017 should be addressed for the final design.

The thicknesses shown are <u>minimum</u> thicknesses. Increasing the thickness of any or all of the above layers will reduce the likelihood of the pavement experiencing distress during its service life. The above recommendations are based on the assumption that proper maintenance and irrigation of the areas adjacent to the roadway will occur through the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement.

Aggregate base material (crushed aggregate base and crushed miscellaneous base) should be compacted to a minimum of 95 percent relative compaction at or slightly above optimum moisture content per ASTM D1557. Subgrade below aggregate base should be compacted to a minimum of 90 percent relative compaction, or the City of Torrance specifications, at or slightly above optimum moisture content per ASTM D1557. Earthwork recommendations are provided in Section 4.1 "Site Earthwork" and the related sub-sections of this report.

4.7 Nonstructural Concrete Flatwork

Nonstructural concrete (such as flatwork, sidewalks, etc.) has a potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete should be designed in accordance with the minimum guidelines outlined below. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints but will <u>not</u> eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

Nonstructural and non-vehicular concrete flatwork placed on compacted subgrade may be a minimum 4-inches in thickness (full) with crack control joints spaced 8 feet apart for flatwork slabs and 6 feet apart for flatwork sidewalks. Crack control joints should be sawcut or deep open tool joint to a minimum of 1/3 the concrete thickness. Reinforcement should consist of No. 3 bars spaced at 24 inches on center, both ways. The compacted subgrade below the nonstructural and non-vehicular concrete flatwork should be wet down prior to placing concrete.

To reduce the potential for nonstructural concrete flatwork to separate from entryways and doorways, the owner may elect to install dowels to tie these two elements together.

4.8 Subsurface Water Infiltration

It should be noted that intentionally infiltrating storm water conflicts with the geotechnical engineering objective of directing surface water away from structures and improvements. The geotechnical stability and integrity of a site is reliant upon appropriately handling surface water.

In general, the vast majority of geotechnical distress issues are directly related to improper drainage. Distress in the form of movement of foundations and other improvements could occur as a result of soil saturation and loss of soil support of foundations and pavements, settlement, collapse, internal soil erosion, and/or expansion. Additionally, off-site properties and improvements may be subjected to seepage, springs, instability, movements of foundations or other impacts as a result of water infiltration and migration. Infiltrated water may enter underground utility pipe zones or other highly permeable layers and migrate laterally along these layers, potentially impacting other improvements located far away from the point of infiltration. Any proposed infiltration system should not be located near slopes or settlement sensitive existing/proposed improvements in order to reduce the potential for slope failures and geotechnical distress issues related to infiltration.

If water must be infiltrated due to regulatory requirements, we recommend the absolute minimum amount of water be infiltrated and that the infiltration areas not be located near settlement-sensitive existing/proposed improvements, basement/retaining walls, or any slopes. As with all systems that are designed to concentrate surface flow and direct the water into the subsurface soils, some minor settlement, nuisance type localized saturation and/or other water related issues should be expected. Due to variability in geologic and hydraulic conductivity characteristics, these effects may be experienced at the onsite location and/or potentially at other locations beyond the physical limits of the subject site. Infiltrated water may enter underground utility pipe zones or flow along heterogeneous soil layers or geologic structure and migrate laterally impacting other improvements which may be located far away or at an elevation much lower than the infiltration source. Recommendations for subsurface water infiltration are provided below.

The design infiltration rate is determined by dividing the measured infiltration rate by a series of reduction factors including; test procedure (RF_t), site variability (RF_v) and long-term siltation plugging and maintenance (RF_s). Based on the Los Angeles County testing guidelines (2021), the reduction factor for long-term siltation plugging and maintenance (RF_s) is the purview of the infiltration system designer. The test procedure reduction factor and recommended site variability reduction factor applied to the measured infiltration rate is provided in Table 8 below. The design infiltration rate is the measured infiltration rate divided by the total reduction factor ($RF_t + RF_v + RF_s$).

TABLE 8

Reduction Factors Applied to Measured Infiltration Rate

Consideration	Reduction Factor
Test procedure, boring percolation, RF_t	1.0
Site variability, number of tests, etc., RF_v	1.0
Long-term siltation plugging and maintenance, RF_s	Per Infiltration Designer
Total Reduction Factor, $RF = RF_t + RF_v + RF_s$	**TBD**

Per the requirements of the Los Angeles County testing guidelines (2021), subsurface materials shall have a design infiltration rate equal to or greater than 0.3 inches per hour. When the Total Reduction Factor (will be at least 3.0, to be determined by the civil engineer) is applied to the measured infiltration rate of infiltration test, the resulting design infiltration rate for infiltration test I-2 may be equal to or greater than the minimum infiltration rate required by the County of Los Angeles for infiltration. Therefore, considering the results of the infiltration testing at I-2 and review of the subsurface data below a depth of 15 feet across the site, if required, stormwater may be infiltrated into the subsurface soils at a depth of at least 15 feet below existing grade using the values presented in Table 1 and Reduction Factors presented above in Table 8. Results of field infiltration testing are provided in Appendix D.

The following should be considered for design of any required infiltration system:

- To facilitate infiltration more favorably, we recommend drilling approximately 8-inch diameter holes to depths of approximately 20 feet below the bottom of the infiltration system bottom (~35 feet below existing grade) and backfilling the holes with clean granular sand or crushed rock. The drilled holes would likely be spaced about 20 feet on center along the infiltration system bottom. Actual dimensions and spacing of drilled holes may differ based on conditions exposed during grading.
- Water discharge from any infiltration systems should not occur within the zone of influence of foundation footings (column and load bearing wall locations). For preliminary purposes we recommend a minimum setback of 15 feet from the structural improvements.
- An adequate setback distance between any infiltration facility and adjacent private property should be maintained.
- The water quality infiltration system should be designed with an overflow system directly connected to the storm drain system in order to prevent failure of the infiltration system, either as a result of lower than anticipated infiltration and/or very high flow volumes.
- The infiltration values provided are based on clean water and this requires the removal of trash, debris, soil particles, etc., and on-going maintenance. Over time, siltation and plugging may reduce the infiltration rate and subsequent effectiveness of the infiltration system. It should be noted that methods to prevent this shall be the responsibility of the infiltration designer and are not the purview of the geotechnical consultant. If adequate measures cannot be incorporated into the design and maintenance of the system, then the infiltration rates may need to be further reduced. These and other factors should be considered in selecting a design infiltration rate.
- Any designed infiltration system will require routine periodic maintenance.
- As with any systems that are designed to concentrate the surface flow and direct the water into the subsurface soils, some type of nuisance water and/or other water-related issues should be expected.
- Contamination and environmental suitability of the site for infiltration was not evaluated by us and should be evaluated by others (environmental consultant). We only addressed the geotechnical issues associated with stormwater infiltration.

LGC Geotechnical should be provided with details for any planned required infiltration system early in the design process for geotechnical input.

4.9 Surface Drainage and Landscaping

Due to the presence of expansive soils, special provisions should be considered to limit the potential for surface water to penetrate the soils adjacent to the proposed structures and improvements.

4.9.1 General

Surface drainage should be carefully taken into consideration during precise grading, building construction, future landscaping, and throughout the design life of the industrial structure. Positive drainage should be provided to direct surface water away from improvements and towards either the street or other suitable drainage devices. Ponding of water, adjacent to any structural improvement foundation, must be avoided. The performance of structural foundations is dependent upon maintaining adequate surface drainage away from them, thereby reducing excessive moisture fluctuations. From a geotechnical perspective, area drains, drainage swales, and finished grade soils should be aligned so as to transport surface water to a minimum distance of 5 feet away from the proposed foundations. Roof gutters and downspout systems should be discharged directly to a pipe or to a paved surface with a positive gradient away from the building and should not outlet directly into unpaved landscape areas.

Decorative gravel tends to act as a reservoir trapping surface water, therefore, we do not recommend it be used adjacent to buildings unless the system is designed with a subsurface drainage system and is properly lined.

4.9.2 Precise Grading

From a geotechnical perspective, we recommend that compacted finished grade soils adjacent to the proposed industrial structures be sloped away from the proposed structures and towards an approved drainage device or unobstructed swale. Drainage swales, wherever feasible, should not be constructed within 5 feet of buildings. Where lot and building geometry necessitates that the drainage swales be routed closer than 5 feet to structural foundations, we recommend the use of area drains together with drainage swales. Drainage swales used in conjunction with area drains should be designed by the project civil engineer so that a properly constructed and maintained system will prevent ponding within 5 feet of the foundation. Code compliance of grades is not the purview of the geotechnical consultant. We do not recommend that area drains be connected to basement/retaining subdrains.

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

4.9.3 Landscaping

Planters adjacent to a building or structure should be avoided wherever possible or be properly designed (e.g., lined with a membrane and properly outlet), to reduce the penetration of water into the adjacent footing subgrades and thereby reduce moisture related damage to the foundation. Planting areas at grade should be provided with appropriate positive drainage. Wherever possible, exposed soil areas should be above adjacent paved grades to facilitate drainage. Planters should not be depressed below adjacent paved grades unless provisions for drainage, such as multiple depressed area drains, are constructed. Adequate drainage gradients, devices, and curbing should be provided to prevent runoff from adjacent pavement or walks into the planting areas. Irrigation methods should promote uniformity of moisture in planters and beneath adjacent concrete flatwork. Overwatering and underwatering of landscape areas must be avoided. Irrigation levels should be kept to the absolute minimum level necessary to maintain healthy plant life.

Area drain inlets should be maintained and kept clear of debris in order to properly function. The building owner should also be made aware that excessive irrigation of neighboring properties can cause seepage and moisture conditions on adjacent lots.

The impact of heavy irrigation or inadequate runoff gradients can create perched water conditions. This may result in seepage or shallow groundwater conditions where previously none existed. Maintaining adequate surface drainage and controlled irrigation will significantly reduce the potential for nuisance-type moisture problems. To reduce differential earth movements such as heaving and shrinkage due to the change in moisture content of foundation soils, which may cause distress to a structure and associated improvements, moisture content of the soils surrounding the structure should be kept as relatively constant as possible.

4.10 Geotechnical Plan Review

Project plans (grading, foundation, retaining wall, etc.) should be reviewed by this office prior to construction to verify that our geotechnical recommendations have been incorporated. Additional or modified geotechnical recommendations may be required based on the proposed layout.

4.11 Geotechnical 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. Geotechnical observation and testing is required per Section 1705 of the 2019 California Building Code (CBC).

Geotechnical observation and/or testing should be performed by LGC Geotechnical at the following stages:

- During grading (removal bottoms, fill placement, etc.);
- During retaining wall backfill and compaction;
- During utility trench backfill and compaction;
- During drilling and backfilling of holes in bottom of infiltration system;
- During precise grading;
- Preparation of building pads and other concrete-flatwork subgrades, and prior to placement of aggregate base or concrete;
- After building and wall footing excavation and prior to placement of steel reinforcement and/or concrete;
- Preparation of pavement subgrade and placement of aggregate base; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

5.0 LIMITATIONS

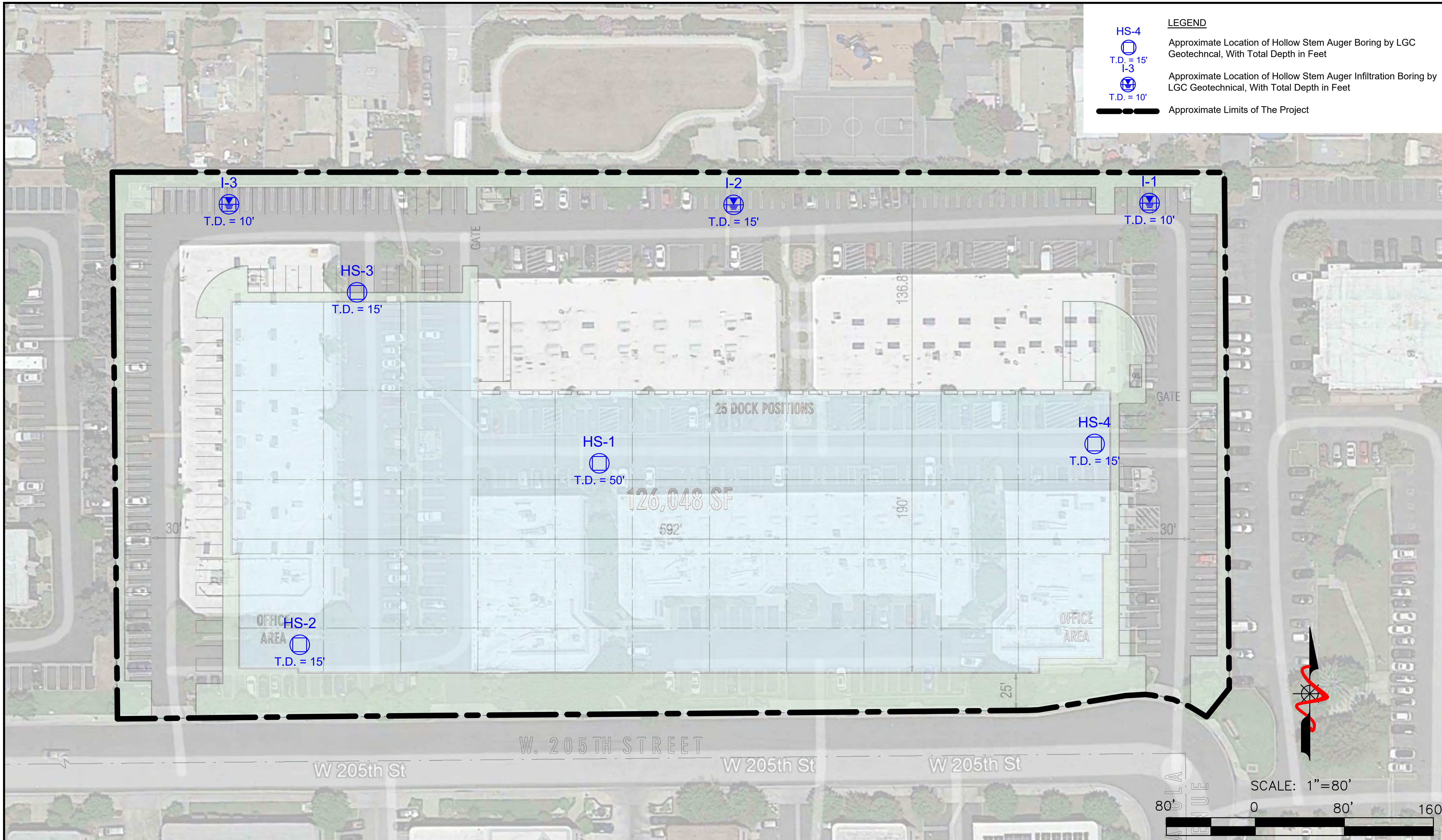
Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable soils engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report.

This report is based on data obtained from limited observations of the site, which have been extrapolated to characterize the site. While the scope of services performed is considered suitable to adequately characterize the site geotechnical conditions relative to the proposed development, no practical evaluation can completely eliminate uncertainty regarding the anticipated geotechnical conditions in connection with a subject site. Variations may exist and conditions not observed or described in this report may be encountered during grading and construction.

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 other consultants (at a minimum the civil engineer, structural engineer, landscape architect) and incorporated into their plans. The contractor should properly implement the recommendations during construction and notify the owner if they consider any of the recommendations presented herein to be unsafe, or unsuitable.

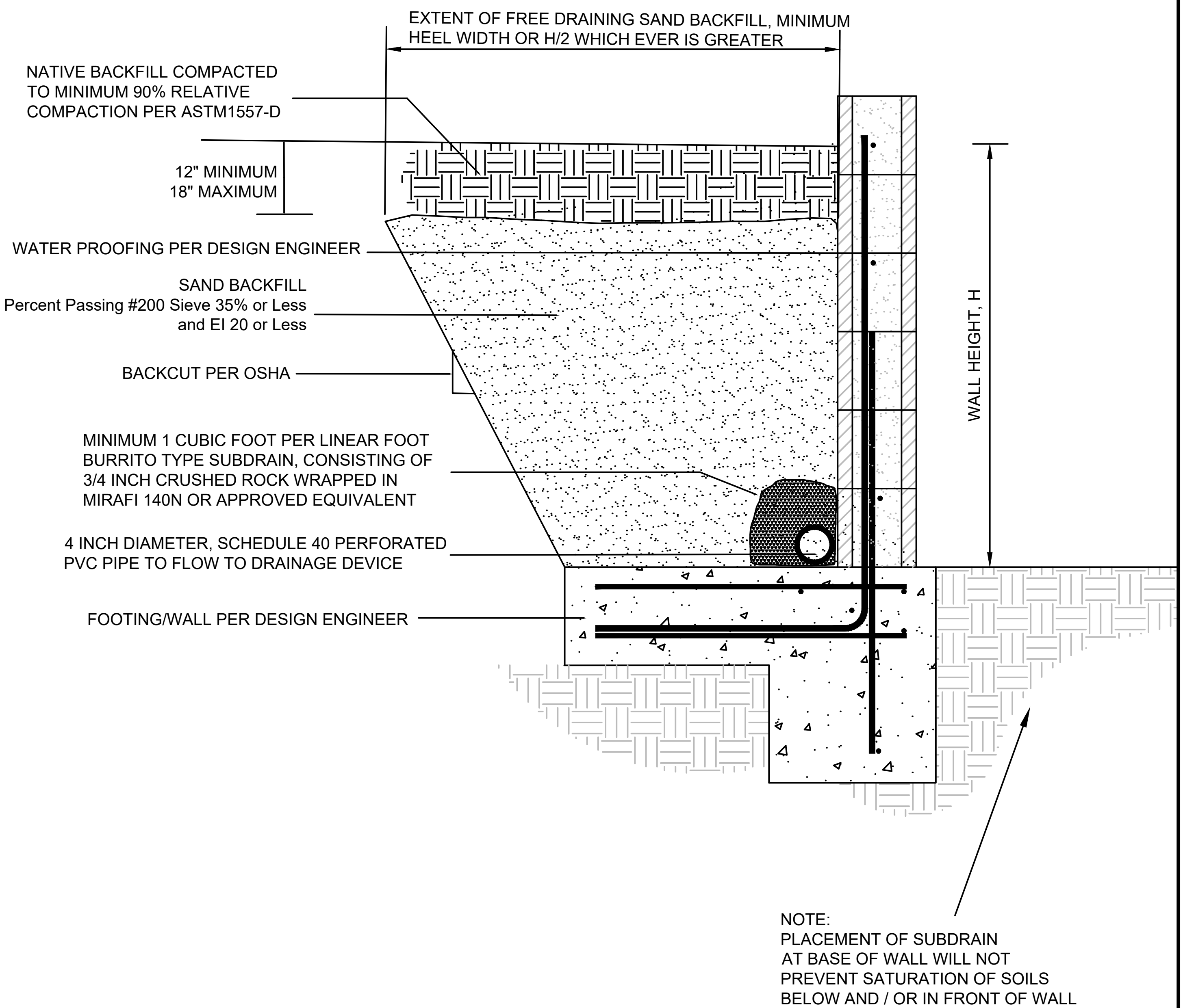
The findings of this report are valid as of the present date. However, changes in the conditions of a site can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. The findings, conclusions, and recommendations presented in this report can be relied upon only if LGC Geotechnical has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site. This report is intended exclusively for use by the client, any use of or reliance on this report by a third party shall be at such party's sole risk.

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. Therefore, this report is subject to review and modification.



LGC Geotechnical, Inc.	LGC Geotechnical, Inc. 131 Calle Iglesia, Ste. 200 San Clemente, CA 92672 TEL (949) 369-6141 FAX (949) 369-6142	**FIGURE 2** **Boring Location Map**

PROJECT NAME	EPD - Torrance
PROJECT NO.	22070-01
ENG. / GEOL.	RLD
SCALE	1" = 80'
DATE	June 2022



LGC Geotechnical, Inc.

FIGURE 3
Retaining Wall Backfill Detail

PROJECT NAME	EPD - Torrance
PROJECT NO.	22070-01
ENG. / GEOL.	RLD
SCALE	Not to Scale
DATE	June 2022