



Appendix 4.7-1

Preliminary Geotechnical Evaluation

August 17, 2023

Project No. 23001-01

Mr. Ingo Giani
Trojan Storage
1732 Aviation Boulevard, Suite 217
Redondo Beach, California 90278

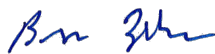
Subject: Preliminary Geotechnical Evaluation, Proposed Self-Storage Development, 7528 North Bellair, North Hollywood, California

In accordance with your request, LGC Geotechnical, Inc. has performed a preliminary geotechnical evaluation for the proposed self-storage development to be located at 7528 North Bellair in North Hollywood, California. The purpose of our study was to evaluate the existing onsite geotechnical conditions and to provide preliminary geotechnical recommendations relative to the proposed development.


Should you 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.



Brad Zellmer, GE 2618
Project Engineer



Kevin B. Colson, CEG 2210
Vice President



BPG/KBC/BTZ/amm

Distribution: (3) Addressee (2 wet-signed copies and 1 PDF on a flash drive for agency submittal)

TABLE OF CONTENTS

<u>Section</u>	<u>Page</u>
1.0 INTRODUCTION.....	1
1.1 Project Description.....	1
1.2 Subsurface Exploration.....	1
1.3 Field Percolation Testing.....	2
1.4 Laboratory Testing.....	2
2.0 GEOTECHNICAL CONDITIONS.....	4
2.1 Regional Geology.....	4
2.2 Site-Specific Geology.....	4
2.3 Geologic Structure.....	4
2.4 Groundwater.....	4
2.5 Landslides.....	5
2.6 Faulting.....	5
2.6.1 Lurching and Shallow Ground Rupture.....	5
2.6.2 Liquefaction and Dynamic Settlement.....	6
2.6.3 Lateral Spreading.....	6
2.6.4 Tsunamis and Seiches.....	6
2.7 Seismic Design Parameters.....	6
2.8 Rippability.....	8
2.9 Oversized Material.....	8
3.0 FINDINGS AND CONCLUSIONS.....	9
4.0 RECOMMENDATIONS.....	10
4.1 Site Earthwork.....	10
4.1.1 Site Preparation.....	10
4.1.2 Removal Depths and Limits.....	11
4.1.3 Temporary Excavations.....	12
4.1.4 Removal Bottoms and Subgrade Preparation.....	12
4.1.5 Material for Fill.....	13
4.1.6 Fill Placement and Compaction.....	13
4.1.7 Trench and Retaining Wall Backfill and Compaction.....	14
4.1.8 Shrinkage and Subsidence.....	14
4.2 Preliminary Foundation Design Parameters.....	15
4.2.1 Slab Design and Construction.....	15
4.2.2 Shallow Foundation Maintenance.....	16
4.3 Soil Bearing and Lateral Resistance.....	16
4.4 Lateral Earth Pressures for Retaining Walls.....	17
4.5 Preliminary Pavement Sections.....	19
4.6 Soil Corrosivity.....	20
4.7 Nonstructural Concrete Flatwork.....	20
4.8 Surface Drainage and Landscaping.....	21
4.8.1 Precise Grading.....	21

TABLE OF CONTENTS cont'd

4.8.2 Landscaping 21

4.9 Subsurface Water Infiltration 22

4.10 Pre-Construction Documentation and Construction Monitoring..... 24

4.11 Geotechnical Plan Review 24

4.12 Geotechnical Observation and Testing During Construction 24

5.0 LIMITATIONS.....25

LIST OF TABLES, ILLUSTRATIONS, & APPENDICES

Tables

- Table 1 – Summary of Field Infiltration Testing (Page 2)
- Table 2 – Seismic Design Parameters (Page 7)
- Table 3 – Lateral Earth Pressures – Sandy Backfill (Page 18)
- Table 4 – Asphalt Concrete Pavement Section Options (Page 19)
- Table 5 – Nonstructural Concrete Flatwork for Very Low Expansion Potential (Page 21)
- Table 6 – Reduction Factors Applied to Measured Infiltration Rate (Page 23)

Figures

- Figure 1 – Site Location Map (Rear of Text)
- Figure 2 – Boring Geotechnical Map (Rear of Text)
- Figure 3 – Retaining Wall Backfill Detail (Rear of Text)

Appendices

- Appendix A – References
- Appendix B – Field Exploration Logs and Field Infiltration Data
- Appendix C – Laboratory Test Results
- Appendix D – General Earthwork and Grading Specifications

1.0 INTRODUCTION

LGC Geotechnical has performed a geotechnical evaluation for the proposed self-storage development to be located at 7528 North Bellair in North Hollywood, California (Figure 1). This report summarizes our findings, conclusions, and preliminary geotechnical recommendations relative to the proposed development.

1.1 Project Description

The approximately 4-acre, roughly rectangular shaped site is bound by Bellaire Ave to the west, existing residential development to the north, SR-170 to the east, and by Saticoy Street and a vacant lot to the south. The site is relatively flat, primarily undeveloped land with the exception of a single-family home and rear shed or barn structure. Based on aerial photographic review, the site appears to have been utilized as a salvage yard since at least the late 1970s when the current structures were built. Prior to that the site looks to have been used for minor agricultural use dating back to the late 1940s (Historic Aerials, 2023).

We understand that the proposed development is to include grading and construction for a three-story self-storage building and office and a two-story apartment building and associated parking, drive aisles, open space and improvements. (Jordan, 2023). Preliminary building (dead plus live) loads were not provided at the time of this report. However, we have estimated the maximum column and wall structural (dead plus live) loads to be 100 kips and 5 kips per lineal foot, respectively. A preliminary grading plan was not available at the time of this report; however, design (not including remedial grading) cuts and fills are anticipated to be relatively minor from existing grades. Retaining walls of any significant height are not anticipated.

The recommendations given in this report are based on the layout and estimated structural loads and grading information as indicated above. LGC Geotechnical should be provided with any updated project information, plans and/or any structural loads when they become available, in order to either confirm or modify the recommendations provided herein.

1.2 Subsurface Exploration

In July of 2023, LGC Geotechnical performed a subsurface geotechnical evaluation of the subject site consisting of the excavation of eight hollow-stem auger borings in order to evaluate onsite geotechnical conditions.

Eight borings (HS-1 through HS-6, I-1 through I-2) were excavated using a truck-mounted drill rig equipped with 8-inch-diameter hollow-stem augers with depths ranging from approximately 10 to 51.5 feet below existing grade. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. Driven soil samples were collected by means of the Standard Penetration Test (SPT) and Modified California Drive (MCD) sampler. The SPT sampler (1.4-inch ID) and MCD sampler (2.4-inch ID, 3.0-inch OD) were driven using a 140-pound hammer falling 30 inches to advance the sampler a total depth of 18 inches or until refusal. The raw blow counts for each 6-inch increment of penetration were

recorded on the boring logs. Bulk samples were also collected and logged for laboratory testing at select depths. In select borings, after removal of the augers the depth of the boring due to caving was measured and is noted on the boring logs. The borings were backfilled with cuttings. Please note that some settlement of the backfill may occur over time and the excavations should be topped off as needed. The approximate locations of our subsurface explorations are provided on the Boring Geotechnical Map (Figure 2). The boring logs are provided in Appendix B.

At the completion of the excavation of Borings I-1 through I-2, an infiltration well was constructed within each boring for testing as outlined in the “Field Percolation Testing” section below. At the completion of infiltration testing, the installed pipe was removed, and the resulting void backfilled with native soils. Please note that some settlement of the backfill may occur over time and the excavations should be topped off as needed.

1.3 Field Percolation Testing

Field percolation tests (I-1 through I-2) were performed to approximate depths of 10 feet below existing grade. The approximate locations are shown on Figure 2. The borings for the infiltration tests were excavated using a drill rig equipped with 8-inch diameter hollow-stem augers. Estimation of infiltration rates was accomplished in general accordance with the guidelines set forth by the County of Los Angeles (2017 & 2021). A 3-inch diameter perforated PVC pipe was placed in the borehole, and the annulus was backfilled with gravel. The infiltration wells were pre-soaked prior to testing.

Based on the County of Los Angeles testing guidelines, the measured infiltration rates are provided in Table 1 below. Please note that the values provided in Table 1 do not include required reduction factors for test method (RF_t), site variability and long-term siltation plugging that are required for a design infiltration rate. Infiltration tests are performed using relatively clean water free of particulates, silt, etc. Infiltration data is provided in Appendix B. Refer to Section 4.9.

TABLE 1

Summary of Field Infiltration Testing

Infiltration Test Location	Measured Infiltration Rate* (inch/hr.)
I-1	134.4
I-2	46.6

*No Reduction Factors Applied (RF_t, Site Variability, and Long-Term Siltation)

1.4 Laboratory Testing

Representative bulk and driven samples were obtained for laboratory testing during our field evaluation. Laboratory testing was performed at a certified geotechnical testing laboratory for the City of Los Angeles (Leighton). We have reviewed and concur with the test results and accept

the responsibility for their use in our analysis. Laboratory testing included in-situ moisture content and dry density, consolidation/collapse potential, sieve analysis, expansion index, laboratory compaction, R-Value and corrosion characteristics (soluble sulfate, chloride, pH and minimum resistivity). A summary of the laboratory test results is presented below with complete results provided in Appendix C.

- Dry density of the samples collected ranged from approximately 99 pounds per cubic foot (pcf) to 128 pcf, with an average of 114 pcf. Field moisture contents ranged from less than 1 percent to approximately 7 percent, with an average of approximately 3 percent.
- Nine fines content/gradation tests were performed and indicated fines content (passing No. 200 sieve) ranging from approximately 4 to 20 percent. According to the Unified Soils Classification System (USCS) the tested samples are classified as “coarse-grained” soil.
- Two Expansion Index (EI) tests indicated EI values of zero, corresponding to “Very Low” expansion potential.
- Four consolidation tests were performed on select samples. Collapse at water inundation was relatively negligible (approximately ¼ percent or less). The deformation versus vertical stress plots are provided in Appendix C.
- A laboratory compaction test resulted in a maximum dry density of 123.5 pcf at optimum moisture content of 8.5 percent.
- Corrosion testing indicated soluble sulfate contents of 0.01 percent or less, a chloride content of 60 parts per million (ppm), pH of 6.7, and a minimum resistivity of 7,350 ohm-centimeters.
- An R-Value test was performed, results indicated a value of 75.

Laboratory test results obtained from our field evaluation are provided in Appendix C. The moisture and dry density results are presented on the boring logs in Appendix B.

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

The subject site is located within the San Fernando Valley which is an east trending structural trough within the Transverse Ranges of Southern California. As the adjacent San Gabriel Mountains have uplifted, the San Fernando Valley has subsided and filled with sediment. The eastern portion of the valley has received sediment from the Pacoima and Tujunga washes which are associated with large river systems in the San Gabriel Mountains. These river systems, along with smaller localized streams, have deposited broad alluvial fans that cover the majority of the San Fernando Valley with sand, silt and gravel (USGS, 1996).

2.2 Site-Specific Geology

Based on our review of regional geologic maps for the area of the site (USGS, 1996), the project area is mapped as being underlain by Quaternary young alluvial fan deposits (Map Symbol – Qyf), which generally include; sand and gravel. These native materials are likely locally overlain by thin areas (less than approximately 2 to 3 feet) of artificial fill associated with original development of the site. For the purposes of this study, these areas of fill are not differentiated from the alluvial fan deposits.

As encountered in our subsurface evaluation, site materials primarily consist of medium dense to very dense sands with varying amounts of silts and gravels. Moisture contents are generally well below optimum.

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 Geologic Structure

Geologic structure was not identified in the subject site geotechnical evaluation. The alluvial materials encountered are generally massive, but may include low angle bedding, dipping in a south-southeasterly direction.

2.4 Groundwater

Groundwater was not encountered during our subsurface evaluation to the maximum explored depth of approximately 51.5 feet below existing ground surface. Estimated historic high groundwater is on the order of 80 feet below existing grade (CGS, 2001).

Seasonal fluctuations of groundwater elevations should be expected over time. 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. Local perched groundwater conditions or surface seepage may develop once site development is completed and landscape irrigation commences.

2.5 Landslides

Our research and field observations do not indicate the presence of landslides on the site or in the immediate vicinity. Review of regional geologic maps of the area do not indicate the presence of known or suspected landslides in the vicinity of the site.

2.6 Faulting

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 (CGS, 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 Fault Rupture Hazard Zone (CGS, 2018). There are no known active or potentially active faults mapped on the site. The possibility of damage due to ground rupture, as a result of faulting, is considered very low since active faults are not known to cross the site.

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, which may affect the site, include ground lurching and 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. A discussion of these secondary effects is provided in the following sections.

2.6.1 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 structures. Ground rupture due to active faulting is not likely to occur onsite due to the absence of known active

fault traces. Ground cracking due to shaking from distant seismic events is not considered a significant hazard, although it is a possibility at any site.

2.6.2 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 saturated, loose 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, depending on their plasticity and moisture content (Bray & Sancio, 2006). Effects of liquefaction on level ground include settlement, sand boils, and bearing capacity failures below structures. Dynamic settlement of dry loose sands can occur as the sand particles tend to settle and densify as a result of a seismic event.

The site is not located within a State of California Seismic Hazard Zone (CGS, 2001) for liquefaction potential. In general, the site will consist of compacted fill over dense to very dense soils. Due to the absence of groundwater the potential for liquefaction and dynamic settlement is considered very low.

2.6.3 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 down-slope 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 very low potential for liquefaction, the potential for lateral spreading is also considered very low.

2.6.4 Tsunamis and Seiches

Based on the elevation of the site, with respect to sea level, there is a very low possibility of damage to the site during a large tsunami event.

2.7 Seismic Design Parameters

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2022 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 34.2074 degrees north and longitude -118.4097 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 below. 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.

TABLE 2
Seismic Design Parameters

Selected Parameters from 2022 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
S_s (Risk-Targeted Spectral Acceleration for Short Periods)	1.979g	From SEAOC, 2023
S_1 (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.670g	From SEAOC, 2023
F_a (per Table 1613.2.3(1))	1.000	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))	1.700	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{MS} for Site Class D [Note: $S_{MS} = F_a S_s$]	1.979g	-
S_{M1} for Site Class D [Note: $S_{M1} = F_v S_1$]	1.139g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{DS} for Site Class D [Note: $S_{DS} = (2/3)S_{MS}$]	1.319g	-
S_{D1} for Site Class D [Note: $S_{D1} = (2/3)S_{M1}$]	0.759g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
C_{RS} (Mapped Risk Coefficient at 0.2 sec)	0.913	ASCE 7 Chapter 22
C_{R1} (Mapped Risk Coefficient at 1 sec)	0.902	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 C_s 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.		

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.83 at a distance of approximately 10.18 km from the site would contribute the most to this ground motion. A deaggregation of the PGA based on a 475-year average return period (Design Earthquake) indicates that an earthquake magnitude of 6.7 at a distance of approximately 12.78 km from the site would contribute the most to this ground motion (USGS, 2014).

Section 1803.5.12 of the 2022 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 0.886g (SEAOC, 2023). The design PGA is equal to 0.591g (2/3 of PGA_M).

2.8 Rippability

In general, excavation for foundations and underground improvements should be achievable with the appropriate equipment.

2.9 Oversized Material

Generation of a surplus of oversized material (material greater than 8 inches in maximum dimension) is generally not anticipated during site grading. However, some oversized material may be encountered, which may result in excavation difficulty for narrow excavations. Recommendations are provided for appropriate handling of oversized materials in Appendix D.

3.0 FINDINGS AND CONCLUSIONS

Based on the results of our geotechnical evaluation, it is our opinion that the proposed site development is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the site design, grading, and construction.

The following is a summary of the primary geotechnical factors which may affect future development of the site.

- In general, our borings indicate that the site contains primarily dense silty sands with varying amounts of gravel and cobble 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). Moisture contents are generally well below optimum.
- From a geotechnical perspective, the existing onsite soils are suitable material for use as general fill, provided that they are relatively free from rocks (larger than 8 inches in maximum dimension), construction debris, and significant organic material.
- The materials encountered in our evaluation were comprised primarily of sands with very low fines content (i.e., silts) and very low moisture content. These soils are considered very susceptible to sloughing when exposed in steep-sided excavations (generally a 1:1 inclination or steeper). Refer to the boring logs in Appendix B.
- Groundwater was not encountered to the maximum explored depth of approximately 50 feet below existing ground surface. Historic high groundwater is on the order of 80 feet below existing grade (CGS, 2001).
- The site is not located within a State of California Seismic Hazard Zone (CGS, 2001) for liquefaction potential. In general, the site will consist of compacted fill over dense to very dense soils. Due to the absence of groundwater the potential for liquefaction and dynamic settlement is considered very low.
- The proposed development will likely be subjected to strong seismic ground shaking during its design life from one of the regional faults. The subject site is not located within an Alquist-Priolo Earthquake Fault Zone and no faults were identified on the site during our site evaluation.
- Soils exposed at the proposed foundation level are anticipated to have a "Very Low" expansion potential (EI not exceeding 20). This shall be confirmed at the completion of site earthwork.
- Excavation for foundations and underground improvements should be achievable with the appropriate equipment.
- Sandy, relatively cohesionless soils (less than 15 percent finer than 0.005 millimeters) are present and must be compacted to at least 95 percent relative compaction (per ASTM D1557) per the requirements of the City of Los Angeles. Contractors should anticipate sandy soils with low fines content are present thereby requiring at least 95 percent relative compaction.
- Preliminary field percolation tests resulted in measured infiltration rates of 46.6 and 134.4 inches per hour. These field percolation rates do not include any reduction factors (e.g., RF_t , Site Variability, and Long-Term Siltation).

4.0 RECOMMENDATIONS

The following recommendations are to be considered preliminary and should be confirmed upon completion of 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 City. It is the responsibility of the builder to ensure these recommendations are provided to the appropriate parties.

It should be noted that the following geotechnical recommendations are intended to provide sufficient information to develop the site in general accordance with the 2022 California Building Code (CBC)/current City of Los Angeles Building Code requirements. With regard to the potential 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 "the 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 although our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions, they 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 exposed conditions.

4.1 Site Earthwork

We anticipate that earthwork at the site will consist of required earthwork removals, foundation construction and utility line construction and backfill. We recommend that earthwork onsite be performed in accordance with the following recommendations, the current City of Los Angeles Building Code and 2022 CBC and the General Earthwork and Grading Specifications included in Appendix D. In case of conflict, the following recommendations shall supersede previous recommendations and those included as part of Appendix D.

4.1.1 Site Preparation

Prior to grading of areas to receive structural fill, engineered structures or improvements should be demolished and the area should be cleared of existing vegetation (shrubs, trees, grass, etc.), surface obstructions, existing debris and potentially compressible or otherwise unsuitable material. Debris should be removed and properly disposed of off-site. Holes resulting from the removal of buried obstructions, which extend below proposed removal bottoms, should be replaced with suitable compacted fill material. Any

abandoned utility lines associated backfill should be completely removed and replaced with properly compacted fill.

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 Controlled Low Strength Material (CLSM) meeting the requirements of the City of Los Angeles. 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 Depths and Limits

In order to provide a relatively uniform bearing condition for the planned improvements, upper loose/compressible soils are to be removed and replaced as properly compacted fills. For preliminary planning purposes, the depth of required removals may be estimated as indicated below. It should be noted that updated recommendations may be required based on changes to building layouts and/or structural loads or additional field evaluation.

Per the requirements of the City of Los Angeles, footings cannot derive support from undocumented fills. Specifically, soils located within a 1:1 projection of the bottom of footings must be compacted fill or competent natural ground. The following recommendations are in consideration of this requirement. However, if conditions encountered during site grading are different than anticipated, adjustments will be made to ensure that the City requirements are met.

Building Structures: Removals should extend a minimum depth of 5 feet below existing grade or 2 feet below the proposed footings, whichever is greater. In general, the envelope for removals should extend laterally a minimum horizontal distance of 5 feet beyond the edges of the proposed improvements. Deeper removals may be required if undocumented fill soils or otherwise unsuitable conditions are encountered.

Retaining and Free-Standing Walls: Removals should extend a minimum of 3 feet below existing grade, or 2 feet below proposed footings, whichever is deeper. In general, the envelope for over-excavation should extend laterally a minimum horizontal distance of 2 feet beyond the edges of the footings. Deeper removals may be required if undocumented fill soils are encountered.

Pavement and Hardscape Areas: Removals should extend to a depth of at least 2 feet below the existing grade. Removals in any design cut areas of the pavement may be reduced by the depth of the design cut but should not be less than 1-foot below the finished subgrade (i.e., below planned aggregate base/asphalt concrete). In general, the envelope for removals should extend laterally a minimum lateral distance of 2 feet beyond the edges of the proposed 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 including localized areas of undocumented fill. The actual depths and lateral extents of grading will be determined by the geotechnical consultant, based on subsurface conditions encountered during grading. Removal areas should be accurately staked in the field by the Project Surveyor.

4.1.3 Temporary Excavations

Based on our field investigation, the majority of site soils are anticipated to be OSHA Type "C" soils (refer to the attached boring logs). 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.

Sandy soils are present and should be considered susceptible to sloughing. Raveling of the sandy soils should be anticipated for temporary excavations/slopes. Flatter slope inclinations should be considered if raveling cannot be tolerated. The exposed slope surface may be kept surficially moist (but not saturated) during construction to reduce (not eliminate) potential sloughing.

The potential for impacting the existing improvements and adjacent properties may be reduced by performing excavations using narrow "A-B-C" slot cuts. This should be further evaluated based on the proposed grading plan. Protection of existing improvements during grading is the responsibility of the contractor.

Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a distance equivalent to a 1:1 projection from the bottom of the excavation, or 5 feet whichever is greater. The contractor will be responsible for providing the "competent person" required by Cal/OSHA standards to evaluate soil conditions. Close coordination with the geotechnical engineer should be maintained to facilitate construction while providing safe excavations. Excavation safety is the responsibility of the contractor. 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.

4.1.4 Removal Bottoms and Subgrade Preparation

In general, removal bottom areas and any areas to receive compacted fill should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition, and re-compacted per project recommendations. Significant moisture conditioning should be anticipated for removal bottoms.

Removal bottoms and areas to receive fill should be observed and accepted by the geotechnical consultant prior to 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). Moisture conditioning of site soils should be anticipated, as outlined in the section below.

Any required 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.

From a geotechnical viewpoint, any required import soils should consist of clean, relatively granular soils of Very Low expansion potential (expansion index 20 or less based on ASTM D4829) and no particles larger than 3 inches in greatest dimension. Source samples of planned importation should be provided to the geotechnical consultant for laboratory testing a minimum of 3 working days prior to any planned importation for required laboratory testing.

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) or Caltrans Class 2 aggregate base.

4.1.6 Fill Placement and Compaction

Sandy, relatively cohesionless soils (less than 15 percent finer than 0.005 millimeters) fill soils should be compacted to at least 95 percent relative compaction (per ASTM D1557) per the requirements of the City of Los Angeles. Soils with a higher fines content (greater than 15 percent finer than 0.005 millimeters) should be compacted to at least 90 percent relative compaction (per ASTM D1557). Contractor should anticipate sandy soils with low fines content are significantly present thereby requiring at least 95 percent relative compaction. Soils should be compacted near or within about 2 percent over optimum moisture content. The moisture content of soils in the upper approximate 5 feet is well below optimum. Significant moisture conditioning of site soils should be anticipated in order to achieve the required degree of compaction. 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 loose 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 the geotechnical consultant. Oversized material as previously defined should be removed from site fills.

Fill placed on any slopes greater than 5:1 (horizontal to vertical) should be properly keyed and benched into firm and competent soils as it is placed in lifts.

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 per ASTM D1557 at or slightly above optimum moisture content unless it contains cohesionless soils (less than 15 percent finer than 0.005 millimeters). Contractor should anticipate sandy subgrade soils with low fines content thereby requiring at least 95 percent relative compaction.

4.1.7 Trench and Retaining Wall Backfill and Compaction

Bedding material used within the pipe zone should conform to the requirements of the current Greenbook and the pipe manufacturer. Where applicable, sand having a sand equivalent (SE) of 20 or greater (per Caltrans Test Method [CTM] 217) may be used to bed and shade the pipes within the bedding zone. Sand backfill should be densified by jetting or flooding and then tamped to ensure adequate compaction. Bedding sand should be from a natural source, manufactured sand from recycled material is not suitable for jetting. 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 greater than 6 inches in maximum dimension, construction debris and organic material. Trench backfill should be compacted in uniform lifts (as outlined above in Section "Material for Fill") by mechanical means to project recommendations (as outlined above in Section "Placement and Compaction of Fills).

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, typically CLSM (sand-cement slurry) may be substituted for compacted backfill. CLSM must be the requirements of the City of Los Angeles. CLSM placed near the surface within landscape areas should be evaluated for potential impacts on planned improvements.

Any required retaining wall backfill should consist of sandy soils as defined in the above section "Material for Fill." The limits of select sandy backfill should extend at minimum $\frac{1}{2}$ the height of the retaining wall or the width of the heel (if applicable), whichever is greater (Refer to Figure 3). Retaining wall backfill soils should be compacted in relatively uniform thin lifts to the applicable minimum relative compaction depending on the soil type (refer to above Section "Placement and Compaction of Fills"). Jetting or flooding of retaining wall backfill materials is not permitted.

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

Allowance in the earthwork volumes budget should be made for an estimated 5 to 15 percent reduction in volume of the upper approximate 5 feet of site soils. It should be

stressed that these values are only estimates and that an actual shrinkage factor would be extremely difficult to predetermine. Subsidence due to earthwork equipment is expected to be on the order of 0.1-foot. These values are estimates only and exclude losses due to removal of any vegetation or debris. The effective shrinkage of onsite soils will depend primarily on the type of compaction equipment and method of compaction used onsite by the contractor.

4.2 Preliminary Foundation Design Parameters

The following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of project plans (i.e., foundation, grading, etc.) as well as completion of earthwork. Please note that foundation recommendations are based on estimated structural loads. Increase of structural loads may require revision of the provided foundation recommendations and parameters and/or revised remedial recommendations.

Based on preliminary laboratory testing, site soils are anticipated to be of Very Low expansion potential (EI of 20 or less per ASTM D4829). However, this must be verified based on as-graded conditions. Recommended soil bearing and estimated static settlement are provided in Section 4.3.

4.2.1 Slab Design and Construction

Slabs are to be supported on compacted fill soils properly prepared in accordance with the recommendations provided in this report. Actual slab reinforcement and thickness should be determined by the structural engineer based on the imposed loading.

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:

$$k = 150 \times [(B+1)/2B]^2$$

k = modulus of vertical subgrade reaction, pounds per cubic inch (pci)

B = foundation width (feet)

It is recommended that subgrade soils below slabs be moisture conditioned in order to maintain the recommended moisture content up to the time of concrete placement. The recommended moisture content of the slab subgrade soils should be between optimum moisture content and approximately 2 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.

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 contractor. The selection and thickness of sand is not a geotechnical engineering issue and is therefore outside our purview.

4.2.2 Shallow Foundation Maintenance

The geotechnical parameters provided herein assume that if the areas adjacent to the foundation are planted and irrigated, these areas will be designed with proper drainage and adequately maintained so that ponding, which causes significant moisture changes below the foundation, does not occur. Our recommendations do not account for excessive irrigation and/or incorrect landscape design. Plants should only be provided with sufficient irrigation for life and not overwatered to saturate subgrade soils. Sunken planters placed adjacent to the foundation should either be designed with an efficient drainage system or liners to prevent moisture infiltration below the foundation. Some lifting of the perimeter foundation beam should be expected even with properly constructed planters.

In addition to the factors mentioned above, future owners/property management personnel should be made aware of the potential negative influences of trees and/or other large vegetation. Roots that extend near the vicinity of foundations can cause distress to foundations. Future owners (and the owner's landscape architect) should not plant trees/large shrubs closer to the foundations than a distance equal to half the mature height of the tree or 20 feet, whichever is more conservative unless specifically provided with root barriers to prevent root growth below the building foundation.

It is the owner's responsibility to perform periodic maintenance during hot and dry periods to ensure that adequate watering has been provided to keep soil from separating or pulling back from the foundation. Future owners and property management personnel should be informed and educated regarding the importance of maintaining a constant level of soil-moisture. The owners should be made aware of the potential negative consequences of both excessive watering, as well as allowing potentially expansive soils to become too dry. Expansive soils can undergo shrinkage during drying, and swelling during the rainy winter season, or when irrigation is resumed. This can result in distress to building structures and hardscape improvements. These recommendations should be provided to future owners and property management personnel.

4.3 Soil Bearing and Lateral Resistance

Provided our earthwork recommendations are implemented, an allowable soil bearing pressure of 1,500 pounds per square foot (psf) may be used for the design of footings having a minimum width of 18 inches and minimum embedment of 12 inches below lowest adjacent ground surface.

This value may be increased by 500 psf for each additional foot of embedment and by 300 psf for each additional foot of foundation width to a maximum value of 3,000 psf. These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5 horizontal feet to 1-foot vertical) conditions only. Bearing values indicated are for total dead loads and frequently applied live loads and may be increased by $\frac{1}{3}$ for short duration loading (i.e., wind or seismic loads). The increase of bearing capacity is based on a reduced factor of safety (seismic factor of safety equal to three-fourths of the static factor of safety) for short duration loading.

Soil settlement is a function of footing dimensions and applied soil bearing pressure. In utilizing the above-mentioned allowable bearing capacity, assumed structural loads, 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 settlement should be anticipated between nearby columns or walls where a large differential loading condition exists. Settlement estimates should be evaluated by LGC Geotechnical when foundation plans are available.

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 0.25 (based on a factor of safety of 1.5) may be assumed with dead-load forces. An allowable passive lateral earth pressure of 250 pcf to a maximum of 2,500 psf may be used for lateral resistance for properly compacted fill and suitable native soils. This allowable passive pressure may be increased to 340 pcf to a maximum of 3,400 for short-duration seismic loading. For isolated pole footings (for items such as a deck, trellis, etc.) generally spaced a minimum of 3 pile diameters on-center, an allowable passive pressure of 500 pcf may be used for passive resistance. The provided passive pressure is based on an arching factor of 2 (e.g., 250 pcf x 2) and should be limited to a maximum of 10 times the value provided above (e.g., 500 pcf to a maximum of 5,000 psf). This value may be increased by one-third for short-duration seismic loading. These provided passive pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. 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 concrete. The provided allowable passive pressure is based on a static and seismic factor of safety of 1.5 and 1.1, respectively.

4.4 Lateral Earth Pressures for Retaining Walls

Retaining walls of any significant height are not anticipated. The following may be used for design any minor 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 3 on the following page for approved select 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). The wall designer should clearly indicate on the retaining wall plans the required sandy soil backfill criteria.

TABLE 3

Lateral Earth Pressures – Sandy Backfill

Conditions	Equivalent Fluid Unit Weight (pcf)
	Level Backfill
	Approved Soils
Active	55
At-Rest	60

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. This would include 90-degree corners of retaining walls. Such walls should be designed for “at-rest.” 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 basement/retaining wall designer. The amount of surcharge loading on a proposed retaining wall structure is primarily a function of the distance, magnitude and lateral extents of the surcharge loading and should be evaluated on a case-by-case basis. 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 wall. In addition to the recommended lateral 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 0.5 and 0.46 may be used for at-rest and active conditions, respectively for a level backfill. The vertical traffic surcharge may be determined by the structural designer. The structural designer should contact the geotechnical consultant 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 26 pcf for a level backfill condition. This seismic increment is based on a K_h equal to 0.295 using the City of Los Angeles requirement of computing K_h as one-half of two-thirds of the PGA_M . This increment should be applied in addition to the provided static lateral earth pressure using a 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 2022 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) and the City of Los Angeles requirements.

Soil bearing and lateral resistance (friction coefficient and passive resistance) are provided in Section 4.3. 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.

4.5 Preliminary Pavement Sections

The following provisional minimum asphalt concrete (AC) pavement sections are provided in Table 4 for Traffic Indices (TI) of 6 and 7. 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. Determination of the Traffic Index (TI) is not the purview of the geotechnical consultant. Final pavement sections should be confirmed by the project civil/transportation engineer based upon the final design Traffic Index. If requested, LGC Geotechnical will provide sections for alternate TI values.

TABLE 4

Asphalt Concrete Pavement Section Options

Assumed Traffic Index	≤ 6	7
R -Value Subgrade	50	50
AC Thickness	4.0 inches	4.0 inches
Aggregate Base Thickness	4.0 inches	5.0 inches

The provided preliminary Portland Cement concrete pavement section is based on the guidelines of the American Concrete Institute (ACI 330R-08). For the final design section, we recommend a traffic study be performed as LGC Geotechnical does not perform traffic engineering. Traffic study should include the design vehicle (number of axles and load per axle) and estimated number of daily repetitions/trips. Based on an assumed Traffic Category C with an assumed Average Daily Truck Traffic (ADTT) of 10, we recommend a preliminary section of a minimum of 5.5 inches of concrete over 4 inches of compacted aggregate base over compacted subgrade. The concrete should have a minimum compressive strength of 4,000 psi and a minimum flexural strength of 550 psi at the time the pavement is subjected to traffic. Steel reinforcement is not required (ACI, 2013). This pavement section assumes 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 not exceeding 10 feet in each direction. Decreasing the spacing of these joints will further reduce, but not eliminate the potential for unsightly cracking. Preliminary pavement section is based on a 20-year design. Truck loading is defined as one 16-kip axle and two 32-kip tandem axles (80 kips). Alternate section(s) may be provided based on anticipated specific traffic loadings and repetitions provided by others. LGC Geotechnical does not perform traffic engineering and determination of traffic loading is not the purview of the geotechnical consultant.

The thicknesses shown are for minimum 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.

Earthwork recommendations regarding aggregate base and subgrade are provided in the previous section "Site Earthwork" and the related sub-sections of this report.

4.6 Soil Corrosivity

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 indicated soluble sulfate contents less than approximately 0.02 percent, a chloride content of 60 ppm, pH of 6.7, and a minimum resistivity of 7,350 ohm-centimeters. 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 laboratory sulfate test results, the near-surface soils have an exposure class of "S0" per ACI 318-14, Table 19.3.1.1 with respect to sulfates. This must be verified based on as-graded conditions.

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 in Table 5 on the following page. These guidelines will reduce the potential for irregular cracking and promote cracking along control 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 for Very Low Expansion Potential

	Flatwork	City Sidewalk Curb and Gutters
Minimum Thickness (in.)	4 (full)	City/Agency Standard
Presoaking	Wet down prior to placing	City/Agency Standard
Reinforcement	No. 3 at 24 inches on centers	City/Agency Standard
Crack Control Joints	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	City/Agency Standard
Maximum Joint Spacing	6 feet	City/Agency Standard
Aggregate Base Thickness (in.)	—	City/Agency Standard

4.8 Surface Drainage and Landscaping

4.8.1 Precise Grading

From a geotechnical perspective, we recommend that compacted finished grade soils adjacent to proposed 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 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.

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.

4.8.2 Landscaping

Planters adjacent to a building or structure should be avoided wherever possible or be properly designed (e.g., lined with a membrane), 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. Owners and property management personnel should also be made aware that excessive irrigation of neighboring properties can cause seepage and moisture conditions. Owners and property management personnel should be furnished with these recommendations communicating the importance of maintaining positive drainage away from structures, towards streets, when they design their improvements.

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.

4.9 Subsurface Water Infiltration

Recent regulatory changes have occurred that mandate that storm water be infiltrated below grade rather than collected in a conventional storm drain system. It should be noted that collecting and concentrating surface water for the purpose of intentionally infiltrating it below grade, conflicts with the geotechnical engineering objective of directing surface water away from slopes, structures and other improvements. The geotechnical stability and integrity of a site is reliant upon appropriately handling surface water. In general, we do not recommend that surface water be intentionally infiltrated into the subsurface soils.

If it is determined that 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 slopes or near settlement sensitive existing/proposed improvements. Contamination and environmental suitability of the site for infiltration is not the purview of the geotechnical consultant and should be evaluated by others. LGC Geotechnical only addressed the geotechnical issues associated with stormwater infiltration.

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 well 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 different than the infiltration source.

Based on the results of our preliminary field infiltration testing the measured (no reduction factors) infiltration rates of 46.6 and 134.4 inches per hour (refer to Table 1). These infiltration rates are quite high. We recommend that the lower value (46.6 inches per hour) be considered for preliminary design. Per the County of Los Angeles (2021), the design infiltration rate is determined by dividing the measured infiltration rate by reduction factors for test procedure (RF_t), site variability (RF_v) and long-term siltation plugging and maintenance (RF_s). 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 6 below. The design infiltration rate is the measured infiltration rate divided by the total added reduction factors ($RF_t + RF_v + RF_s$).

TABLE 6

Reduction Factors Applied to Measured Infiltration Rate

Correction Factors	Reduction Factor
Test procedure, RF_t	1
Site variability, number of tests, etc., RF_v	1
Long-term siltation plugging and maintenance, RF_s	Per Infiltration System Designer
Total Reduction Factor, $RF = RF_t + RF_v + RF_s$	TBD

The following should be considered in the design of the required infiltration system:

- An adequate setback distance between the proposed infiltration system and adjacent private property should be maintained.
- An adequate setback distance between the proposed infiltration system and proposed building foundations should be maintained.
- The contractor should carefully evaluate the onsite geotechnical conditions as it relates to selecting an appropriate construction technique (including, but not limited to, auger type, construction sequence, etc.) for the construction of the infiltration system and any required temporary shoring system. Difficult drilling should be anticipated. Refer to Appendix B.
- Subsurface soils contain sands with low moisture contents and gravels which are considered very susceptible to caving thereby caving of drilled holes should be anticipated. No borehole should be left open overnight. The contractor should anticipate that any borehole left open for any extended period of time will likely experience additional caving and also potential perched groundwater conditions.

LGC Geotechnical should be provided with updated details for the infiltration system when available for geotechnical review.

4.10 Pre-Construction Documentation and Construction Monitoring

It is recommended that a program of documentation and monitoring be devised and put into practice before the onset of any groundwork. LGC Geotechnical can perform these services at your request. This should include, but not necessarily be limited to, detailed documentation of the existing improvements, buildings, and utilities around the area of proposed excavation, with particular attention to any distress that is already present prior to the start of work. At the completion of construction, we recommend that the adjacent properties be re-documented to confirm their condition after potentially damaging activities are completed. In the event of future claims, any post-construction damage may be attributed to other causes.

4.11 Geotechnical Plan Review

Project plans (e.g., grading, foundation, etc.) and any other improvement plans, and final project drawings should be reviewed by this office prior to construction to verify that our geotechnical recommendations, provided herein, have been appropriately incorporated. Additional or modified geotechnical recommendations may be required based on the proposed design.

4.12 Geotechnical Observation and Testing During Construction

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 are required per Section 1705 of the 2022 CBC.

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

- During grading (removal bottoms, fill placement, etc.);
- During utility trench backfill and compaction;
- During retaining wall subdrain placement and backfill and compaction (if applicable);
- Preparation of subgrade and placement of aggregate base;
- After building and wall footing excavation and prior to placing reinforcement and/or concrete; 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 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. Therefore, 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.

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, and should not be relied upon after a period of 3 years.

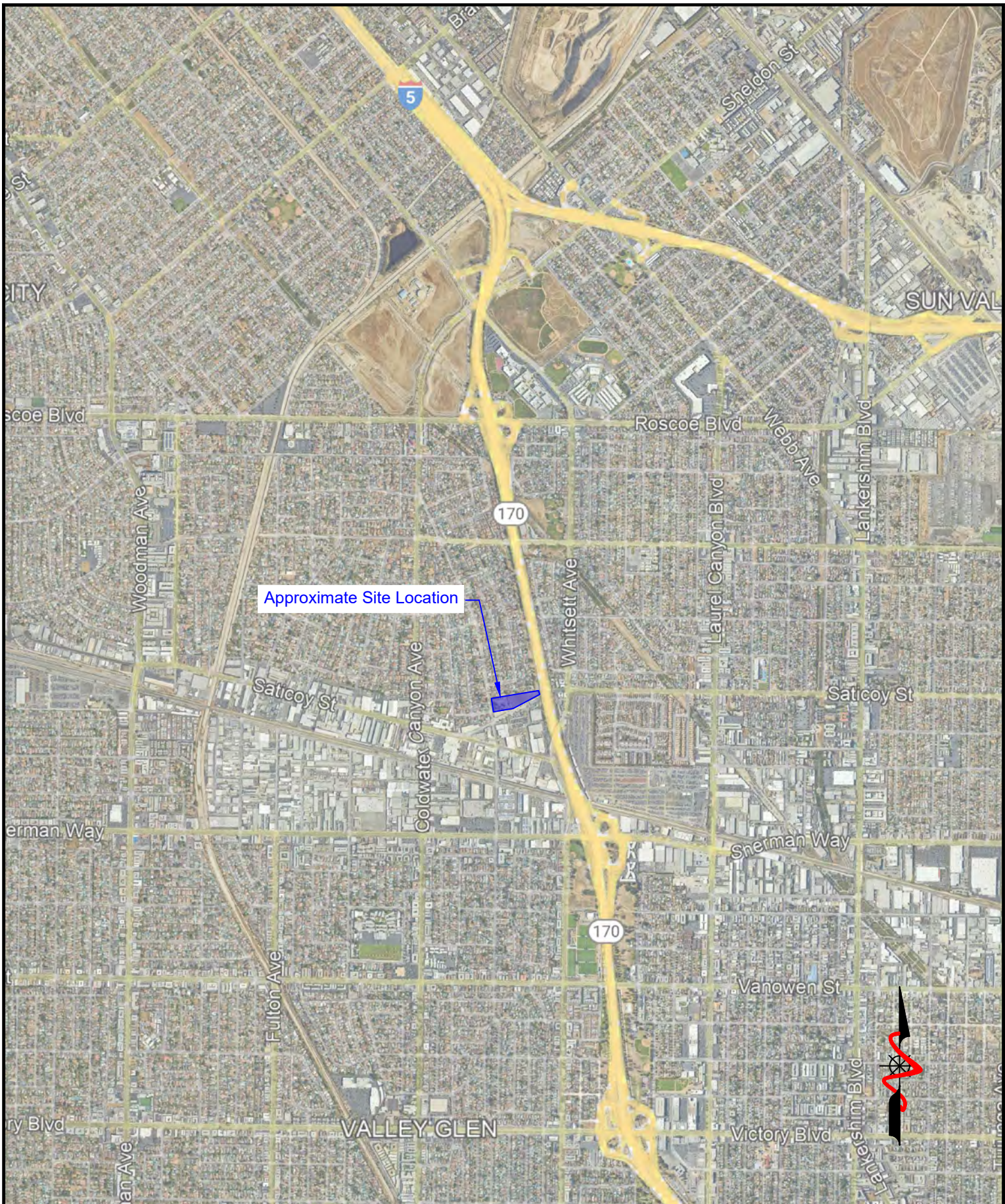
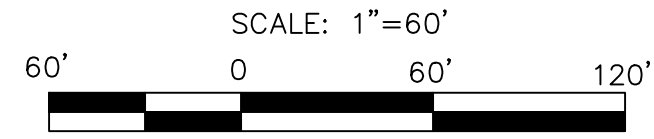
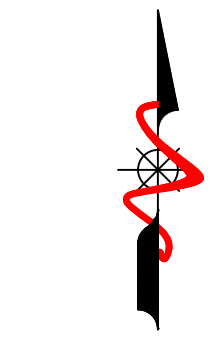
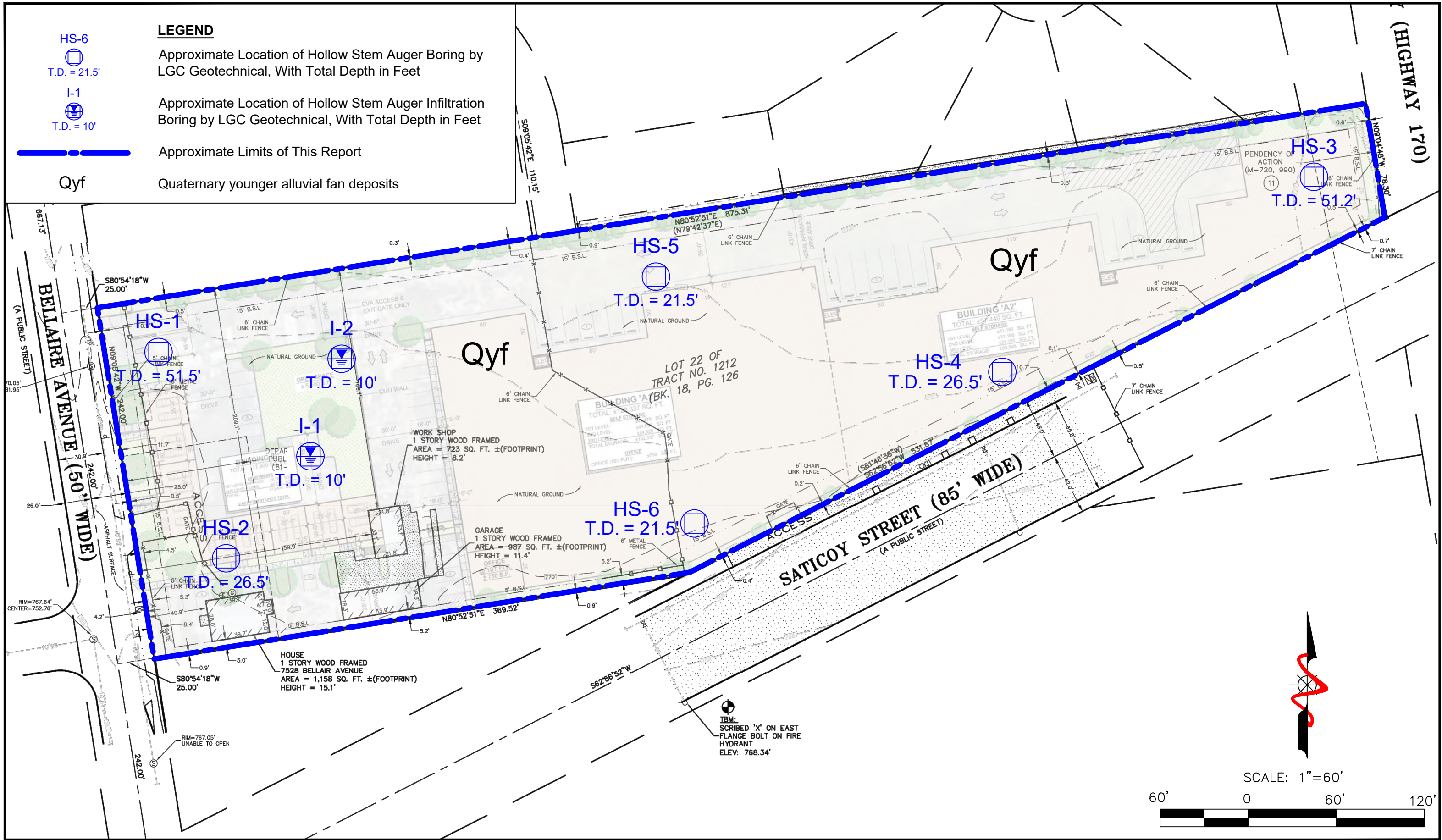


FIGURE 1
Site Location Map

PROJECT NAME	Trojan - 7528 N Bellaire
PROJECT NO.	23001-01
ENG. / GEOL.	BTZ / KBC
SCALE	Not to Scale
DATE	August 2023




LGC Geotechnical, Inc.
 131 Calle Iglesia, Ste. 200
 San Clemente, CA 92672
 TEL (949) 369-6141 FAX (949) 369-6142

FIGURE 2
Geotechnical Map

PROJECT NAME	Trojan - 7528 N Bellaire
PROJECT NO.	23001-01
ENG. / GEOL.	BTZ/KBC
SCALE	1" = 60'
DATE	August 2023

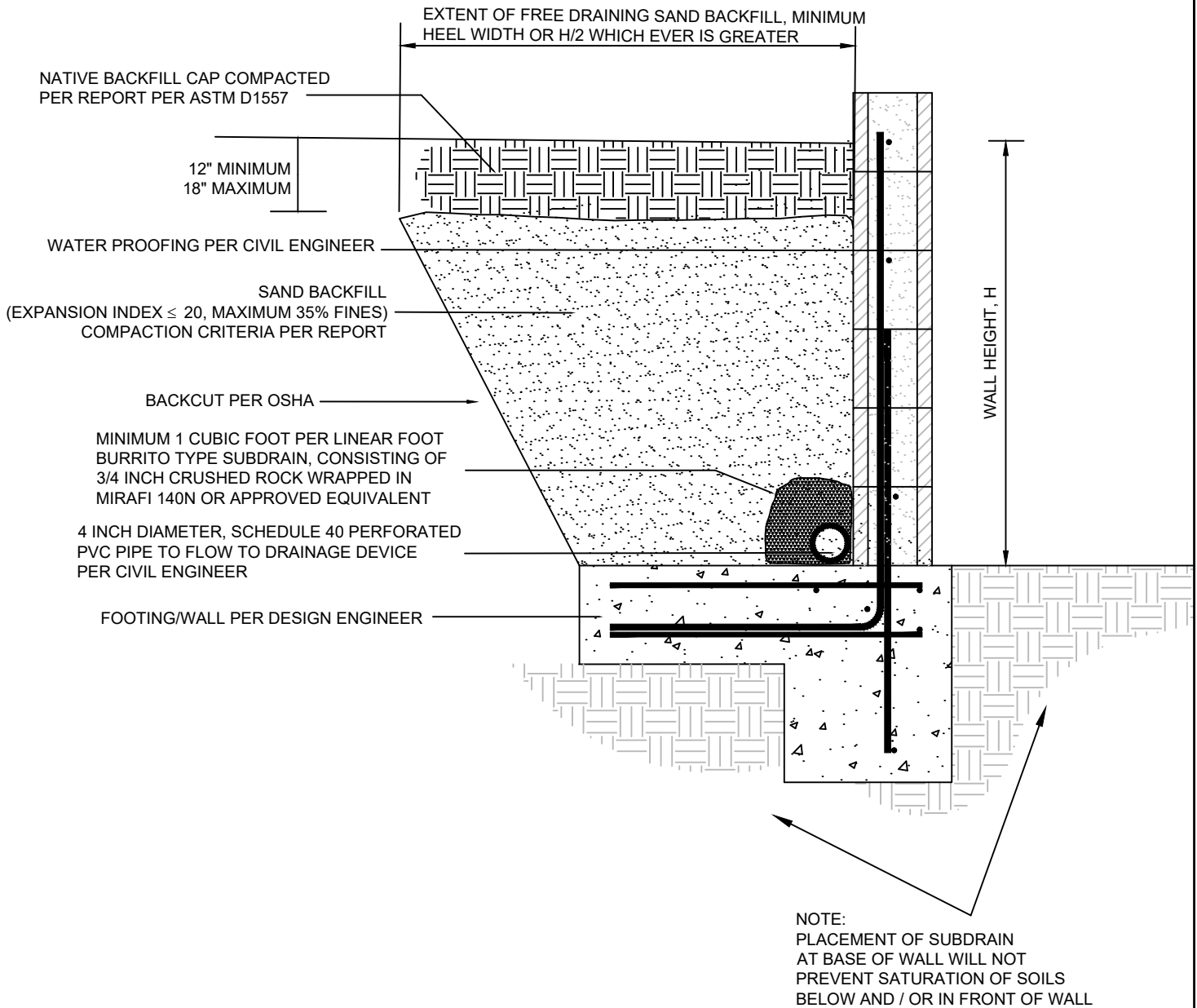


FIGURE 3
Recommended
Retaining Wall
Backfill Detail

PROJECT NAME	7528 N. Bellair
PROJECT NO.	23001-01
ENG./GEOL.	BTZ/KBC
SCALE	Not to Scale
DATE	August 2023



Appendix A
References

APPENDIX A

References

- American Concrete Institute, 2013, Guide for the Design and Construction of Concrete Parking Lots (ACI 330R-08), fifteenth printing, November 2013.
- _____, 2019, American Concrete Institute (ACI), 2019, Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19), June 2019.
- American Society of Civil Engineers (ASCE), 2017, Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-16, 2017.
- _____, 2018, Standard 7-16, Minimum Design Loads for Buildings and Associated Criteria for Buildings and Other Structures, Supplement 1, effective: December 12, 2018.
- American Society for Testing and Materials (ASTM), Volume 04.08 Soil and Rock (I):D420 – D5876.
- California Building Standards Commission, 2022, California Building Code, California Code of Regulations Title 24, Volumes 1 and 2, dated July 2022.
- California Geological Survey (CGS), (Previously California Division of Mines and Geology [CDMG]), 1997, State of California Seismic Hazard Zone Report for the Van Nuys 7.5-Minute Quadrangle, Los Angeles County, California, Seismic Hazard Zone Report 008, revised 2001.
- _____, 1998, State of California Seismic Hazard Zones, Van Nuys Quadrangle, Official Map, scale: 1:24,000, Release Date: February 1, 1998.
- _____, 2018, Earthquake Fault Zones, Special Publication 42, Revised 2018.
- _____, 2008, California Geological Society Special Publication 117A: Guidelines for Evaluating and Mitigating Seismic Hazards in California.
- Caltrans, 2021, Corrosion Guidelines, Version 3.2, dated May 2021.
- County of Los Angeles, 2017, Administrative Manual, Department of Public Works, Geotechnical and Materials Engineering Division, Guidelines for Design, Investigation, and Reporting Low Impact Development Stormwater Infiltration, GS200.2, dated June 30, 2017.
- _____, 2021, Administrative Manual, Department of Public Works, Geotechnical and Materials Engineering Division, Guidelines for Design, Investigation, and Reporting Low Impact Development Stormwater Infiltration, GS200.1, dated June 30, 2021.
- Historic Aerials, 2022, historicareials.com/views, retrieved August 17, 2023.
- Jordan Architects, 2023, Brett Henry Trojan Storage, North Hollywood, CA, Preliminary Site Plan, Scheme E, Job Number 22-831, dated June 15, 2023.

Lew, et al, 2010, Seismic Earth Pressures on Deep Basements, Structural Engineers Association of California (SEAOC) Convention Proceedings.

Structural Engineers Association of California (SEAOC), 2023, Seismic Design Maps, Retrieved July 14, 2023, from <https://seismicmaps.org/>

United States Geological Survey (USGS), 1996, Preliminary Geologic Map of the Van Nuys 7.5' Quadrangle, Southern California, Compiled by R.F. Yerkes, Open File Report 96-86, Scale 1:24,000, dated 1996.

United States Geological Survey (USGS), 2014, Unified Hazard Tool, Dynamic: Conterminous U.S. 2014 (update) (v4.2.0), Retrieved July 14, 2023, from: <https://earthquake.usgs.gov/hazards/interactive/>

Appendix B
Field Exploration Logs and
Field Infiltration Data

Geotechnical Boring Log Borehole HS-1

Date: 7/10/2023	Drilling Company: MR Drilling
Project Name: Trojan - 7528 N Bellaire	Type of Rig: Truck Mounted Rig
Project Number: 23001-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~770' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@ 0' - Dry grass / Dry Weeds	EI SC
			R-1	2 5	98.7	2.4	ML	@ 2.5' - SILT: light olive gray, slightly moist, medium stiff	
765	5	B-1	SPT-1	2 3 4		1.1	SP-SM	@ 5' - Poorly-Graded SAND with Silt: light olive gray, dry, loose	SA
			R-2	8 10 15	109.5	0.6	SW	@ 7.5' - Well-Graded SAND: light gray, dry, medium dense	
760	10		SPT-2	5 9 11		1.0	SP-SM	@ 10' - Poorly-Graded SAND with Silt: light olive gray, dry, medium dense	#200
755	15		R-3	12 18 24	108.1	1.1	SM	@ 15' - Silty SAND: light olive gray, dry, dense	
750	20		SPT-3	7 10 16		1.0	SP-SM	@ 20' - Poorly-Graded SAND with Silt and Gravel: light olive gray, dry, dense	#200
745	25		R-4	13 20 26	115.8	1.7	SM	@ 25' - Silty SAND: olive gray, dry, dense	
740	30								

Last Edited: 7/17/2023





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. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES: B BULK SAMPLE R RING SAMPLE (CA Modified Sampler) G GRAB SAMPLE SPT STANDARD PENETRATION TEST SAMPLE GROUNDWATER TABLE	TEST TYPES: DS DIRECT SHEAR MD MAXIMUM DENSITY SA SIEVE ANALYSIS S&H SIEVE AND HYDROMETER EI EXPANSION INDEX CN CONSOLIDATION CR CORROSION AL ATTERBERG LIMITS CO COLLAPSE/SWELL RV R-VALUE #200 % PASSING # 200 SIEVE
--	--

Geotechnical Boring Log Borehole HS-1

Date: 7/10/2023	Drilling Company: MR Drilling
Project Name: Trojan - 7528 N Bellaire	Type of Rig: Truck Mounted Rig
Project Number: 23001-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~770' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		SPT-4	12 17 21		1.0	SP	@ 30' - Poorly-Graded SAND: light olive gray, dry, dense	
735	35		R-5	17 28 50/5"	128.4	0.7	SW	@ 35' - Well-Graded SAND with Gravel: light gray, dry, very dense	
730	40		SPT-5	16 17 20		2.0	SP	@ 40' - Poorly-Graded SAND: pale olive, slightly moist, dense	
725	45		R-6	23 17 37	112.6	1.2		@ 45' - Poorly-Graded SAND: olive gray, dry, very dense	
720	50		SPT-6	44 37 47		1.5		@ 50' - Poorly-Graded SAND with Gravel: olive gray, dry, very dense	
715	55							Total Depth = 51.5' Groundwater Not Encountered Caving: Hole measured approximately 10' deep after Removal of the Augers Backfilled with Cuttings on 7/10/2023	
710	60								

	<p>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. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.</p>	<p>SAMPLE TYPES: B BULK SAMPLE R RING SAMPLE (CA Modified Sampler) G GRAB SAMPLE SPT STANDARD PENETRATION TEST SAMPLE</p> <p> GROUNDWATER TABLE</p>	<p>TEST TYPES: DS DIRECT SHEAR MD MAXIMUM DENSITY SA SIEVE ANALYSIS S&H SIEVE AND HYDROMETER EI EXPANSION INDEX CN CONSOLIDATION CR CORROSION AL ATTERBERG LIMITS CO COLLAPSE/SWELL RV R-VALUE #200 % PASSING # 200 SIEVE</p>
---	--	--	---

Geotechnical Boring Log Borehole HS-2

Date: 7/10/2023	Drilling Company: MR Drilling
Project Name: Trojan - 7528 N Bellaire	Type of Rig: Truck Mounted Rig
Project Number: 23001-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~770' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@ 0' - Dry grass / Dry Weeds	S&H MD
			SPT-1	2 2 4		2.2	SP	@ 2.5' - Poorly-Graded SAND: pale olive, slightly moist, loose	
765	5	B-1	R-1	3 6 9	107.6	1.9		@ 5' - Poorly-Graded SAND: light olive brown, dry, medium dense	
			SPT-2	5 8 9		3.1		@ 7.5' - Poorly-Graded SAND with Gravel: olive gray, slightly moist, medium dense	
760	10		R-2	11 20 30	117.9	2.4		@ 10' - Poorly-Graded SAND: light olive brown, slightly moist, dense	CN
			SPT-3	7 11 16		3.9		@ 15' - Poorly-Graded SAND: pale olive, slightly moist, dense	
755	15								
750	20		R-3	14 16 22	125.7	2.8		@ 20' - Poorly-Graded SAND: light olive, slightly moist, dense	
			SPT-4	10 21 32		4.2		@ 25' - Poorly-Graded SAND with Gravel: pale olive, slightly moist, very dense	
								Total Depth = 26.5'	
								Groundwater Not Encountered	
								Caving: Hole measured approximately 10' deep after Removal of the Augers	
740	30							Backfilled with Cuttings on 7/10/2023	



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. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.



SAMPLE TYPES: B BULK SAMPLE R RING SAMPLE (CA Modified Sampler) G GRAB SAMPLE SPT STANDARD PENETRATION TEST SAMPLE GROUNDWATER TABLE	TEST TYPES: DS DIRECT SHEAR MD MAXIMUM DENSITY SA SIEVE ANALYSIS S&H SIEVE AND HYDROMETER EI EXPANSION INDEX CN CONSOLIDATION CR CORROSION AL ATTERBERG LIMITS CO COLLAPSE/SWELL RV R-VALUE #200 % PASSING # 200 SIEVE
--	--

Last Edited: 7/17/2023

Geotechnical Boring Log Borehole HS-3

Date: 7/10/2023	Drilling Company: MR Drilling
Project Name: Trojan - 7528 N Bellaire	Type of Rig: Truck Mounted Rig
Project Number: 23001-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~772' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1



Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@ 0' - Dry Grass / Dry Weeds	EL
770			SPT-1	20 50/5"		3.2	SM	@ 2.5' - Silty SAND: light olive brown, slightly moist, very dense	RV CR SA
	5	B-1	R-1	16 25 38	116.3	2.2	SP	@ 5' - Poorly-Graded SAND with Gravel: olive gray, slightly moist, very dense	
765			SPT-2	7 9 12		4.7		@ 7.5' - Poorly-Graded SAND: pale olive, slightly moist, medium dense	-#200
	10		R-2	8 11 16	104.3	4.0		@ 10' - Poorly-Graded SAND: light olive brown, slightly moist, medium dense	CN
760			SPT-3	15 30 39		2.3	SP-SM	@ 15' - Poorly Graded SAND with Silt and Gravel: light olive gray, slightly moist, very dense	-#200
755			R-3	13 17 21	116.2	3.3	SP	@ 20' - Poorly-Graded SAND: light olive, slightly moist, dense	
750			SPT-4	9 12 16		5.3		@ 25' - Poorly-Graded SAND: olive gray, slightly moist, medium dense	
745									
	30								

	<p style="font-size: small;">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. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.</p>	<p style="font-size: x-small;">SAMPLE TYPES:</p> <p>B BULK SAMPLE R RING SAMPLE (CA Modified Sampler) G GRAB SAMPLE SPT STANDARD PENETRATION TEST SAMPLE</p> <p style="text-align: center;"> GROUNDWATER TABLE</p> <p style="font-size: x-small;">TEST TYPES:</p> <p>DS DIRECT SHEAR MD MAXIMUM DENSITY SA SIEVE ANALYSIS S&H SIEVE AND HYDROMETER EI EXPANSION INDEX CN CONSOLIDATION CR CORROSION AL ATTERBERG LIMITS CO COLLAPSE/SWELL RV R-VALUE -#200 % PASSING # 200 SIEVE</p>
---	--	--

Geotechnical Boring Log Borehole HS-3

Date: 7/10/2023	Drilling Company: MR Drilling
Project Name: Trojan - 7528 N Bellaire	Type of Rig: Truck Mounted Rig
Project Number: 23001-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~772' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
735	30		R-4	18 27 39	119.3	3.3	SP	@ 30' - Poorly-Graded SAND: olive gray, slightly moist, very dense	
730	35		SPT-5	12 17 22		6.0		@ 35' - Poorly-Graded SAND with Gravel: pale olive, moist, very dense	
725	40		R-5	14 23 34	121.1	2.9		@ 40' - Poorly-Graded SAND: light olive, slightly moist, dense	
720	45		SPT-6	18 23 28		3.2		@ 45' - Poorly-Graded SAND: olive gray, slightly moist, very dense	
715	50		R-6	29 30 50/3"	121.2	2.3		@ 50' - Poorly-Graded SAND: light olive gray, slightly moist, very dense	
710	55							Total Depth = 56.2' Groundwater Not Encountered Backfilled with Cuttings on 7/10/2023	
60									

	<p>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. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.</p>	<p>SAMPLE TYPES: B BULK SAMPLE R RING SAMPLE (CA Modified Sampler) G GRAB SAMPLE SPT STANDARD PENETRATION TEST SAMPLE</p> <p> GROUNDWATER TABLE</p>	<p>TEST TYPES: DS DIRECT SHEAR MD MAXIMUM DENSITY SA SIEVE ANALYSIS S&H SIEVE AND HYDROMETER EI EXPANSION INDEX CN CONSOLIDATION CR CORROSION AL ATTERBERG LIMITS CO COLLAPSE/SWELL RV R-VALUE #200 % PASSING # 200 SIEVE</p>
---	--	--	---

Geotechnical Boring Log Borehole HS-4

Date: 7/10/2023	Drilling Company: MR Drilling
Project Name: Trojan - 7528 N Bellaire	Type of Rig: Truck Mounted Rig
Project Number: 23001-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~771' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
770	0							@ 0' - Dry Grass / Dry Weeds	
			R-1	7 7 9	113.6	3.7	SP	@ 2.5' - Poorly-Graded SAND: olive brown, slightly moist, medium dense	
765	5	B-1	SPT-1	5 7 10		2.8		@ 5' - Poorly-Graded SAND: light brown, slightly moist, medium dense	#200
			R-2	6 10 17	104.7	2.7		@ 7.5' - Poorly-Graded SAND: light olive brown, slightly moist, medium dense	
760	10		SPT-2	5 6 7		5.4		@ 10' - Poorly-Graded SAND: light olive brown, slightly moist, medium dense	
755	15		R-3	18 19 21	117.2	7.0		@ 15' - Poorly-Graded SAND: light olive gray, moist, dense	
750	20		SPT-3	10 13 15		6.0		@ 20' - Poorly-Graded SAND: light olive brown, moist, dense	
745	25		R-4	15 25 32	111.4	3.7		@ 25' - Poorly-Graded SAND: olive gray, slightly moist, dense	
	30							Total Depth = 26.5' Groundwater Not Encountered Caving: Hole measured approximately 25' deep after Removal of the Augers Backfilled with Cuttings on 7/10/2023	



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. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES: B BULK SAMPLE R RING SAMPLE (CA Modified Sampler) G GRAB SAMPLE SPT STANDARD PENETRATION TEST SAMPLE GROUNDWATER TABLE	TEST TYPES: DS DIRECT SHEAR MD MAXIMUM DENSITY SA SIEVE ANALYSIS S&H SIEVE AND HYDROMETER EI EXPANSION INDEX CN CONSOLIDATION CR CORROSION AL ATTERBERG LIMITS CO COLLAPSE/SWELL RV R-VALUE #200 % PASSING # 200 SIEVE
--	--

Geotechnical Boring Log Borehole HS-5

Date: 7/10/2023	Drilling Company: MR Drilling
Project Name: Trojan - 7528 N Bellaire	Type of Rig: Truck Mounted Rig
Project Number: 23001-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~772' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@ 0' - Dry Grass / Dry Weeds	
770			SPT-1	4 5 7		2.6	SP-SM	@ 2.5' - Poorly-Graded SAND with Silt: light olive brown, slightly moist, medium dense	#200
	5		R-1	5 7 9	112.5	3.3	SP	@ 5' - Poorly-Graded SAND: light olive gray, slightly moist, medium dense	
765			SPT-2	7 8 10		4.5		@ 7.5' - Poorly-Graded SAND: light olive brown, slightly moist, medium dense	
	10		R-2	10 15 21	109.5	3.5		@ 10' - Poorly-Graded SAND: light olive gray, slightly moist, medium dense	CN
760									
	15		SPT-3	8 13 17		3.4		@ 15' - Poorly-Graded SAND with Gravel: light olive brown, slightly moist, dense	
755									
	20		R-3	12 16 25	117.0	3.4		@ 20' - Poorly-Graded SAND: light olive brown, slightly moist, dense	
750								Total Depth = 21.5' Groundwater Not Encountered Caving: Hole measured approximately 16' deep after Removal of the Augers Backfilled with Cuttings on 7/10/2023	
	25								
745									
	30								



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. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:
 B BULK SAMPLE
 R RING SAMPLE (CA Modified Sampler)
 G GRAB SAMPLE
 SPT STANDARD PENETRATION TEST SAMPLE

GROUNDWATER TABLE

TEST TYPES:
 DS DIRECT SHEAR
 MD MAXIMUM DENSITY
 SA SIEVE ANALYSIS
 S&H SIEVE AND HYDROMETER
 EI EXPANSION INDEX
 CN CONSOLIDATION
 CR CORROSION
 AL ATTERBERG LIMITS
 CO COLLAPSE/SWELL
 RV R-VALUE
 #200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-6

Date: 7/10/2023	Drilling Company: MR Drilling
Project Name: Trojan - 7528 N Bellaire	Type of Rig: Truck Mounted Rig
Project Number: 23001-01	Drop: 30" Hole Diameter: 6"
Elevation of Top of Hole: ~770' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@ 0' - Dry Grass / Dry Weeds	
			R-1	6 7 10	105.5	2.7	SP	@ 2.5' - Poorly-Graded SAND: light olive, slightly moist, medium dense	
765	5	B-1	SPT-1	4 6 9		3.0		@ 5' - Poorly-Graded SAND: light olive brown, slightly moist, medium dense	
			R-2	11 13 17	110.8	2.6		@ 7.5' - Poorly-Graded SAND: light gray, slightly moist, medium dense	
760	10		SPT-2	7 15 21		2.9		@ 10' - Poorly-Graded SAND with Gravel: light olive brown, slightly moist, dense	
755	15		R-3	18 20 23	118.2	3.6		@ 15' - Poorly-Graded SAND: light olive brown, slightly moist, dense	CN
750	20		SPT-3	10 13 16		3.9	SP	@ 20' - Poorly-Graded SAND with Gravel: light olive brown, slightly moist, dense	
								Total Depth = 21.5' Groundwater Not Encountered Caving: Hole measured approximately 19' deep after Removal of the Augers Backfilled with Cuttings on 7/10/2023	
745	25								
740	30								



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. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES: B BULK SAMPLE R RING SAMPLE (CA Modified Sampler) G GRAB SAMPLE SPT STANDARD PENETRATION TEST SAMPLE GROUNDWATER TABLE	TEST TYPES: DS DIRECT SHEAR MD MAXIMUM DENSITY SA SIEVE ANALYSIS S&H SIEVE AND HYDROMETER EI EXPANSION INDEX CN CONSOLIDATION CR CORROSION AL ATTERBERG LIMITS CO COLLAPSE/SWELL RV R-VALUE #200 % PASSING # 200 SIEVE
--	--

Last Edited: 7/17/2023

Geotechnical Boring Log Borehole I-1

Date: 7/10/2023	Drilling Company: MR Drilling
Project Name: Trojan - 7528 N Bellaire	Type of Rig: Truck Mounted Rig
Project Number: 23001-01	Drop: 30" Hole Diameter: 8"
Elevation of Top of Hole: ~770' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
765	0							@ 0' - Dry Grass / Dry Weeds	
760	5		SPT-1	6 7 10		4.4	SP	@ 7.5' - Poorly-Graded SAND: pale olive, slightly moist, medium dense	
755	10							Total Depth = 10' Groundwater Not Encountered 3" Pipe with Filter Sock Installed Surrounded by Gravel, and Presoaked on 7/10/23 Backfilled with Cuttings on 7/11/2023	
750	15								
745	20								
740	25								
740	30								



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. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

SAMPLE TYPES:	TEST TYPES:
B BULK SAMPLE	DS DIRECT SHEAR
R RING SAMPLE (CA Modified Sampler)	MD MAXIMUM DENSITY
G GRAB SAMPLE	SA SIEVE ANALYSIS
SPT STANDARD PENETRATION TEST SAMPLE	S&H SIEVE AND HYDROMETER
	EI EXPANSION INDEX
	CN CONSOLIDATION
	CR CORROSION
	AL ATTERBERG LIMITS
GROUNDWATER TABLE	CO COLLAPSE/SWELL
	RV R-VALUE
	#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole I-1

Date: 7/10/2023	Drilling Company: MR Drilling
Project Name: Trojan - 7528 N Bellaire	Type of Rig: Truck Mounted Rig
Project Number: 23001-01	Drop: 30" Hole Diameter: 8"
Elevation of Top of Hole: ~770' MSL	Drive Weight: 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
765	0							@ 0' - Dry Grass / Dry Weeds	
760	5		R-1	10 19 30	110.8	1.9	SP	@ 7.5' - Poorly-Graded SAND: light olive brown, dry, dense	
755	10							Total Depth = 10' Groundwater Not Encountered 3" Pipe with Filter Sock Installed Surrounded by Gravel, and Presoaked on 7/10/23 Backfilled with Cuttings on 7/11/2023	
750	15								
745	20								
740	25								
	30								



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. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

<p>SAMPLE TYPES:</p> <p>B BULK SAMPLE R RING SAMPLE (CA Modified Sampler) G GRAB SAMPLE SPT STANDARD PENETRATION TEST SAMPLE</p> <p> GROUNDWATER TABLE</p>	<p>TEST TYPES:</p> <p>DS DIRECT SHEAR MD MAXIMUM DENSITY SA SIEVE ANALYSIS S&H SIEVE AND HYDROMETER EI EXPANSION INDEX CN CONSOLIDATION CR CORROSION AL ATTERBERG LIMITS CO COLLAPSE/SWELL RV R-VALUE #200 % PASSING # 200 SIEVE</p>
---	---

Infiltration Test Data Sheet

LGC Geotechnical, Inc

131 Calle Iglesia Suite A, San Clemente, CA 92672 tel. (949) 369-6141

Project Name: 7528 N Bellaire
Project Number: 23001-01
Date: 7/11/2023
Location: I-1

Test hole dimensions (if circular)	
Boring Depth (feet)*:	10
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

Test pit dimensions (if rectangular)	
Pit Depth (feet):	_____
Pit Length (feet):	_____
Pit Breadth (feet):	_____

Pre-Soak /Pre-Test

No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Comments
Pre-Test	13:17	13:17	0.5	9	9.6	0.6	Fast

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, ΔD (feet)	Surface Area of Test Section (feet ^2)	Raw Percolation Rate (in/hr)
1	13:19	13:19	0.5	9.00	9.84	0.84	2.44	172.8
2	13:21	13:21	0.5	9.00	9.86	0.86	2.44	176.9
3	13:22	13:22	0.5	9.00	9.75	0.75	2.44	154.3
4	13:24	13:24	0.5	9.00	9.80	0.80	2.44	164.6
5	13:25	13:25	0.5	9.00	9.76	0.76	2.44	156.3
6	13:27	13:27	0.5	9.00	9.73	0.73	2.44	150.2
7	13:29	13:29	0.5	9.00	9.70	0.70	2.44	144.0
8	13:30	13:30	0.5	9.00	9.75	0.75	2.44	154.3
9	13:31	13:31	0.5	9.00	9.69	0.69	2.44	141.9
10	13:32	13:32	0.5	9.00	9.67	0.67	2.44	137.8
11	13:34	13:34	0.5	9.00	9.65	0.65	2.44	133.7
12	13:35	13:35	0.5	9.00	9.64	0.64	2.44	131.7
Measured Infiltration Rate							134.4	

Sketch:

Notes:

Based on Guidelines from: LA County dated 06/2021

Spreadsheet Revised on: 6/22/2023



Infiltration Test Data Sheet

LGC Geotechnical, Inc

131 Calle Iglesia Suite A, San Clemente, CA 92672 tel. (949) 369-6141

Project Name: 7528 N Bellaire
Project Number: 23001-01
Date: 7/11/2023
Location: I-2

Test hole dimensions (if circular)	
Boring Depth (feet)*:	10
Boring Diameter (inches):	8
Pipe Diameter (inches):	3

*measured at time of test

Test pit dimensions (if rectangular)	
Pit Depth (feet):	_____
Pit Length (feet):	_____
Pit Breadth (feet):	_____

Pre-Soak /Pre-Test

No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval (min)	Initial Depth to Water (feet)	Final Depth to Water (feet)	Total Change in Water Level (feet)	Comments
Pre-Test	9:45	9:46	1.0	9	9.67	0.67	

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Δt (min)	Initial Depth to Water, D _o (feet)	Final Depth to Water, D _f (feet)	Change in Water Level, ΔD (feet)	Surface Area of Test Section (feet ^2)	Raw Percolation Rate (in/hr)
1	9:49	9:50	1.0	9.00	9.63	0.63	2.44	64.8
2	9:51	9:52	1.0	9.00	9.70	0.70	2.44	72.0
3	9:53	9:54	1.0	9.00	9.63	0.63	2.44	64.8
4	9:55	9:56	1.0	9.00	9.54	0.54	2.44	55.5
5	9:58	9:59	1.0	9.00	9.52	0.52	2.44	53.5
6	10:00	10:01	1.0	9.00	9.52	0.52	2.44	53.5
7	10:02	10:03	1.0	9.00	9.50	0.50	2.44	51.4
8	10:04	10:05	1.5	9.00	9.59	0.59	2.44	40.5
9	10:06	10:07	1.0	9.00	9.49	0.49	2.44	50.4
10	10:08	10:09	1.0	9.00	9.48	0.48	2.44	49.4
11	10:10	10:11	1.0	9.00	9.45	0.45	2.44	46.3
12	10:13	10:14	1.0	9.00	9.43	0.43	2.44	44.2
Measured Infiltration Rate							46.6	

Sketch:

Notes:

Based on Guidelines from: LA County dated 06/2021

Spreadsheet Revised on: 6/22/2023



Appendix C
Laboratory Test Results

APPENDIX C

Laboratory 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 a table summarizing the test results.

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

Grain Size Distribution/Fines Content: 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 (ASTM D1140). Where applicable, 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 D6913 (sieve) or ASTM D422 (sieve and hydrometer).

Sample Location	Description	% Passing # 200 Sieve
HS-1 @ 5 ft	Sand with Silt	11.6
HS-1 @ 10 ft	Sand with Silt	5.4
HS-1 @ 20 ft	Sand with Silt and Gravel	7.1
HS-2 @ 1-5 ft	Silty Sand	19.7
HS-3 @ 2.5 ft	Silty Sand	13.8
HS-3 @ 7.5 ft	Sand	3.5
HS-3 @ 15 ft	Sand with Silt and Gravel	6.9
HS-4 @ 5 ft	Sand	3.5
HS-5 @ 2.5 ft	Sand with Silt	7.2

Expansion Index: The expansion potential of selected representative samples was evaluated by the Expansion Index Test per ASTM D4829. The results are presented in the table below.

Sample Location	Expansion Index	Expansion Potential*
HS-1 @ 1-5 ft	5	Very Low
HS-3 @ 1-5 ft	0	Very Low

* Per ASTM D4829

APPENDIX C (Cont'd)

Laboratory Test Results

Consolidation: Consolidation tests were performed per ASTM D2435. Samples (2.4 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 provided in this Appendix.

Laboratory Compaction: The maximum dry density and optimum moisture content of typical materials were determined in accordance with ASTM D1557. The results are presented in the table below.

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-2 @ 1-5 ft	Olive Brown Silty Sand	123.5	8.5

R-value Test: R-value test was performed in general accordance with California Test Method 301. The plot is attached.

Sample No.	R-Value
HS-3 @ 1-5 ft	75

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods (CTM 417). The test results are presented in the table below.

Sample Location	Sulfate Content (ppm)	Sulfate Content (%)
HS-1 @ 1-5 ft	99	< 0.01
HS-3 @ 1-5 ft	103	< 0.02

APPENDIX C (Cont'd)

Laboratory Test Results

Chloride Content: Chloride content was tested per CTM 422. The results are presented below.

Sample Location	Chloride Content (ppm)
HS-3 @ 1-5 ft	60

Minimum Resistivity and pH Tests: Minimum resistivity and pH tests were performed in general accordance with CTM 643 and standard geochemical methods. The results are presented in the table below.

Sample Location	pH	Minimum Resistivity (ohms-cm)
HS-3 @ 1-5 ft	6.7	7,350



PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS

ASTM D 6913

Project Name: [N. Bellaire, North Hollywood](#)

Tested By: [G. Bathala](#) Date: [07/21/23](#)

Project No.: [23001-01](#)

Checked By: [J. Ward](#) Date: [08/07/23](#)

Boring No.: [HS-1](#)

Depth (feet): [5.0](#)

Sample No.: [SPT-1](#)

Soil Identification: [Light olive gray poorly-graded sand with silt \(SP-SM\), organic material noted](#)

		Moisture Content of Total Air - Dry Soil	
Container No.:	4-C	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	735.7	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	108.5	Wt. of Container No. _____ (g)	1.0
Dry Wt. of Soil (g)	627.2	Moisture Content (%)	0.0

After Wet Sieve	Container No.	4-C
	Wt. of Dry Soil + Container (g)	676.1
	Wt. of Container (g)	108.5
	Dry Wt. of Soil Retained on # 200 Sieve (g)	567.6

U. S. Sieve Size		Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
(in.)	(mm.)		
1 1/2"	37.5		
1"	25.0		
3/4"	19.0		
1/2"	12.5		
3/8"	9.5	0.0	100.0
#4	4.75	2.2	99.6
#8	2.36	7.3	98.8
#16	1.18	25.4	96.0
#30	0.600	87.3	86.1
#50	0.300	262.0	58.2
#100	0.150	440.1	29.8
#200	0.075	554.4	11.6
PAN			

GRAVEL: **0 %**

SAND: **88 %**

FINES: **12 %**

GROUP SYMBOL: **SP-SM**

Cu = D60/D10 = 4.43

Cc = (D30)²/(D60*D10) = 1.04

Remarks: _____

GRAVEL				SAND				FINES			
COARSE		FINE		COARSE	MEDIUM	FINE		SILT		CLAY	

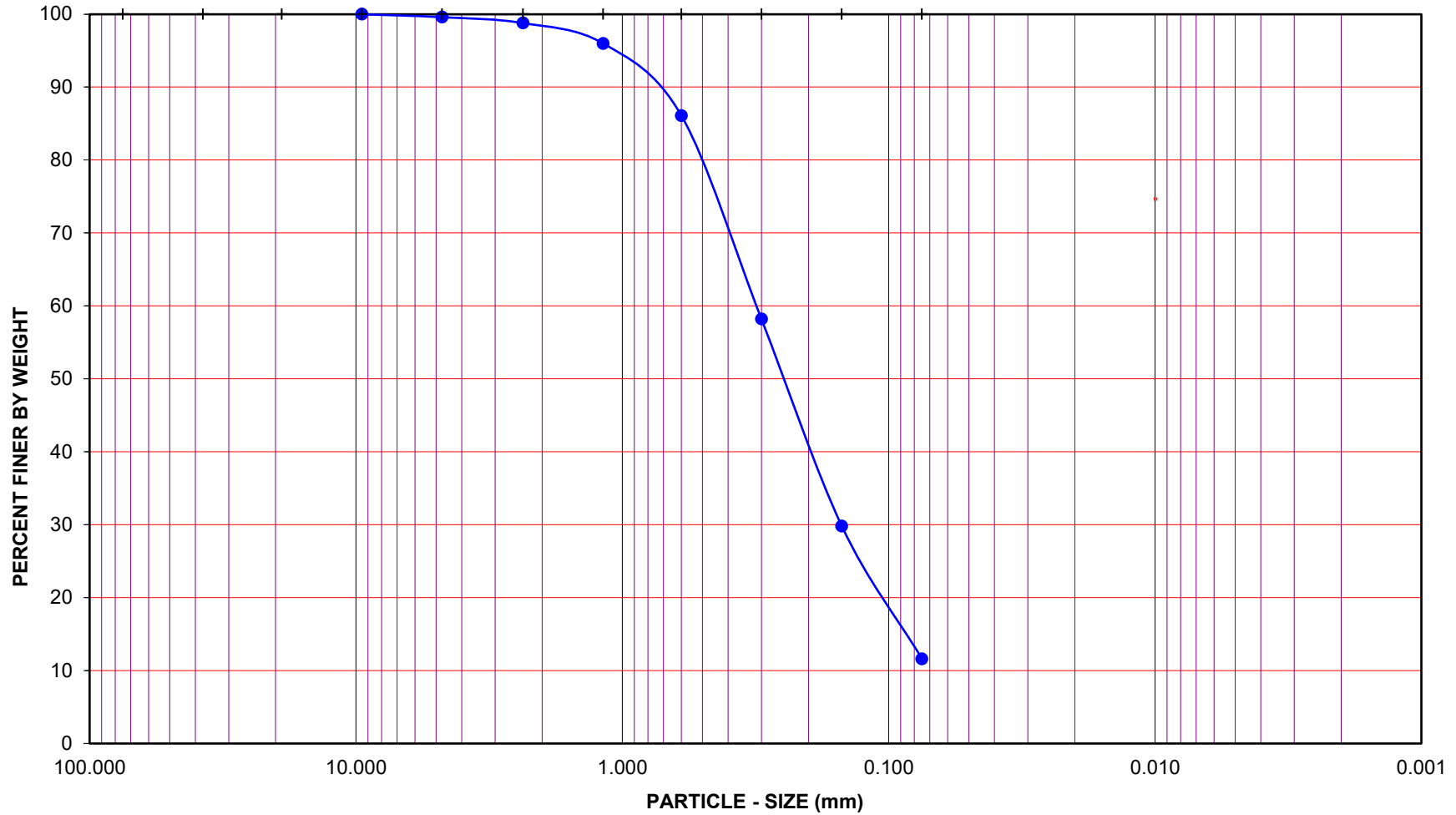
U.S. STANDARD SIEVE OPENING

3.0" 1 1/2" 3/4" 3/8"

U.S. STANDARD SIEVE NUMBER

#4 #8 #16 #30 #50 #100 #200

HYDROMETER



Project Name: N. Bellaire, North Hollywood

Project No.: 23001-01

Boring No.: HS-1

Depth (feet): 5.0

Soil Identification: Light olive gray poorly-graded sand with silt (SP-SM), organic material noted

Sample No.: SPT-1

Soil Type : SP-SM

GR:SA:FI : (%) 0 : 88 : 12



**PARTICLE - SIZE
DISTRIBUTION
ASTM D 6913**

Aug-23

PARTICLE-SIZE ANALYSIS OF SOILS

ASTM D 7928 & D 6913

Project Name: N. Bellaire, North Hollywood

Tested By: GB/GEB/JD

Date: 07/19/23

Project No.: 23001-01

Data Input By: J. Ward

Date: 08/07/23

Boring No.: HS-2

Sample No.: B-1

Depth (feet): 1-5

Soil Identification: Light olive brown silty sand (SM), organic material noted

% Gravel	2	Soil Type SM	Dry Weight of Air-Dry Soil Passing #4	Moisture Content of Oven-Dry Soil Passing #10	After Hydrometer & Wet Sieve ret. in #200 Sieve
% Sand	78				
% Fines	20				

Specific Gravity (Assumed)	2.70	Wt. of Air-Dry Soil + Cont.(g)	0.00	143.66	
Correction for Specific Gravity	0.99	Dry Wt. of Soil + Cont. (g)	614.40	143.23	178.55
Wt. of Air-Dry Soil + Cont. (g)	13681.80	Wt. of Container No. ____ (g)	0.00	59.20	88.87
Wt. of Container	0.00	Moisture Content (%)	N/A	0.51	
Dry Wt. of Soil (g)	13681.80	Wt. of Dry Soil (g)	614.40		89.68

Coarse Sieve		
U.S. Sieve	Cumulative Wt. Of Dry Soil Retained (g)	% Passing
6"	0.00	100.0
3"	0.00	100.0
1½"	0.00	100.0
¾"	0.00	100.0
⅜"	71.26	99.5
No. 4	235.31	98.3
No. 10	19.98	95.1
Pan		

← 1st sample split
← 2nd sample split

Sieve after Hydrometer & Wet Sieve			
U.S. Sieve Size	Cumulative Wt. Of Dry Soil Retained (g)	% Passing	% Total Sample
No. 10	0.00	100.0	95.1
No. 16	3.99	96.3	91.6
No. 30	14.89	86.3	82.0
No. 50	37.67	65.3	62.1
No. 100	67.85	37.5	35.6
No. 200	86.03	20.7	19.7
Pan			

Hydrometer

Wt. of Air-Dry Soil (g)

109.04

Wt. of Dry Soil (g)

108.48

Deflocculant 125 cc of 4% Solution

Date	Time	Elapsed Time (min)	Water Temperature (°C)	Composite Correction 152H	Actual Hydrometer Readings	% Total Sample (%)	Soil Particle Diameter (mm)
20-Jul-23	7:38	0		7.0			
	7:40	2	23.2	7.0	19.0	10.4	0.0333
	7:43	5	23.2	7.0	17.0	8.7	0.0213
	7:53	15	23.1	7.0	14.0	6.1	0.0125
	8:08	30	23.0	7.0	13.0	5.2	0.0089
	8:38	60	22.9	7.0	12.0	4.3	0.0064
	9:38	120	22.6	7.0	11.5	3.9	0.0045
	11:48	250	22.5	7.0	10.0	2.6	0.0032
21-Jul-23	7:38	1440	22.3	7.0	10.0	2.6	0.0013

BOULDERS	COBBLES	GRAVEL		SAND			FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY

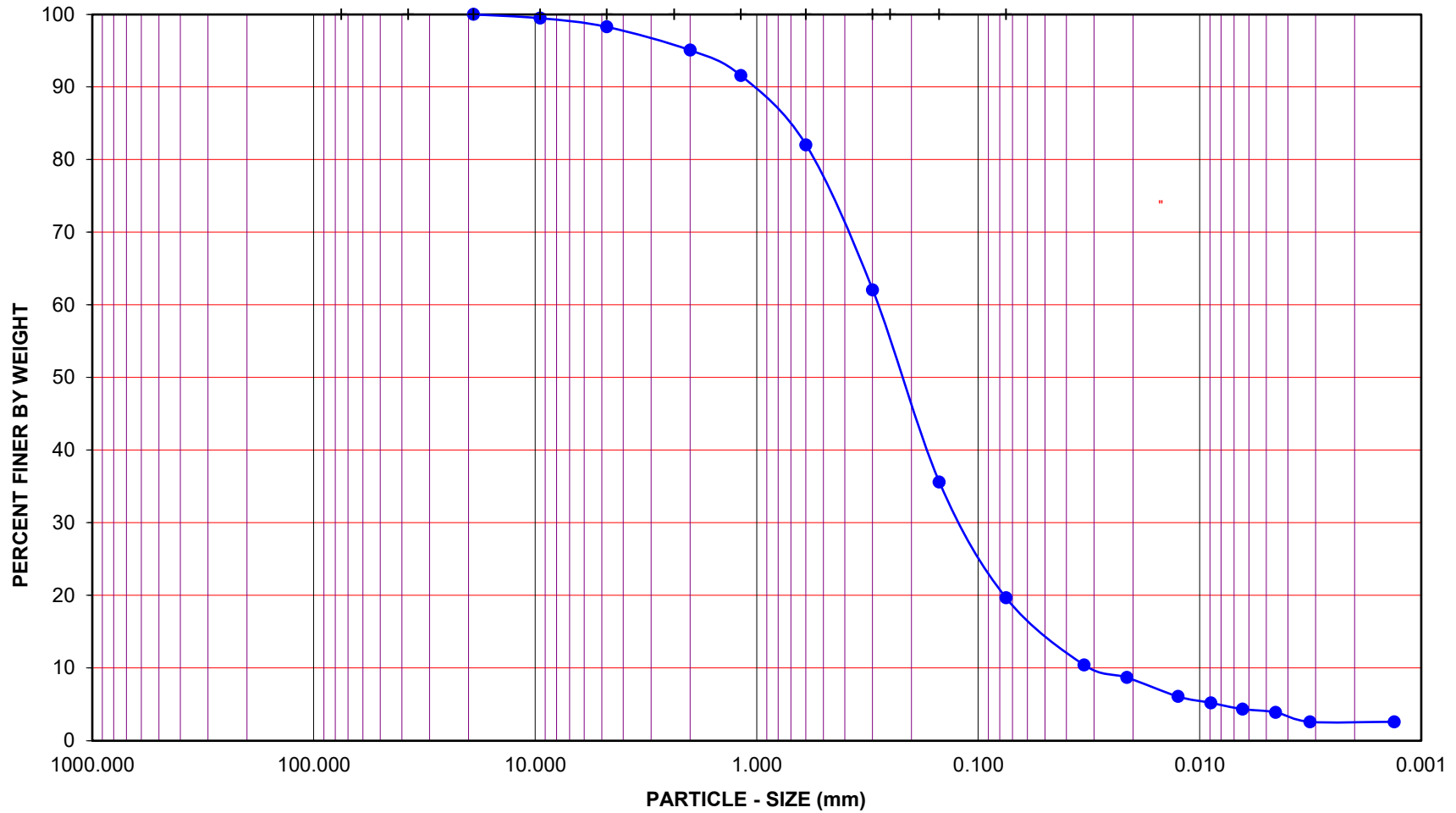
U.S. STANDARD SIEVE OPENING

6" 3" 1 1/2" 3/4" 3/8" #4

U.S. STANDARD SIEVE NUMBER

#8 #16 #30 #50 #100 #200

HYDROMETER



Project Name: N. Bellaire, North Hollywood

Project No.: 23001-01

Boring No.: HS-2

Sample No.: B-1

Depth (feet): 1-5

Soil Type : SM

Soil Identification: Light olive brown silty sand (SM), organic material noted

GR:SA:FI : (%) 2 : 78 : 20



**PARTICLE - SIZE
DISTRIBUTION
ASTM D 7928 & D 6913**

Aug-23



PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS

ASTM D 6913

Project Name: N. Bellaire, North Hollywood

Tested By: G. Bathala Date: 07/21/23

Project No.: 23001-01

Checked By: J. Ward Date: 08/07/23

Boring No.: HS-3

Depth (feet): 2.5

Sample No.: SPT-1

Soil Identification: Light olive brown silty sand (SM)

		Moisture Content of Total Air - Dry Soil	
Container No.:	100	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	676.1	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	110.5	Wt. of Container No. _____ (g)	1.0
Dry Wt. of Soil (g)	565.6	Moisture Content (%)	0.0

After Wet Sieve	Container No.	100
	Wt. of Dry Soil + Container (g)	606.1
	Wt. of Container (g)	110.5
	Dry Wt. of Soil Retained on # 200 Sieve (g)	495.6

U. S. Sieve Size		Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
(in.)	(mm.)		
1 1/2"	37.5		
1"	25.0	0.0	100.0
3/4"	19.0	16.8	97.0
1/2"	12.5	33.4	94.1
3/8"	9.5	45.5	92.0
#4	4.75	71.4	87.4
#8	2.36	103.6	81.7
#16	1.18	158.1	72.0
#30	0.600	243.4	57.0
#50	0.300	353.0	37.6
#100	0.150	436.7	22.8
#200	0.075	487.4	13.8
PAN			

GRAVEL: **13 %**

SAND: **73 %**

FINES: **14 %**

GROUP SYMBOL: **SM**

Cu = D60/D10 = _____

Cc = (D30)²/(D60*D10) = _____

Remarks: _____

GRAVEL				SAND				FINES			
COARSE		FINE		COARSE	MEDIUM	FINE		SILT		CLAY	

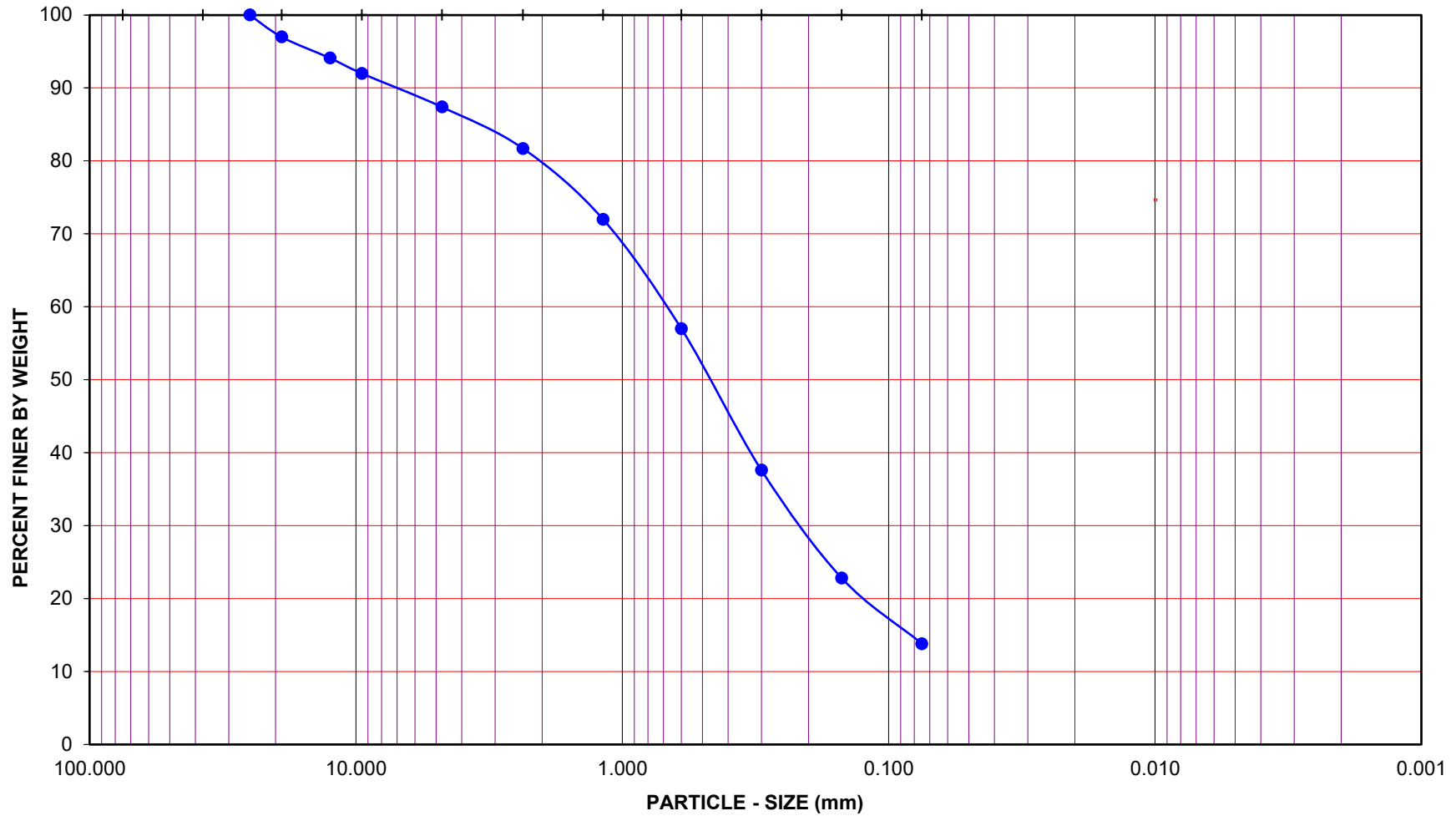
U.S. STANDARD SIEVE OPENING

3.0" 1 1/2" 3/4" 3/8"

U.S. STANDARD SIEVE NUMBER

#4 #8 #16 #30 #50 #100 #200

HYDROMETER



Project Name: N. Bellaire, North Hollywood

Project No.: 23001-01

Boring No.: HS-3

Depth (feet): 2.5

Soil Identification: Light olive brown silty sand (SM)

Sample No.: SPT-1


Soil Type : SM

GR:SA:FI : (%) 13 : 73 : 14



**PARTICLE - SIZE
DISTRIBUTION
ASTM D 6913**

Aug-23

Boring No.	HS-1	HS-1	HS-3	HS-3	HS-4	HS-5		
Sample No.	SPT-2	SPT-3	SPT-2	SPT-3	SPT-1	SPT-1		
Depth (ft.)	10.0	20.0	7.5	15.0	5.0	2.5		
Sample Type	SPT	SPT	SPT	SPT	SPT	SPT		
Soil Identification	Light olive gray poorly-graded sand with silt (SP-SM)	Light olive gray poorly-graded sand with silt and gravel (SP-SM)g	Pale olive poorly-graded sand (SP)	Light olive gray poorly-graded sand with silt and gravel (SP-SM)g	Light brown poorly-graded sand (SP)	Light olive brown poorly-graded sand with silt (SP-SM)		
Moisture Correction								
Wet Weight of Soil + Container (g)	0.00	0.00	0.00	0.00	0.00	0.00		
Dry Weight of Soil + Container (g)	0.00	0.00	0.00	0.00	0.00	0.00		
Weight of Container (g)	1.00	1.00	1.00	1.00	1.00	1.00		
Moisture Content (%)	0.00	0.00	0.00	0.00	0.00	0.00		
Sample Dry Weight Determination								
Weight of Sample + Container (g)	879.65	786.98	811.20	596.09	557.37	984.53		
Weight of Container (g)	159.62	108.05	108.12	105.82	107.52	110.89		
Weight of Dry Sample (g)	720.03	678.93	703.08	490.27	449.85	873.64		
Container No.:								
After Wash								
Method (A or B)	A	A	A	A	A	A		
Dry Weight of Sample + Cont. (g)	841.11	738.48	786.36	562.15	541.58	921.90		
Weight of Container (g)	159.62	108.05	108.12	105.82	107.52	110.89		
Dry Weight of Sample (g)	681.49	630.43	678.24	456.33	434.06	811.01		
% Passing No. 200 Sieve	5.4	7.1	3.5	6.9	3.5	7.2		
% Retained No. 200 Sieve	94.6	92.9	96.5	93.1	96.5	92.8		
	PERCENT PASSING No. 200 SIEVE ASTM D 1140				Project Name: <u>N. Bellaire, North Hollywood</u>			
					Project No.: <u>23001-01</u>			
					Tested By: <u>G. Bathala</u>		Date: <u>07/21/23</u>	



EXPANSION INDEX of SOILS
ASTM D 4829

Project Name: N. Bellaire, North Hollywood Tested By: G. Berdy Date: 07/21/23
 Project No.: 23001-01 Checked By: J. Ward Date: 08/07/23
 Boring No.: HS-1 Depth (ft.): 1-5
 Sample No.: B-1
 Soil Identification: Brown silty, clayey sand (SC-SM), organic material noted

Dry Wt. of Soil + Cont.	(g)	1000.00
Wt. of Container No.	(g)	0.00
Dry Wt. of Soil	(g)	1000.00
Weight Soil Retained on #4 Sieve		0.00
Percent Passing # 4		100.00

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0040
Wt. Comp. Soil + Mold (g)	610.50	450.25
Wt. of Mold (g)	191.40	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	0	0
Wet Wt. of Soil + Cont. (g)	842.70	641.65
Dry Wt. of Soil + Cont. (g)	780.30	579.41
Wt. of Container (g)	0.00	191.40
Moisture Content (%)	8.00	16.04
Wet Density (pcf)	126.4	135.3
Dry Density (pcf)	117.1	116.6
Void Ratio	0.440	0.446
Total Porosity	0.306	0.309
Pore Volume (cc)	63.3	64.1
Degree of Saturation (%) [S _{meas}]	49.1	97.1

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
07/21/23	8:00	1.0	0	0.5665
07/21/23	8:10	1.0	10	0.5655
Add Distilled Water to the Specimen				
07/21/23	8:45	1.0	35	0.5690
07/24/23	6:11	1.0	4201	0.5705

Expansion Index (EI _{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	5
---	----------



EXPANSION INDEX of SOILS
ASTM D 4829

Project Name: N. Bellaire, North Hollywood Tested By: G. Berdy Date: 07/21/23
 Project No.: 23001-01 Checked By: J. Ward Date: 08/07/23
 Boring No.: HS-3 Depth (ft.): 1-5
 Sample No.: B-1
 Soil Identification: Brown silty sand (SM), organic material noted

Dry Wt. of Soil + Cont.	(g)	1000.00
Wt. of Container No.	(g)	0.00
Dry Wt. of Soil	(g)	1000.00
Weight Soil Retained on #4 Sieve		0.00
Percent Passing # 4		100.00

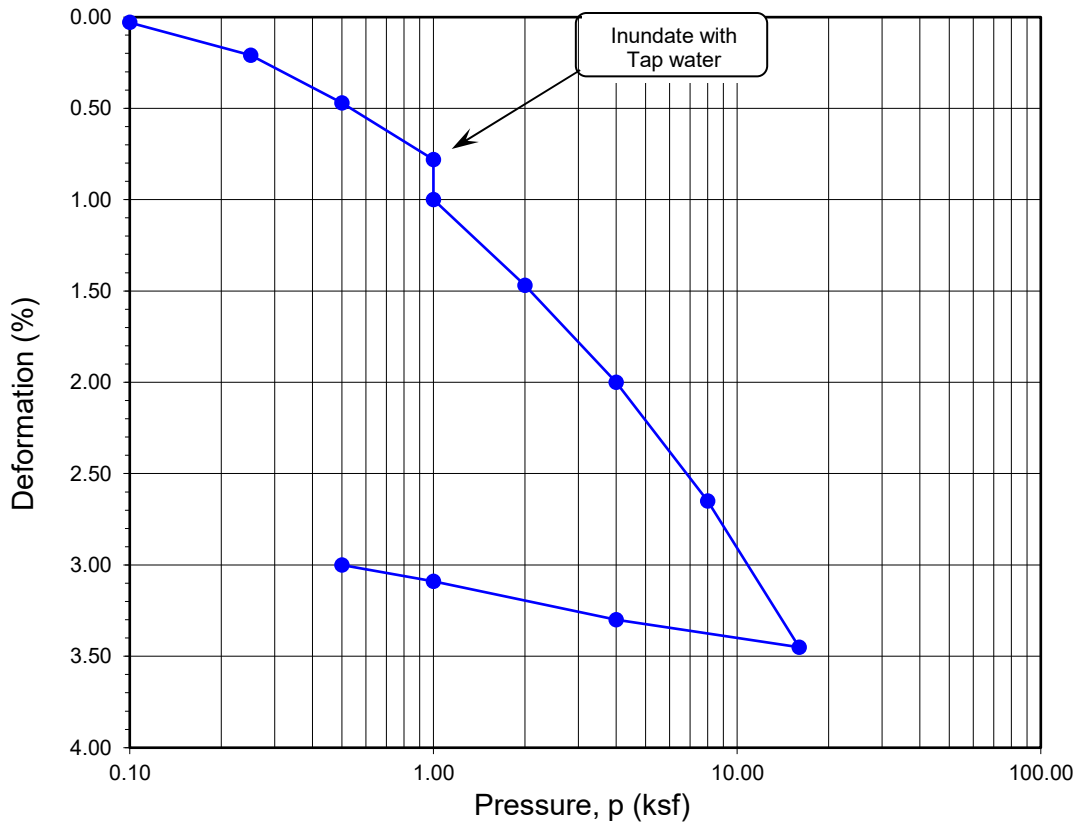
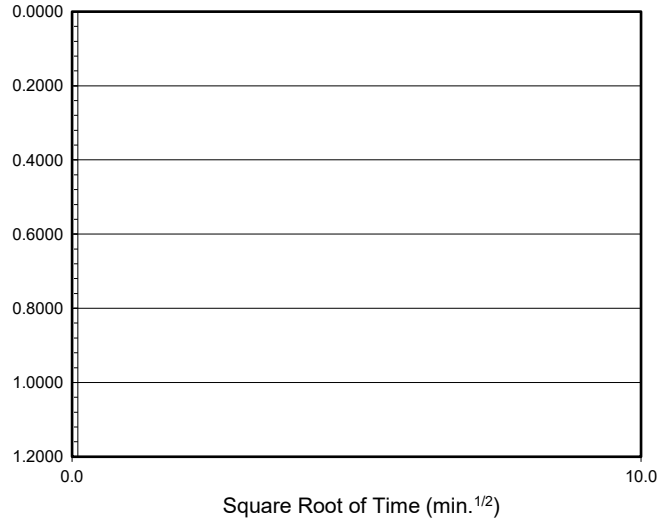
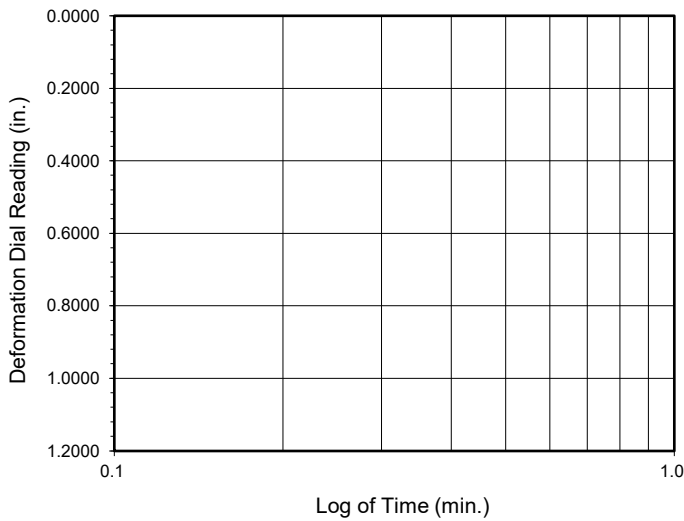
MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	0.9985
Wt. Comp. Soil + Mold (g)	609.60	446.91
Wt. of Mold (g)	180.60	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	0	0
Wet Wt. of Soil + Cont. (g)	854.90	627.51
Dry Wt. of Soil + Cont. (g)	795.20	579.71
Wt. of Container (g)	0.00	180.60
Moisture Content (%)	7.51	11.98
Wet Density (pcf)	129.4	135.0
Dry Density (pcf)	120.4	120.6
Void Ratio	0.401	0.398
Total Porosity	0.286	0.285
Pore Volume (cc)	59.2	58.9
Degree of Saturation (%) [S _{meas}]	50.6	81.2

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
07/21/23	7:24	1.0	0	0.4675
07/21/23	7:34	1.0	10	0.4670
Add Distilled Water to the Specimen				
07/21/23	7:59	1.0	25	0.4670
07/24/23	6:11	1.0	4237	0.4660

Expansion Index (EI _{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	0
---	----------

Time Readings



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-2	R-2	10	2.4	15.6	117.1	117.3	0.440	0.397	14	96

Soil Identification: Light olive brown poorly-graded sand (SP)

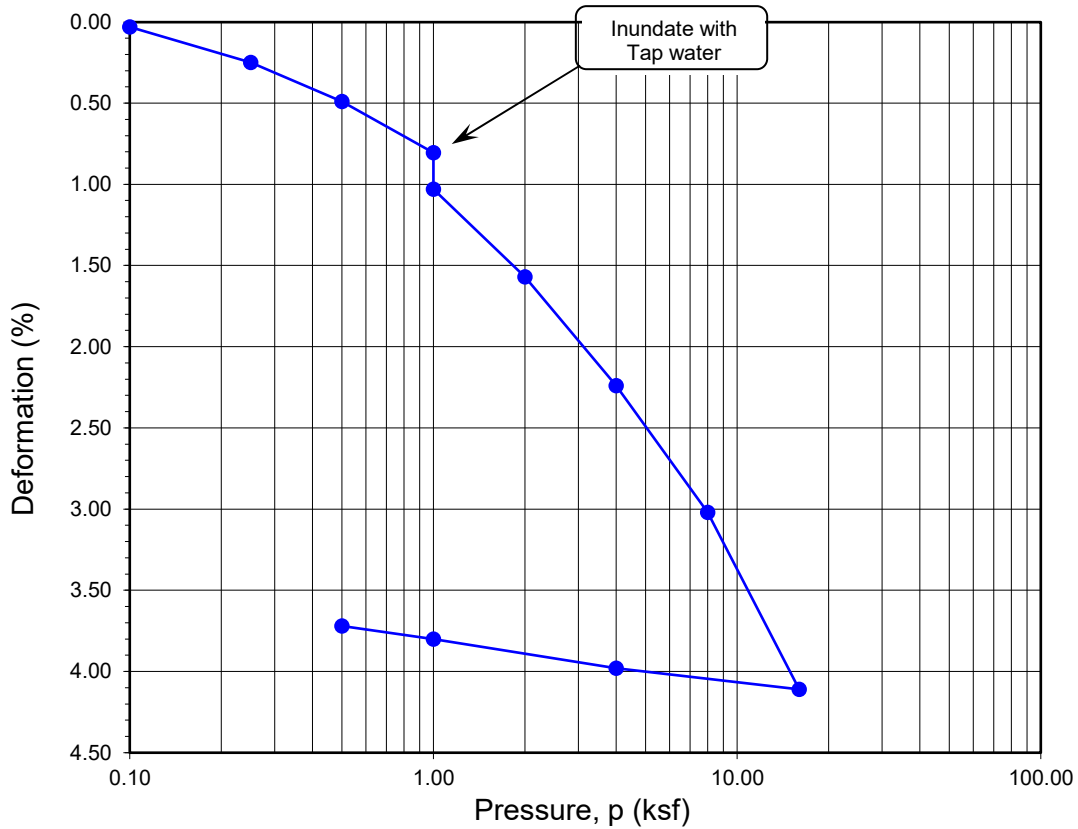
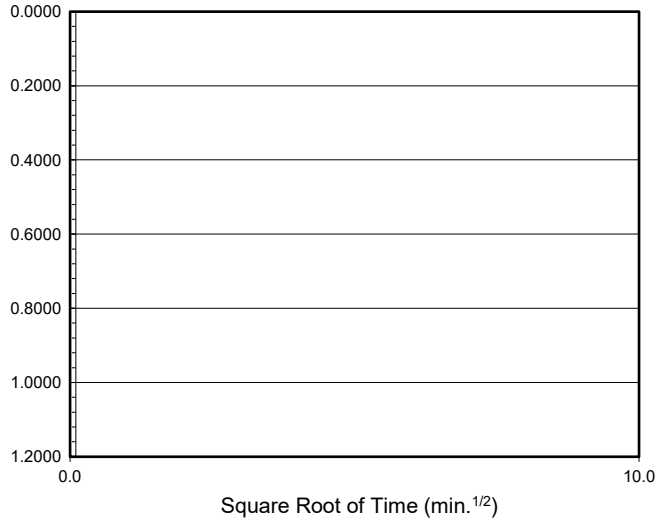
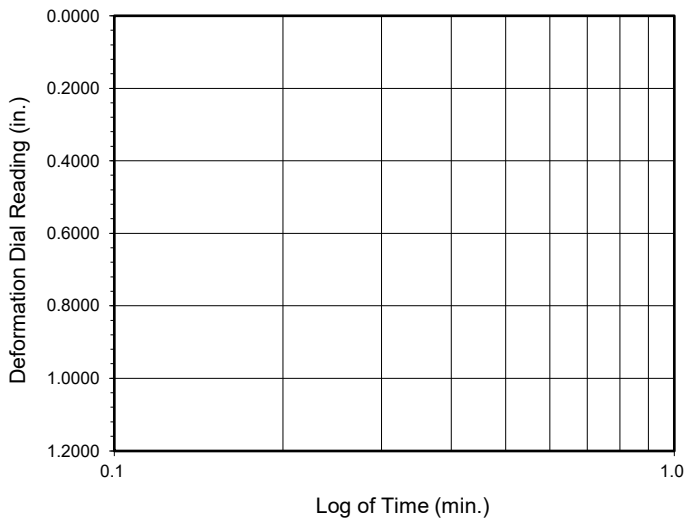


**ONE-DIMENSIONAL CONSOLIDATION
PROPERTIES of SOILS
ASTM D 2435**

Project No.: 23001-01

N. Bellaire, North Hollywood

Time Readings



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-3	R-2	10	4.0	18.1	104.3	106.3	0.616	0.556	18	84

Soil Identification: Light olive brown poorly-graded sand (SP)

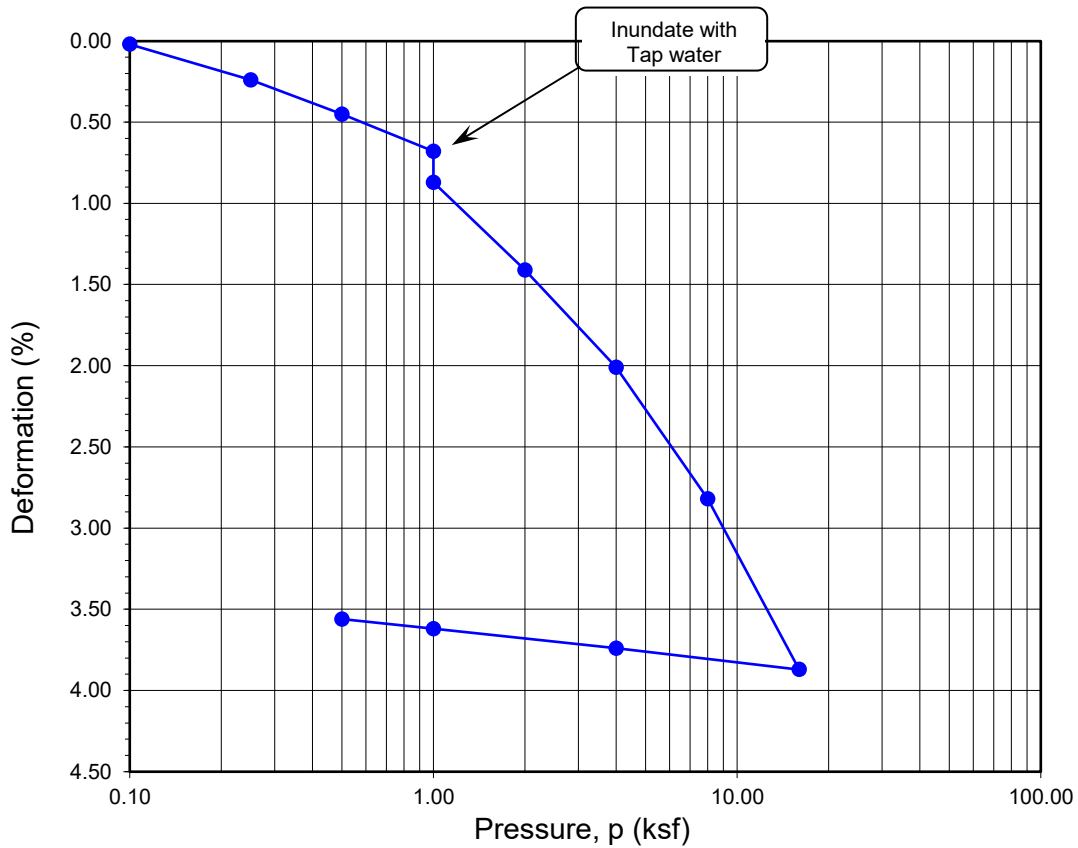
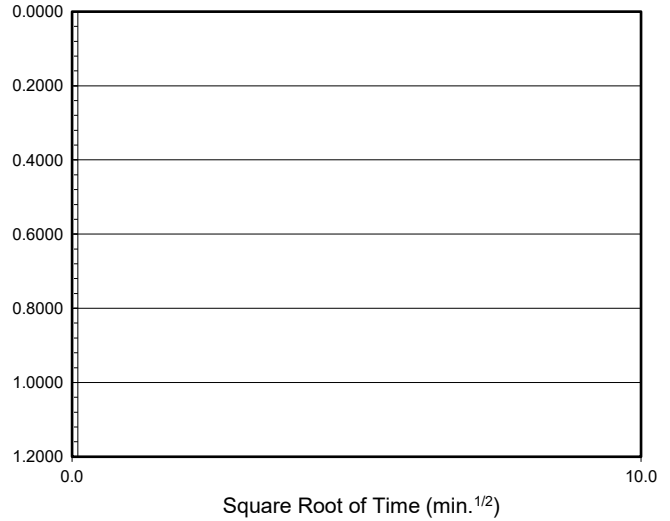
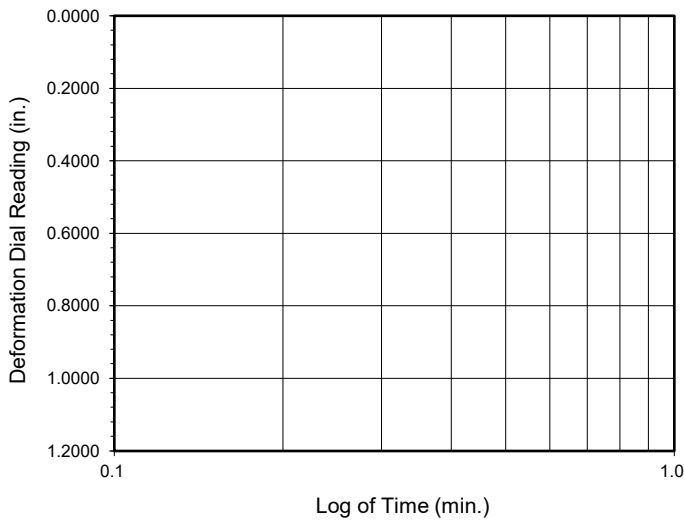


**ONE-DIMENSIONAL CONSOLIDATION
PROPERTIES of SOILS
ASTM D 2435**

Project No.: 23001-01

N. Bellaire, North Hollywood

Time Readings



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-5	R-2	10	3.5	16.8	109.5	111.1	0.540	0.485	18	88

Soil Identification: Light olive gray poorly-graded sand (SP)

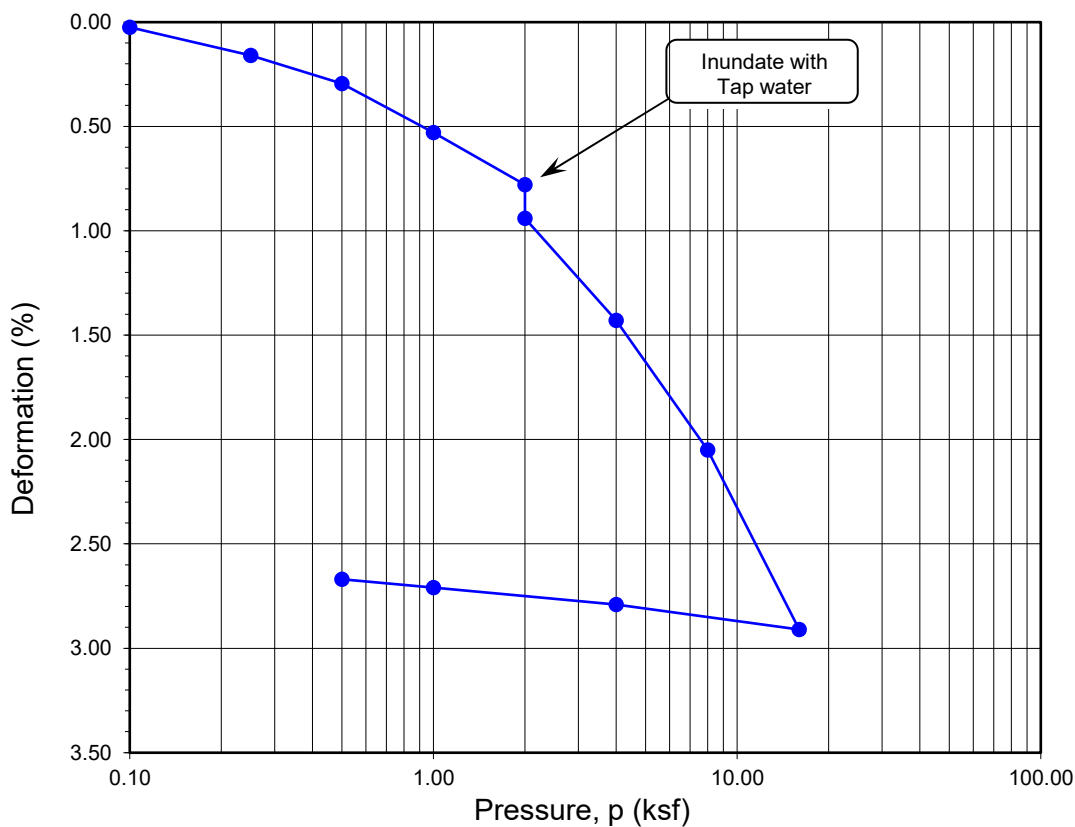
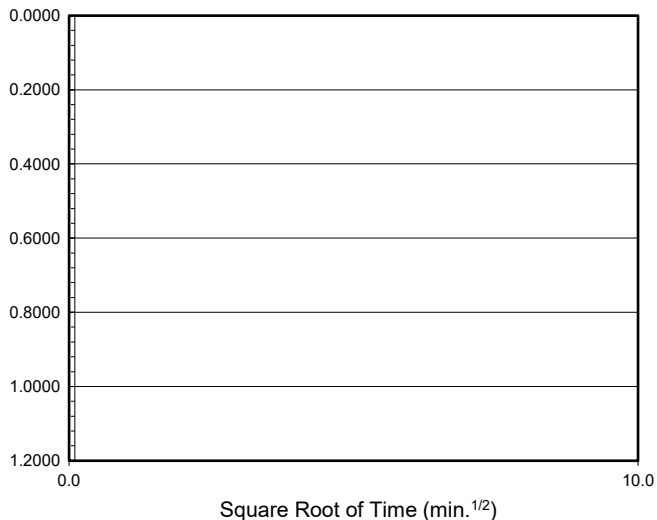
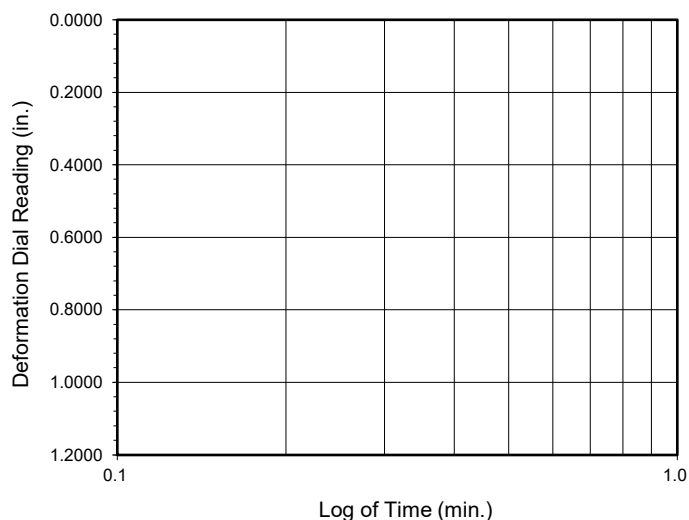


**ONE-DIMENSIONAL CONSOLIDATION
PROPERTIES of SOILS
ASTM D 2435**

Project No.: 23001-01

N. Bellaire, North Hollywood

Time Readings



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
HS-6	R-3	15	3.6	13.6	118.2	120.7	0.426	0.388	23	92

Soil Identification: Light olive brown poorly-graded sand (SP)



**ONE-DIMENSIONAL CONSOLIDATION
PROPERTIES of SOILS
ASTM D 2435**

Project No.: 23001-01

N. Bellaire, North Hollywood

Project Name: N. Bellaire, North Hollywood Tested By: A. Santos Date: 07/18/23
 Project No.: 23001-01 Checked By: J. Ward Date: 08/07/23
 Boring No.: HS-2 Depth (ft.): 1-5
 Sample No.: B-1
 Soil Identification: Light olive brown silty sand (SM), organic material noted

Preparation Method:

Moist
 Dry

Mechanical Ram
 Manual Ram

Mold Volume (ft³)

0.03320

Ram Weight = 10 lb.; Drop = 18 in.

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3774	3837	3796			
Weight of Mold (g)	1808	1808	1808			
Net Weight of Soil (g)	1966	2029	1988			
Wet Weight of Soil + Cont. (g)	733.9	697.7	763.5			
Dry Weight of Soil + Cont. (g)	691.0	644.4	689.8			
Weight of Container (g)	75.3	88.3	76.4			
Moisture Content (%)	6.97	9.58	12.01			
Wet Density (pcf)	130.5	134.7	132.0			
Dry Density (pcf)	122.0	122.9	117.8			

Maximum Dry Density (pcf)

123.5

Optimum Moisture Content (%)

8.5

PROCEDURE USED

Procedure A

Soil Passing No. 4 (4.75 mm) Sieve
 Mold : 4 in. (101.6 mm) diameter
 Layers : 5 (Five)
 Blows per layer : 25 (twenty-five)
 May be used if + #4 is 20% or less

Procedure B

Soil Passing 3/8 in. (9.5 mm) Sieve
 Mold : 4 in. (101.6 mm) diameter
 Layers : 5 (Five)
 Blows per layer : 25 (twenty-five)
 Use if + #4 is >20% and +3/8 in. is 20% or less

Procedure C

Soil Passing 3/4 in. (19.0 mm) Sieve
 Mold : 6 in. (152.4 mm) diameter
 Layers : 5 (Five)
 Blows per layer : 56 (fifty-six)
 Use if +3/8 in. is >20% and +3/4 in. is <30%

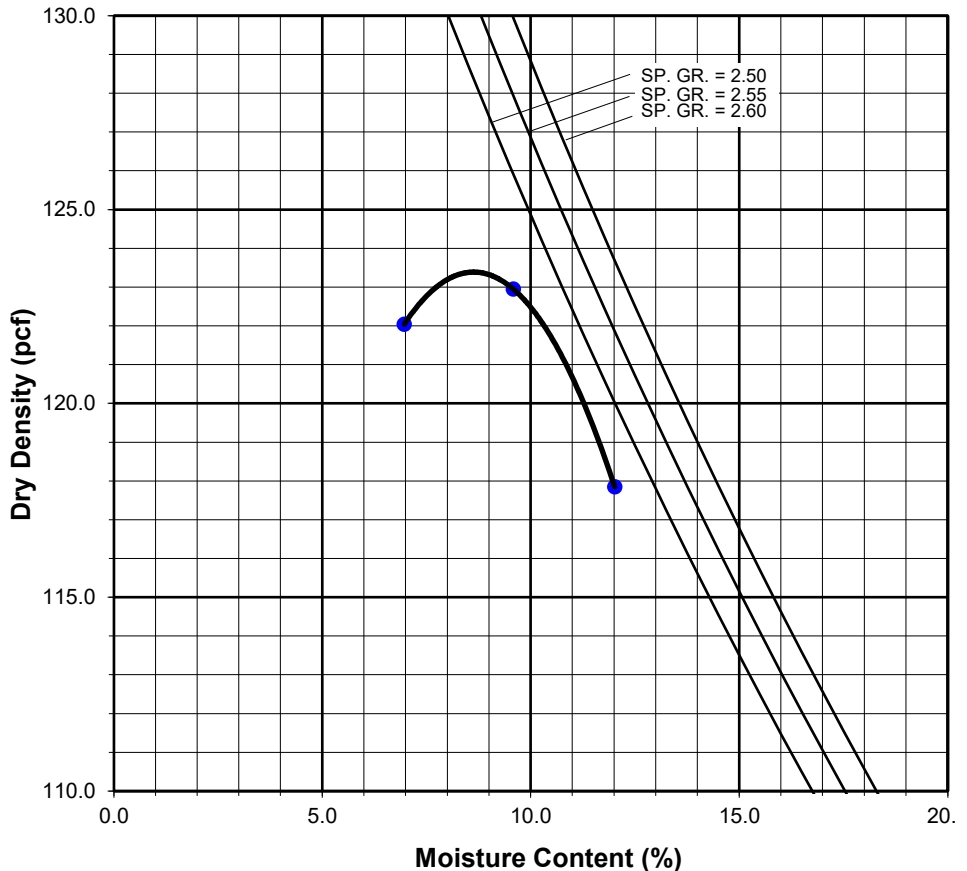
Particle-Size Distribution:

2:78:20

GR:SA:FI

Atterberg Limits:

LL, PL, PI



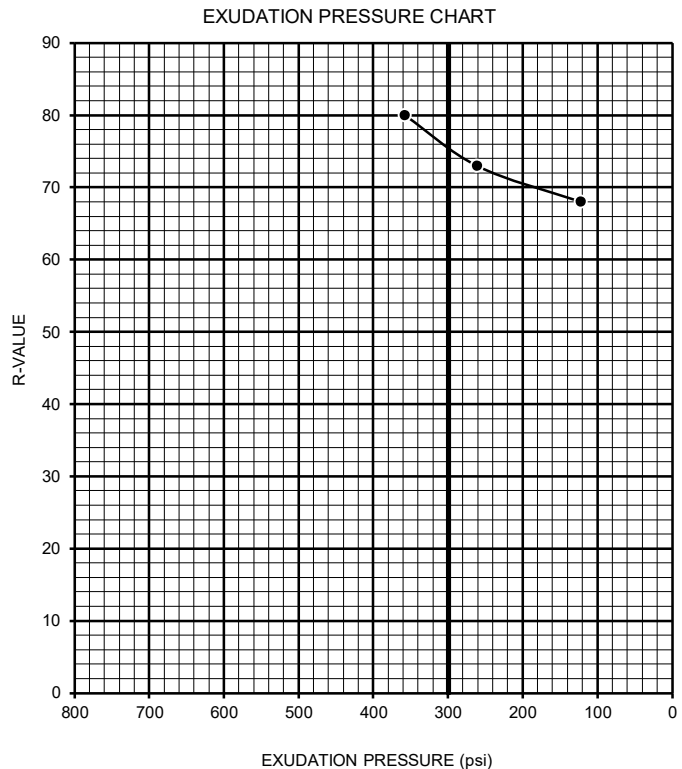
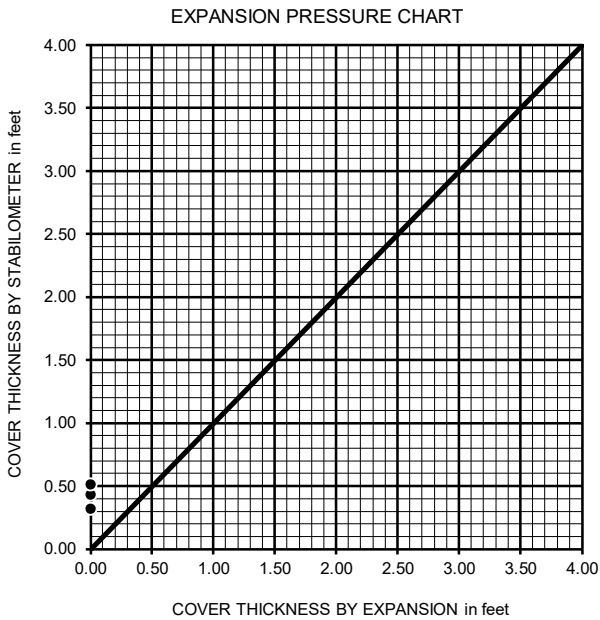


R-VALUE TEST RESULTS DOT CA Test 301

PROJECT NAME:	N. Bellaire, North Hollywood	PROJECT NUMBER:	23001-01
BORING NUMBER:	HS-3	DEPTH (FT.):	1-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Brown silty sand (SM), organic material noted		
		DATE COMPLETED:	7/19/2023

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	8.6	9.3	10.2
HEIGHT OF SAMPLE, Inches	2.49	2.54	2.55
DRY DENSITY, pcf	126.9	126.5	126.0
COMPACTOR PRESSURE, psi	350	350	250
EXUDATION PRESSURE, psi	358	261	123
EXPANSION, Inches x 10exp-4	0	0	0
STABILITY Ph 2,000 lbs (160 psi)	17	23	28
TURNS DISPLACEMENT	5.40	5.50	5.60
R-VALUE UNCORRECTED	80	73	68
R-VALUE CORRECTED	80	73	68

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.32	0.43	0.51
EXPANSION PRESSURE THICKNESS, ft.	0.00	0.00	0.00



R-VALUE BY EXPANSION:	N/A
R-VALUE BY EXUDATION:	75
EQUILIBRIUM R-VALUE:	75



**TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS**

Project Name: N. Bellaire, North Hollywood Tested By : GB/GEB Date: 07/19/23
Project No. : 23001-01 Checked By: J. Ward Date: 08/07/23

Boring No.	HS-1			
Sample No.	B-1			
Sample Depth (ft)	1-5			
Soil Identification:	Brown SC-SM, organic material noted			
Wet Weight of Soil + Container (g)	0.00			
Dry Weight of Soil + Container (g)	0.00			
Weight of Container (g)	1.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.25			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	2			
Crucible No.	300			
Furnace Temperature (°C)	860			
Time In / Time Out	7:30/8:15			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	58.4917			
Wt. of Crucible (g)	58.4893			
Wt. of Residue (g) (A)	0.0024			
PPM of Sulfate (A) x 41150	98.76			
PPM of Sulfate, Dry Weight Basis	99			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)				
ml of AgNO ₃ Soln. Used in Titration (C)				
PPM of Chloride (C -0.2) * 100 * 30 / B				
PPM of Chloride, Dry Wt. Basis	N/A			

pH TEST, DOT California Test 643

pH Value	N/A			
Temperature °C				



**TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS**

Project Name: N. Bellaire, North Hollywood Tested By : GB/GEB Date: 07/19/23
Project No. : 23001-01 Checked By: J. Ward Date: 08/07/23

Boring No.	HS-3			
Sample No.	B-1			
Sample Depth (ft)	1-5			
Soil Identification:	Brown SM, organic material noted			
Wet Weight of Soil + Container (g)	0.00			
Dry Weight of Soil + Container (g)	0.00			
Weight of Container (g)	1.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.04			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	8			
Crucible No.	1			
Furnace Temperature (°C)	860			
Time In / Time Out	7:30/8:15			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	10.6923			
Wt. of Crucible (g)	10.6898			
Wt. of Residue (g) (A)	0.0025			
PPM of Sulfate (A) x 41150	102.87			
PPM of Sulfate, Dry Weight Basis	103			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO ₃ Soln. Used in Titration (C)	0.5			
PPM of Chloride (C -0.2) * 100 * 30 / B	60			
PPM of Chloride, Dry Wt. Basis	60			

pH TEST, DOT California Test 643

pH Value	6.74			
Temperature °C	22.9			



SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name: N. Bellaire, North Hollywood
 Project No. : 23001-01
 Boring No.: HS-3
 Sample No. : B-1

Tested By : G. Berdy Date: 07/24/23
 Checked By: J. Ward Date: 08/07/23
 Depth (ft.) : 1-5

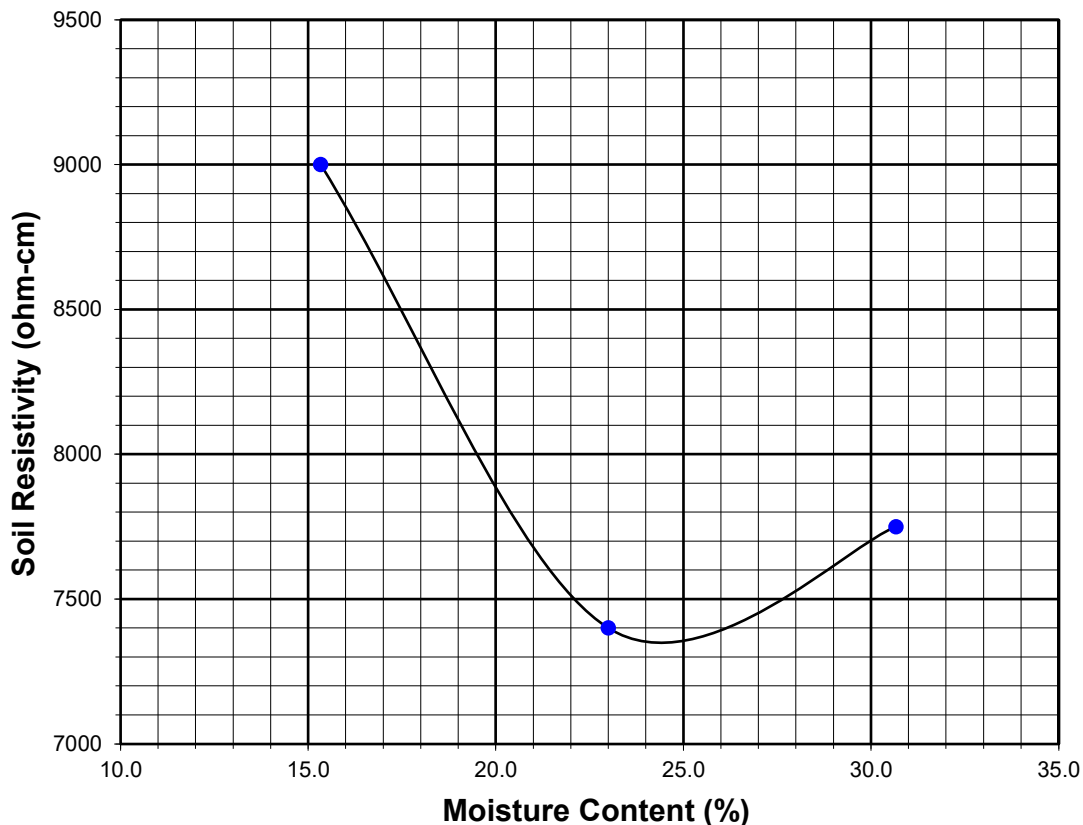
Soil Identification:* Brown SM, organic material noted


*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.


Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	15.34	9000	9000
2	30	23.00	7400	7400
3	40	30.67	7750	7750
4				
5				


Moisture Content (%) (Mci)	0.00
Wet Wt. of Soil + Cont. (g)	0.00
Dry Wt. of Soil + Cont. (g)	0.00
Wt. of Container (g)	1.00
Container No.	
Initial Soil Wt. (g) (Wt)	130.42
Box Constant	1.000
$MC = (((1 + Mci / 100) \times (Wa / Wt + 1)) - 1) \times 100$	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II		DOT CA Test 643	
7350	24.4	103	60	6.74	22.9



Boring No.	HS-1	HS-1	HS-1	HS-1	HS-1	HS-1	HS-2	HS-2
Sample No.	R-1	R-2	R-3	R-4	R-5	R-6	R-1	R-2
Depth (ft.)	2.5	7.5	15.0	25.0	35.0	45.0	5.0	10.0
Sample Type	Ring	Ring	Ring	Ring	Ring	Ring	Ring	Ring
Soil Identification	Light olive gray silt (ML)	Light gray well-graded sand (SW), loose	Light olive gray silty sand (SM), loose	Olive gray silty sand (SM)	Light gray well-graded sand with gravel (SW)g, loose	Olive gray poorly-graded sand (SP)	Light olive brown poorly-graded sand (SP)	Light olive brown poorly-graded sand (SP)
Pocket Penetrometer (tons/ft ²)	0.50	N/A	N/A	0.75	N/A	N/A	N/A	N/A
Weight Soil + Rings / Tube (g)	995.86	884.31	878.95	930.23	1199.40	1088.60	881.05	1137.10
Weight of Rings / Tube (g)	266.40	222.00	222.00	222.00	266.40	266.40	222.00	266.40
Average Length (in.)	6.00	5.00	5.00	5.00	6.00	6.00	5.00	6.00
Average Diameter (in.)	2.415	2.415	2.415	2.415	2.415	2.415	2.415	2.415
Wet. Wt. of Soil + Cont. (g)	194.94	664.76	246.41	444.41	828.64	213.47	193.69	217.56
Dry Wt. of Soil + Cont. (g)	191.80	661.20	244.45	438.19	823.25	211.76	191.02	213.73
Weight of Container (g)	61.75	74.94	64.66	75.43	75.17	72.16	51.18	51.18
Container No.								
Wet Density	101.1	110.2	109.3	117.8	129.3	114.0	109.6	120.7
Moisture Content (%)	2.4	0.6	1.1	1.7	0.7	1.2	1.9	2.4
Dry Density (pcf)	98.7	109.5	108.1	115.8	128.4	112.6	107.6	117.9
Degree of Saturation (%)	9.2	3.0	5.3	10.2	6.2	6.7	9.1	14.8
	MOISTURE & DENSITY of SOILS ASTM D 2216 & ASTM D 2937				Project Name: <u>N. Bellaire, North Hollywood</u>			
					Project No.: <u>23001-01</u>			
					Tested By: <u>G. Bathala</u>		Date: <u>07/19/23</u>	

Boring No.	HS-2	HS-3	HS-3	HS-3	HS-3	HS-3	HS-4	HS-4
Sample No.	R-3	R-1	R-3	R-4	R-5	R-6	R-1	R-2
Depth (ft.)	20.0	5.0	20.0	30.0	40.0	50.0	2.5	7.5
Sample Type	Ring	Ring	Ring	Ring	Ring	Ring	Ring	Ring
Soil Identification	Light olive brown poorly-graded sand (SP)	Olive gray poorly-graded sand with gravel (SP)g	Light olive brown poorly-graded sand (SP)	Olive gray poorly-graded sand (SP)	Light olive gray poorly-graded sand (SP)	Light olive gray poorly-graded sand (SP)	Olive brown poorly-graded sand (SP)	Light olive brown poorly-graded sand (SP)
Pocket Penetrometer (tons/ft ²)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Weight Soil + Rings / Tube (g)	1198.20	749.08	1132.50	1155.90	1165.50	1161.40	1116.40	868.26
Weight of Rings / Tube (g)	266.40	177.60	266.40	266.40	266.40	266.40	266.40	222.00
Average Length (in.)	6.00	4.00	6.00	6.00	6.00	6.00	6.00	5.00
Average Diameter (in.)	2.415	2.415	2.415	2.415	2.415	2.415	2.415	2.415
Wet. Wt. of Soil + Cont. (g)	313.53	294.01	196.09	200.08	230.20	195.16	220.55	246.91
Dry Wt. of Soil + Cont. (g)	306.85	288.90	191.66	195.45	225.44	192.20	214.96	242.21
Weight of Container (g)	66.43	57.48	57.95	56.84	61.84	65.20	63.40	66.33
Container No.								
Wet Density	129.2	118.8	120.1	123.3	124.6	124.1	117.8	107.5
Moisture Content (%)	2.8	2.2	3.3	3.3	2.9	2.3	3.7	2.7
Dry Density (pcf)	125.7	116.3	116.2	119.3	121.1	121.2	113.6	104.7
Degree of Saturation (%)	22.0	13.2	19.9	21.8	20.0	16.1	20.6	11.8
	MOISTURE & DENSITY of SOILS ASTM D 2216 & ASTM D 2937			Project Name: <u>N. Bellaire, North Hollywood</u>				
				Project No.: <u>23001-01</u>				
				Tested By: <u>G. Bathala</u>		Date: <u>07/20/23</u>		

Boring No.	HS-4	HS-4	HS-5	HS-5	HS-6	HS-6	I-2	
Sample No.	R-3	R-4	R-1	R-3	R-1	R-2	R-1	
Depth (ft.)	15.0	25.0	5.0	20.0	2.5	7.5	7.5	
Sample Type	Ring	Ring	Ring	Ring	Ring	Ring	Ring	
Soil Identification	Light olive gray poorly-graded sand with gravel (SP)g	Olive gray poorly-graded sand (SP)	Light olive gray poorly-graded sand (SP)	Light olive brown poorly-graded sand (SP)	Light olive brown poorly-graded sand (SP)	Light gray poorly-graded sand (SP)	Light olive brown poorly-graded sand (SP)	
Pocket Penetrometer (tons/ft ²)	N/A	N/A	N/A	N/A	N/A	N/A	N/A	
Weight Soil + Rings / Tube (g)	1171.20	1100.30	1105.00	1139.00	873.91	904.92	1080.60	
Weight of Rings / Tube (g)	266.40	266.40	266.40	266.40	222.00	222.00	266.40	
Average Length (in.)	6.00	6.00	6.00	6.00	5.00	5.00	6.00	
Average Diameter (in.)	2.415	2.415	2.415	2.415	2.415	2.415	2.415	
Wet. Wt. of Soil + Cont. (g)	210.40	205.95	218.35	228.78	211.06	183.94	178.35	
Dry Wt. of Soil + Cont. (g)	200.36	200.94	213.25	223.36	206.95	180.76	176.07	
Weight of Container (g)	57.18	66.87	57.85	63.19	57.23	56.16	55.14	
Container No.								
Wet Density	125.4	115.6	116.2	121.0	108.4	113.6	112.9	
Moisture Content (%)	7.0	3.7	3.3	3.4	2.7	2.6	1.9	
Dry Density (pcf)	117.2	111.4	112.5	117.0	105.5	110.8	110.8	
Degree of Saturation (%)	43.2	19.7	17.8	20.7	12.4	13.2	9.8	
	MOISTURE & DENSITY of SOILS ASTM D 2216 & ASTM D 2937				Project Name: <u>N. Bellaire, North Hollywood</u>			
					Project No.: <u>23001-01</u>			
					Tested By: <u>G. Bathala</u>		Date: <u>07/20/23</u>	



MOISTURE CONTENT

ASTM D 2216

Project Name: N. Bellaire, North Hollywood

Project No.: 23001-01

Tested By: G. Bathala

Date: 07/17/23

Checked By: J. Ward

Date: 08/08/23

Boring No.	HS-1	HS-1	HS-1	HS-1	HS-1
Sample No.	SPT-1	SPT-2	SPT-3	SPT-4	SPT-5
Depth (ft)	5.0	10.0	20.0	30.0	40.0
Sample Type	SPT	SPT	SPT	SPT	SPT
Sample Description	Light olive gray poorly-graded sand with silt (SP-SM), organics noted	Light olive gray poorly-graded sand with silt (SP-SM)	Light olive gray poorly-graded sand with silt and gravel (SP-SM)g	Light olive gray poorly-graded sand (SP)	Pale olive poorly-graded sand (SP)
Wt. wet soil + container (g)	742.53	886.58	793.63	318.83	266.50
Wt. dry soil + container (g)	735.73	879.65	786.98	316.21	262.49
Weight of container (g)	108.52	159.62	108.05	63.21	63.42
Moisture Content (%)	1.1	1.0	1.0	1.0	2.0

Boring No.	HS-1	HS-2	HS-2	HS-2	HS-2
Sample No.	SPT-6	SPT-1	SPT-2	SPT-3	SPT-4
Depth (ft)	50.0	2.5	7.5	15.0	25.0
Sample Type	SPT	SPT	SPT	SPT	SPT
Sample Description	Olive gray poorly-graded sand with gravel (SP)g	Pale olive poorly-graded sand (SP)	Olive gray poorly-graded sand with gravel (SP)g	Pale olive poorly-graded sand (SP)	Pale olive poorly-graded sand with gravel (SP)g
Wt. wet soil + container (g)	1242.60	200.04	254.61	246.37	263.68
Wt. dry soil + container (g)	1225.40	197.20	248.76	239.27	255.96
Weight of container (g)	107.33	66.87	57.94	56.15	72.16
Moisture Content (%)	1.5	2.2	3.1	3.9	4.2



MOISTURE CONTENT

ASTM D 2216

Project Name: N. Bellaire, North Hollywood
Project No.: 23001-01

Tested By: G. Bathala
Date: 07/18/23
Checked By: J. Ward
Date: 08/08/23

Boring No.	HS-3	HS-3	HS-3	HS-3	HS-3
Sample No.	SPT-1	SPT-2	SPT-3	SPT-4	SPT-5
Depth (ft)	2.5	7.5	15.0	25.0	35.0
Sample Type	SPT	SPT	SPT	SPT	SPT
Sample Description	Light olive brown silty sand (SM)	Pale olive poorly-graded sand (SP)	Light olive gray poorly-graded sand with silt and gravel (SP-SM)g	Olive gray poorly-graded sand (SP)	Pale olive poorly-graded sand with gravel (SP)g
Wt. wet soil + container (g)	694.04	844.15	607.42	192.14	257.65
Wt. dry soil + container (g)	676.06	811.20	596.09	185.38	246.75
Weight of container (g)	110.48	108.12	105.82	56.83	64.65
Moisture Content (%)	3.2	4.7	2.3	5.3	6.0

Boring No.	HS-3	HS-4	HS-4	HS-4	HS-5
Sample No.	SPT-6	SPT-1	SPT-2	SPT-3	SPT-1
Depth (ft)	45.0	5.0	10.0	20.0	2.5
Sample Type	SPT	SPT	SPT	SPT	SPT
Sample Description	Olive gray poorly-graded sand (SP)	Light brown poorly-graded sand (SP)	Light olive brown poorly-graded sand (SP)	Light olive brown poorly-graded sand (SP)	Light olive brown poorly-graded sand with silt (SP-SM)
Wt. wet soil + container (g)	226.74	570.16	202.85	257.15	1007.60
Wt. dry soil + container (g)	221.83	557.37	195.39	246.41	984.53
Weight of container (g)	66.35	107.52	57.18	66.43	110.89
Moisture Content (%)	3.2	2.8	5.4	6.0	2.6



MOISTURE CONTENT

ASTM D 2216

Project Name: N. Bellaire, North Hollywood
Project No.: 23001-01

Tested By: G. Bathala
Date: 07/18/23
Checked By: J. Ward
Date: 08/08/23

Boring No.	HS-5	HS-5	HS-6	HS-6	HS-6
Sample No.	SPT-2	SPT-3	SPT-1	SPT-2	SPT-3
Depth (ft)	7.5	15.0	5.0	10.0	20.0
Sample Type	SPT	SPT	SPT	SPT	SPT
Sample Description	Light olive brown poorly-graded sand (SP)	Light olive brown poorly-graded sand with gravel (SP)g	Light olive brown poorly-graded sand (SP)	Light olive brown poorly-graded sand with gravel (SP)g	Light olive brown poorly-graded sand with gravel (SP)g
Wt. wet soil + container (g)	216.70	281.10	211.79	170.93	237.21
Wt. dry soil + container (g)	210.14	273.72	207.46	167.17	231.06
Weight of container (g)	65.42	55.14	64.01	38.34	72.45
Moisture Content (%)	4.5	3.4	3.0	2.9	3.9

Boring No.	I-1				
Sample No.	SPT-1				
Depth (ft)	7.5				
Sample Type	SPT				
Sample Description	Pale olive poorly-graded sand (SP)				
Wt. wet soil + container (g)	227.33				
Wt. dry soil + container (g)	220.52				
Weight of container (g)	65.20				
Moisture Content (%)	4.4				

Appendix D
General Earthwork and Grading Specifications

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). 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 over-excavated as specified in the following section. Scarification shall continue until soils are broken down and free of oversize material and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Over-excavation

In addition to removals and over-excavations 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 over-excavated 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 over-excavated 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 of 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 the geotechnical consultant. 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).

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). 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 sheepfoot 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.

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 OSHA 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.