February 1, 2024 Project No. 23221-01

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Subject: ***Preliminary Geotechnical Evaluation, Proposed Industrial Development, Site 1, APN 8005-015-051, 12400 Hawkins Street, Santa Fe Springs, California***

In accordance with your request, LGC Geotechnical, Inc. has performed a geotechnical evaluation for the proposed industrial development, Site 1, APN 8005-015-051, located at 12400 Hawkins Street in the City of Santa Fe Springs, California. This report summarizes the results of our background review, subsurface exploration, and geotechnical analyses of the data collected, and presents our findings, conclusions, and preliminary recommendations for the proposed industrial development.

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

Respectfully,

LGC Geotechnical, Inc.

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This report presents the results of our geotechnical evaluation for the proposed industrial development, Site 1, APN 8005-015-051, located at 12400 Hawkins Street in the City of Santa Fe Springs, California (see Site Location Map, Figure 1). The purpose of our work was to collect subsurface data in order to prepare a geotechnical report providing preliminary recommendations for design and construction of the proposed project. Our scope of services included:

- Review of pertinent readily available geotechnical information and geologic maps (Appendix A).
- Subsurface investigation including excavation, sampling, and logging of 11 small-diameter hollow stem borings.
- Performed 3 infiltration tests within the hollow stem borings.
- Laboratory testing of representative samples obtained during our subsurface investigation (Appendix C).
- Geotechnical analysis and evaluation of the data obtained.
- Preparation of this report presenting our preliminary findings, conclusions and recommendations with respect to the proposed site development.

1.2 Background

The subject industrial development is approximately 26.8-acre site is bound to the west and north by existing commercial and industrial developments, to the east by Santa Fe Springs Road and to the south by Telegraph Road. The site is currently occupied by an active oil field and associated equipment. Review of historic aerial photographs suggests the following.

<u>1953 through 1972 Aerial Photos</u>: At this time, the subject site consisted of undeveloped land with a series of oil derricks, manmade (dirt) access roads, above ground storage tanks, and a few miscellaneous small structures.

<u>1988 Aerial Photo</u>: The above ground storage tanks have been removed and some of the oil derricks appeared to have moved to different locations. A structure appeared in the northwestern portion of the site.

<u>1999 Aerial Photo</u>: The manmade roadways throughout the site appear to have been refurbished and appear more defined. The oil derricks remain across the site.

<u>2005 through 2020 Aerial Photos</u>: The site remained relatively unchanged.

1.3 Project Description

Based on the preliminary conceptual site plan (RGA, 2023), two industrial warehouse structure with on-grade parking areas, drive aisles, and a water quality system are proposed. The two proposed industrial warehouse structure designated as "Building North" and "Building South" are approximately 300,800 square feet and 288,400 square feet, respectively. The proposed industrial buildings are anticipated to be at-grade concrete tilt-up structures with estimated maximum column and wall loads of approximately 150 kips and 10 kips per linear foot, respectively. Please note no structural loads or preliminary grading plans were provided to us at the time of this report.

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

1.4 <u>Subsurface Evaluation</u>

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

The eight hollow-stem borings (HS-1 through HS-8) and three hollow-stem borings used for infiltration testing (I-1 through I-3) were drilled to a depths ranging from approximately 10 to 50 feet below existing grade. An LGC Geotechnical representative observed the drilling operations, logged the borings, and collected soil samples for laboratory testing. The borings were excavated using a truck-mounted drill rig equipped with an 8-inch-diameter hollow-stem auger. Driven soil samples were collected by means of the Standard Penetration Test (SPT) and Modified California Drive (MCD) sampler generally obtained at 2.5 to 5-foot vertical increments. The MCD is a split-barrel sampler with a tapered cutting tip and lined with a series of 1-inch-tall brass rings. The SPT sampler and MCD sampler were driven using a 140-pound automatic hammer falling 30 inches to advance the sampler a total depth of 18 inches. The raw blow counts for each 6-inch increment of penetration were recorded on the boring logs. Bulk samples were also collected and logged at select depths for laboratory testing. At the completion of drilling, the borings were backfilled with the native soil cuttings and tamped. Some settlement of the backfill soils may occur over time.

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

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

1.5 Laboratory Testing

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

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

- Dry density of the samples collected ranged from approximately 80 pounds per cubic foot (pcf) to 123 pcf, with an average of 101.5 pcf. Field moisture contents ranged from approximately 2 to 47 percent, with an average of approximately 24.5 percent.
- Four fines content tests were performed and indicated a fines content (passing No. 200 sieve) ranging from approximately 15 to 78 percent. Based on the Unified Soils Classification System (USCS), the tested samples would be classified as "coarse-grained" and "fine-grained."
- Two Atterberg Limit (liquid limit and plastic limit) tests were performed. Results indicate Plasticity Index values of Non-Plastic and 8. The plots are provided in Appendix C
- One laboratory compaction tests of near surface samples indicated a maximum dry density of 123.0 pcf with optimum moisture content of 10.0 percent.
- One direct shear test was performed. The plot is provided in Appendix C.
- Two consolidation tests were performed. The load versus deformation plots are provided in Appendix C.
- Expansion potential testing indicated expansion index values of 12 and 15, corresponding to "Very Low" expansion potential.
- One R-value test was performed. Results indicated an R-value of 43.
- Corrosion testing indicated soluble sulfate contents less than approximately 0.01 percent, a chloride content of 160 parts per million (ppm), pH of 7.82, and a minimum resistivity of 1,048 ohm-centimeters.

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

LGC Geotechnical, Inc.

FIGURE 1
Site Location Map

PROJECT NAME	EPD - 12400 Hawkins Street (Site 1)
PROJECT NO.	23221-01
ENG. / GEOL.	RLD
SCALE	Not to Scale
DATE	February 2024

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

The site is generally located within the Peninsular Ranges Geomorphic Province, specifically within an area known as the Downey Plain, at the eastern margin of the broad Los Angeles Sedimentary Basin. The subject site is bounded approximately 4 miles to the north by the uplifted, northwest-trending Puente Hills. The active, right-lateral strike slip, Whittier Fault Zone is located along the southern front of the Puente Hills. The San Gabriel and Rio Honda Rivers to the west of the site provides major drainage of the areas to the north of the Puente Hills. Existing local drainage pathways to the east of the subject site include the La Canada Verde drainage, respectively. Surface sediments within the area generally consist of older, alluvial fan deposits, except where the local drainages dissect the fans and recent alluvium is deposited.

The Puente Hills located north of the subject site are the nearest bedrock outcrops that were uplifted along the Whittier Hills Fault. Alluvial deposits in this area extend to the ocean going south to the area of Long Beach (Dibblee, 2001).

2.2 Generalized Subsurface Conditions

Based on review of available geologic maps (Dibblee, 2001; Saucedo, 2016), the primary geologic unit underlying the site is Quaternary old alluvial fan deposits, late to middle Pleistocene deposits generally described as moderately to well consolidated sand, clay, and silt. As encountered in our subsurface evaluation, older alluvial deposits consisted of gray to brown, dry to slightly moist, silty sand, sand, and sandy silts, with lesser amounts of clay to the total depth evaluated, approximately 51.5 feet below the surface.

Additionally, undocumented artificial fill consisting generally of silt, sand, and clay was observed at depths of up to approximately 15 feet. The encountered undocumented fill is likely associated with the site history of oil drilling and extraction development within the site.

It should be noted that the 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 can be considered generally massive. No faults have been mapped on or in the vicinity of the site nor were any encountered during our field study.

2.4 Groundwater

Groundwater was not encountered to the maximum explored depth of approximately 51.5 feet below existing grade during this evaluation. Historic high groundwater is mapped at approximately 30 feet below current grade based on the seismic hazard zone report for the Whittier quadrangle (CDMG, 1998).

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

2.5 Field Infiltration Testing

Estimation of infiltration rates was performed in general accordance with guidelines set forth by the County of Los Angeles (2021). In general, a 3-inch diameter perforated PVC pipe was placed in each borehole to be tested and the annulus was backfilled with gravel, including placement of about 2 to 4 inches of gravel at the bottom of the borehole. Infiltration tests were performed using relatively clean water free of particulates, silt, etc. The infiltration wells were pre-soaked during the day of drilling and a 30-minute pre-test was performed during the day of testing. During the pre-test, water was added to the boring and was observed after 10 minutes and 30 minutes to determine test methodology. The measured infiltration rates are considered representative of the site soils in the area of the proposed infiltration system. These measured infiltration rates do not include any factor of safety. Measured infiltration rates have been normalized to correct the 3-Dimensional flow that occurs within the field test to 1-Dimensional flow out of the bottom of the boring. The approximate infiltration test locations are shown on the Boring Location Map (Figure 2) and the infiltration test data is located in Appendix D and is summarized below in Table 1.

TABLE 1

Summary of Infiltration Testing

Infiltration Test Location	Infiltration Test Approx. Depth Below Existing Grade (ft)	Measured Infiltration Rate* (inch/hour)
I-1	10.0	0.0
I-2	15.0	1.2
I-3	15.0	0.5

*Normalized to One-Dimensional Flow, does not include any Reduction Factors.

It should be emphasized that infiltration test results are only representative of the location and depth where they are performed. Varying subsurface conditions may exist outside of the test locations which could alter the calculated infiltration rates indicated above. Infiltration tests are performed using relatively clean water free of particulates, silt, etc. Please refer to Section 4.8 for subsurface water infiltration recommendations including a discussion on Reduction Factors.

2.6 *Faulting and Seismic Hazards*

Prompted by damaging earthquakes in Northern and Southern California, State legislation and policies concerning the classification and land-use criteria associated with faults have been developed. The Alquist-Priolo Earthquake Fault Zoning Act was implemented in 1972 to prevent the construction of urban developments across the trace of active faults. California Geologic Survey Special Publication 42 was created to provide guidance for following and implementing the law requirements. Special Publication 42 was most recently revised in 2018 (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 Earthquake Fault Zone (i.e., Alquist-Priolo Earthquake Fault Act Zone) and no faults were identified on the site during our evaluation. The possibility of damage due to ground rupture is considered low since no active faults are known to cross the site.

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region, which may affect the site, include ground lurching, shallow ground rupture, soil liquefaction and dynamic settlement. These secondary effects of seismic shaking are a possibility throughout the Southern California region and are dependent on the distance between the site and causative fault and the onsite geology. Some of the major active nearby faults that could produce these secondary effects include the Lake Elsinore-Whittier Fault Zone, Newport-Inglewood, and San Andreas Faults, among others (CGS, 2018). A discussion of these secondary effects is provided in the following sections.

2.6.1 *Liquefaction and Dynamic Settlement*

Liquefaction is a seismic phenomenon in which loose, saturated, granular soils behave similarly to a fluid when subject to high-intensity ground shaking. Liquefaction occurs when three general conditions coexist: 1) shallow groundwater; 2) low density non-cohesive (granular) soils; and 3) high-intensity ground motion. Studies indicate that 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 also occur as the sand particles tend to settle and densify as a result of a seismic event.

Based on our review of the State of California Seismic Hazard Zone for liquefaction potential (CDMG, 1999), the site is not located within a liquefaction hazard zone. However, historic high groundwater is mapped at a depth of approximately 30 feet below existing grade (CDMG, 1998); therefore, liquefaction analysis was performed. The alluvial soils encountered below a depth of approximately 30 feet were generally found to be either fine-grained or relatively dense sandy soils and generally not susceptible to liquefaction, except for a few isolated layers. Liquefaction potential was evaluated using the procedures outlined by Special Publication 117A (SCEC, 1999 & CGS, 2008) and the applicable seismic criteria (e.g., 2022 CBC). Liquefaction induced settlement was estimated using the PGA_M per the 2022 CBC and a moment of magnitude of 6.87 (USGS, 2014).

Based on the data obtained from our field evaluation, liquefaction settlement is estimated to be on the order of about 1-inch. Differential seismic settlement may be estimated as one-half of the total seismic settlement over a horizontal span of 40 feet.

2.6.2 Lateral Spreading

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

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

2.7 Seismic Design Criteria

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 33.9441 degrees north and longitude -118.0661 degrees west were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) and adjusted design spectral response acceleration parameters (S_{DS} and S_{D1}) for Site Class D are provided in Table 2 on the following page. Since site soils are Site Class D, additional adjustments are required to code acceleration response spectrums as outlined below and provided in ASCE 7-16. The structural designer should contact the geotechnical consultant if structural conditions (e.g., number of stories, seismically isolated structures, etc.) require site-specific ground motions.

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.87 at a distance of approximately 10.21 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 0.827g (SEAOC, 2023).

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
Ss (Risk-Targeted Spectral Acceleration for Short Periods)	1.743g	From SEAOC, 2023
S_1 (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.623g	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: $S_{MS} = F_aS_S$]	1.743g	-
S_{M1} for Site Class D [Note: $S_{M1} = F_vS_1$]	1.059g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{DS} for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	1.162g	-
S_{D1} for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.706g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
C_{RS} (Mapped Risk Coefficient at 0.2 sec)	0.903	ASCE 7 Chapter 22
C_{R1} (Mapped Risk Coefficient at 1 sec)	0.901	ASCE 7 Chapter 22
*Since site soils are Site Class D and S_1 is greater than or equal to 0.2, the seismic response coefficient Cs is determined by Eq. 12.8-2 for values of $T \leq 1.5T_s$ and taken equal to 1.5 times the value calculated in accordance with either Eq. 12.8-3 for $T_L \geq T > T_s$, or Eq. 12.8-4 for $T > T_L$. Refer to ASCE 7-16.		

2.8 *Oversized Material*

Oversized material (material larger than 8 inches in maximum dimension) may be encountered during site grading. Recommendations are provided for appropriate handling of oversized materials in Appendix E. If feasible, crushing oversized materials onsite or exporting oversized materials may be considered. Incorporating oversized materials into "rock fills" (windrows, rock blankets or individual rock burial) is likely not feasible due to the limited depth of grading. Special handling recommendations should be provided on a case-by-case basis, if necessary.

2.9 *Expansion Potential*

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

3.0 CONCLUSIONS

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

- As encountered at the subject site, soils encountered below the recommended removal and recompaction depth generally consisted of medium dense to very dense sands and silty sands and stiff to very stiff sandy silts and clays to the maximum explored depth of approximately 50 feet below existing grade. The near-surface loose and compressible soils are not suitable for the planned improvements in their present condition (refer to Section 4.1).
- From a geotechnical perspective, onsite soils are anticipated to be suitable for use as general compacted fill, provided they are screened of construction debris and any oversized material (8 inches in greatest dimension).
- Groundwater was not encountered during our subsurface evaluation at a maximum explored depth of approximately 51.5 feet below existing ground surface. Historic high groundwater is estimated to be about 30 feet below existing grade (CDMG, 1998).
- The subject study area is not located within a mapped State of California Earthquake Fault Zone (i.e., Alquist-Priolo Earthquake Fault Act Zone), and based upon our review of published geologic mapping, no known active or potentially active faults are known to exist within or in the immediate vicinity of the site. Therefore, the potential for ground rupture as a result of faulting is considered very low.
- The main seismic hazard that may affect the site is ground shaking from one of the active regional faults. The subject site will likely experience strong seismic ground shaking during its design life.
- The site is not located in a State of California Seismic Hazard Zone for liquefaction. Site soils are considered susceptible to liquefaction. Total dynamic settlement is estimated to be on the order of 1-inch or less. Differential dynamic settlement can be estimated at half of the total settlement over a horizontal span of 40 feet for design of foundations.
- Based on the results of preliminary laboratory testing, site soils are anticipated to have “Very Low” expansion potential. Final design expansion potential must be determined at the completion of grading.
- Oversized material (material larger than 8 inches in maximum dimension) may be encountered during site grading. Recommendations are provided for appropriate handling of oversized materials in Appendix E.
- Excavations into the existing site soils should be feasible with heavy construction equipment in good working order. We anticipate that the sandy and silty earth materials generated from the excavations will be generally suitable for re-use as compacted fill, provided they are relatively free of rocks larger than 8 inches in dimension, construction debris, and significant organic material.
- Some of the on-site soils should be suitable for backfill of site retaining walls; therefore, select grading and stockpiling and/or import of select sandy materials should be anticipated by the contractor.

4.0 RECOMMENDATIONS

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

It should be noted that the following geotechnical recommendations are intended to provide sufficient information to develop the site in general accordance with the 2022 CBC requirements. With regard to the possible occurrence of potentially catastrophic geotechnical hazards such as fault rupture, earthquake-induced landslides, liquefaction, etc. the following geotechnical recommendations should provide adequate protection for the proposed development to the extent required to reduce seismic risk to an "acceptable level." The "acceptable level" of risk is defined by the California Code of Regulations as "that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project" [Section 3721(a)]. Therefore, repair and remedial work of the proposed improvement may be required after a significant seismic event. With regards to the potential for less significant geologic hazards to the proposed development, the recommendations contained herein are intended as a reasonable protection against the potential damaging effects of geotechnical phenomena such as expansive soils, fill settlement, groundwater seepage, etc. It should be understood, however, that our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions but cannot preclude the potential for some cosmetic distress or nuisance issues to develop as a result of the site geotechnical conditions.

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

4.1 Site Earthwork

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

We recommend that earthwork onsite be performed in accordance with the following recommendations, future grading plan review report(s), the 2022 CBC/City of Santa Fe Springs requirements, and the General Earthwork and Grading Specifications for Rough Grading included in Appendix E. In case of conflict, the following recommendations shall supersede those included in Appendix E. The following recommendations may be revised within future grading plan review reports or based on the actual conditions encountered during site grading.

4.1.1 Site Preparation

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

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

4.1.2 Removal and Recompaction Depths and Limits

In order to provide a relatively uniform bearing condition for the planned building structures, upper loose/compressible soils are to be temporarily removed and recompacted as properly compacted fills. Existing undocumented artificial fill was encountered at depths ranging from approximately 5 to 15 feet below existing grades (Appendix B). Within the influence of the proposed structural improvements, existing undocumented artificial fill should be removed to suitable, competent native materials prior to placement of artificial fill to design grades. For preliminary planning purposes, the depth of required removals and recompaction may be estimated as indicated below. Updated recommendations may be required based on additional fieldwork, changes to building layouts, and actual structural loads.

Buildings: Soils shall be temporarily removed and recompacted to a minimum depth of 6 feet below existing grade or 3 feet below the bottom of foundations, whichever is deeper. Additionally, existing undocumented fill (up to approximately 15 feet deep) encountered within the building footprints should be temporarily removed to competent native materials and recompacted as fill. Where space is available, the envelope for removal and recompaction should extend laterally a minimum distance equal to the depth of removal and recompaction below finish grade or 5 feet beyond the edges of the proposed building improvements, whichever is larger.

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

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

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

4.1.3 Temporary Excavations

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

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

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

4.1.4 Subgrade Preparation

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

4.1.5 *Material for Fill*

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

From a geotechnical viewpoint, import soils for general fill (i.e., non-retaining wall backfill) should consist of clean, granular soils of Very Low expansion potential (expansion index of 20 or less based on ASTM D4829). Import for retaining wall backfill should meet the criteria outlined in the paragraph below. Source samples should be provided to the geotechnical consultant for laboratory testing a minimum of three working days prior to any planned importation.

Retaining wall backfill should consist of sandy soils with a maximum of 35 percent fines (passing the No. 200 sieve) per American Society for Testing and Materials (ASTM) Test Method D1140 (or ASTM D6913/D422) and a Very Low expansion potential (EI of 20 or less per ASTM D4829). Soils should also be screened of organic materials, construction debris, and any material greater than 3 inches in maximum dimension. Some of the on-site soils should be suitable for retaining wall backfill due to their low fines content (i.e., silt and clay content) and very low expansion potential; therefore, select grading and stockpiling or import of select sandy materials should be anticipated by the contractor. Samples of retaining wall backfill should be sampled prior to construction to confirm the findings of the investigation.

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

The placement of demolition materials in compacted fill is acceptable from a geotechnical viewpoint provided the demolition material is broken up into pieces not larger than typically used for aggregate base (approximately 1 to 3 inches in maximum dimension) and well blended into fill soils with essentially no resulting voids. Demolition material placed in fills must be free of construction debris (wood, organics, etc.) and reinforcing steel. If asphalt concrete fragments will be incorporated into the demolition materials, approval from an environmental viewpoint may be required and is not the purview of the geotechnical consultant. From our previous experience, we recommend that asphalt concrete fragments be limited to fill areas within planned street areas (i.e., not within building pad areas).

4.1.6 *Placement and Compaction of Fills*

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

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

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

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

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

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 at least 90 percent relative compaction (per ASTM D1557). If gap-graded rock is used for trench backfill, refer to the above Section.

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

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

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

4.1.8 Shrinkage and Subsidence

Allowance in the earthwork volumes budget should be made for an estimated 5 to 20 percent reduction (shrink) in volume of near-surface (upper approximate 5 feet) 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 operations, is expected to be on the order of 0.1 feet. These values are estimates only and exclude losses due to removal of 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 and accuracy of the topographic survey. The above shrinkage estimates are intended as an aid for others in determining preliminary earthwork quantities. However, these estimates should be used with some caution since they are not absolute values.

4.2 Preliminary Foundation Recommendations

The proposed structures may be supported on spread or continuous footings and conventional slabs, provided earthwork is performed in accordance with the recommendations presented in this report. Since the site soils are anticipated to be "Very Low" expansion potential (EI of 20 or less per ASTM D4829), special design considerations from a geotechnical perspective are not anticipated, however, this must be verified based on as-graded conditions. Footings should be supported on properly compacted fill. Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of grading.

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

4.2.1 Slab Design and Construction

From a geotechnical perspective, minimum slab thicknesses of 6 inches and 4 inches are recommended for new slabs in the warehouse areas and office areas, respectively. Slabs are to be supported on compacted fill soils properly prepared in accordance with the recommendations provided in this report. Actual slab reinforcement and thickness should be determined by the structural engineer based on the imposed loading. Additional slab-on-grade recommendations can be provided for alternative building types upon request.

The foundation designer may use a modulus of vertical subgrade reaction (k) of 200 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:

$$k = 200 \times [(B+1)/2B]^2$$

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

4.2.2 ***Foundation Design Parameters***

For the proposed industrial warehouse structures, minimum continuous wall and column footing widths should be 12 inches and 24 inches, respectively, minimum foundation embedment should extend a minimum of 18 inches below the adjacent exterior grade, and interior column footings should be embedded a minimum of 12 inches beneath the adjacent subgrade. The following allowable bearing pressures for both continuous and column spread footings presented in Table 3 on the following page are recommended for corresponding footing widths and embedments.

TABLE 3

Allowable Soil Bearing Pressures

Allowable Static Bearing Pressure (psf)	Minimum Footing Width (feet)	Minimum Footing Embedment* (feet)
4,000	4.0	2.0
3,500	3.0	2.0
3,000	2.0	1.5
2,000	1.0	1.0

* Refers to minimum depth measured below lowest adjacent grade.

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

In utilizing the above-mentioned allowable bearing capacity and provided our earthwork recommendations are implemented, foundation settlement due to structural loads is anticipated to be on the order of 1-inch or less. Differential static settlement may be taken as half of the static settlement (i.e., ½-inch over a horizontal span of 40 feet). Seismic settlement potential is discussed in Section 2.6.1.

4.2.3 Foundation Construction

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

The geotechnical parameters provided herein assume that if the areas adjacent to the foundations 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.

4.2.4 Lateral Load Resistance

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.35 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 250 psf per foot of depth (or pcf) to a maximum of 2,500 psf may be used for the sides of footings poured against properly compacted fill. Allowable passive pressure may be increased to 340 pcf (maximum of 3,400 psf) for short duration seismic loading. This passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions. Frictional resistance and passive pressure may be used in combination without reduction. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt. The provided allowable passive pressures are based on a factor of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively.

4.3 Lateral Earth Pressures for Retaining Walls

The following preliminary lateral earth pressures may be used for site retaining walls. Lateral earth pressures are provided as equivalent fluid unit weights, in pound per square foot (psf) per foot of depth or pcf. These values do not contain an appreciable factor of safety, so the retaining wall designer should apply the applicable factors of safety and/or load factors during design. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of soil over the wall footing.

The following lateral earth pressures are presented on Table 4 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). Retaining wall backfill should also be limited to fill material not exceeding 3 inches in greatest dimension. The wall designer should clearly indicate on the retaining wall plans the required sandy soil backfill criteria. Some of the on-site soils should be suitable for retaining wall backfill due to their low fines content (i.e., silt and clay content) and very low expansion potential; therefore, select grading and stockpiling or import of select sandy materials should be anticipated by the contractor.

TABLE 4

Lateral Earth Pressures – Select Sandy Backfill

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

If the wall can yield enough to mobilize the full shear strength of the soil, it can be designed for “active” pressure. If the wall cannot yield under the applied load, the earth pressure will be higher. The equivalent fluid pressure values assume free-draining conditions. Retaining wall structures should be provided with appropriate drainage and appropriately waterproofed (Refer to Figure 3). Please note that waterproofing and outlet systems are not the purview of the geotechnical consultant. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical consultant.

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

If required, the retaining wall designer may use a seismic lateral earth pressure increment of 10 pcf for level backfill conditions up to a maximum retained height of 10 feet. This increment should be applied in addition to the provided static lateral earth pressure using a "normal" triangular distribution with the resultant acting at H/3 in relation to the base of the retaining structure (where H is the retained height). For the restrained, at-rest condition, the seismic increment may be added to the applicable active lateral earth pressure (in lieu of the at-rest lateral earth pressure) when analyzing short duration seismic loading. Per Section 1803.5.12 of the 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).

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

4.4 <u>Corrosivity to Concrete and Metal</u>

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

Corrosion testing of near-surface bulk samples indicated soluble sulfate contents less than approximately 0.01 percent, chloride content of approximately 160 parts per million (ppm), pH value of approximately 7.82, and minimum resistivity value of 1,048 ohm-cm. Based on Caltrans Corrosion Guidelines (2021), soils are considered corrosive if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 1,500 ppm (0.15 percent) or greater. Based on the test results, soils are not considered corrosive using Caltrans criteria. Note that based on minimum resistivity the soils are considered corrosive to metallic improvements. If improvements that may be susceptible to corrosion are proposed, it is recommended that further evaluation by a corrosion engineer be performed.

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

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

4.5 Preliminary Asphalt Concrete Pavement Sections

Preliminary laboratory test of the onsite soils indicated an R-value of 43. For the purposes of these preliminary recommendations, we used a design R-value of 40 and calculated pavement sections for Traffic Indices of 5.0 (or less), 7.0, and 9.0. R-value testing of the drive aisles and parking lot subgrade will need to be performed to confirm our preliminary testing results/assumptions once the drive aisles and parking areas have been graded to finish subgrade elevations and the final Traffic Index is determined by the Civil Engineer. Determination of the Traffic Index is not the purview of the geotechnical consultant. Final street sections should be confirmed by the project civil engineer based upon the projected design Traffic Index. If requested, LGC Geotechnical will provide sections for alternate TI values.

TABLE 5

Preliminary Asphalt Concrete Pavement Sections

Assumed Traffic Index	5.0 (or less)	7.0	9.0
R -Value Subgrade	40	40	40
AC Thickness	4.0 inches	4.5 inches	6.0 inches
CAB Thickness	4.0 inches	6.0 inches	9.0 inches

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

Earthwork recommendations are provided in Section 4.1 "Site Earthwork" and the related sub-sections of this report.

4.6 Preliminary Portland Cement Concrete Pavement Sections

The provided preliminary Portland Cement concrete (PCC) section options are based on the guidelines of the American Concrete Institute (ACI 330.2R-17). 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. LGC Geotechnical does not perform traffic engineering and determination of traffic loading is not the purview of the geotechnical consultant. 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, 2017). The provided pavement sections assume that edge restraints like a curb and gutter will be provided. To reduce the potential (but not eliminate) for cracking, paving should provide control joints at regular intervals in each direction not exceeding the maximum values provided below. Decreasing the spacing of these joints will further reduce, but not eliminate the potential for unsightly cracking.

The primary input for anticipated loadings over the lifetime of the concrete pavement is based on the Average Daily Truck Traffic (ADTT). Truck loading is defined one 16-kip axle and two 32-kip tandem axles. Other factor to be considered are potentially the use of industrial vehicles (e.g., lift trucks, mobile cranes, gantry cranes, reach stackers, etc.). Static loads from containers and temporary structures stored on the pavement. If semi-trailers are to be disconnected from the tractors from dolly jacks the design should consider concentrated loads imposed on the concrete pavement. These loads typically exceed the axle loads of the semi-trailer combination and are applied to smaller contact areas, especially if applied near joint locations. If these irregular loadings are confined to specific areas of the site the pavement section required thickness can be economized. These and other factors (e.g., traffic patterns, irregular loading, doweled vs un-doweled joints, etc.) outlined in ACI, 2017 should be addressed for the final design.

TABLE 6

Preliminary Portland Cement Concrete Pavement Section Options

No. of Trucks per day design lane	Concrete Thickness* (inch)	Aggregate Base Thickness (inch)	Maximum Joint Spacing Thickness (inch)
10	5.5	4.0	12
100	6.5	6.0	14
300	7.0	6.0	15

*Minimum concrete compressive strength and Modulus of Rupture as indicated above.

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 throughout the design life of the pavement. Failure to maintain a proper maintenance and/or irrigation program may jeopardize the integrity of the pavement.

Additional earthwork recommendations regarding aggregate base are provided in Section 4.1 "Site Earthwork" and the related sub-sections of this report.

4.7 *Nonstructural Concrete Flatwork*

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

Nonstructural and non-vehicular concrete flatwork placed on compacted subgrade may be a minimum 4-inches in thickness with crack control joints spaced 8 feet apart for flatwork slabs and 6 feet apart for flatwork sidewalks. Crack control joints should be sawcut or deep open tool joint to a minimum of 1/3 the concrete thickness. The compacted subgrade below the nonstructural and non-vehicular concrete flatwork should be wet down prior to placing concrete.

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

4.8 *Subsurface Water Infiltration*

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

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

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

The design infiltration rate is determined by dividing the measured infiltration rate by total reduction factor. The total reduction factor is calculated from a series of reduction factors, including; test procedure (RF_t), site variability (RF_v) and long-term siltation plugging and maintenance (RF_s). Based on the Los Angeles County testing guidelines (2021), the reduction factor for long-term siltation plugging and maintenance (RF_s) is the purview of the infiltration system designer (others).

The reduction factor recommendations are provided in Table 7 below. The total reduction factor is calculated as the product of the series of reduction factors listed in Table 7 below ($RF_t + RF_v + RF_s$).

TABLE 7

Shallow Surface Infiltration - Reduction Factors Applied to Measured Infiltration Rate

Consideration	Reduction Factor
Test procedure, boring percolation, RF_t	1.0
Site variability, number of tests, etc., RF_v	1.5
Long-term siltation plugging and maintenance, RF_s	1.0*
Total Reduction Factor, $RF = RF_t + RF_v + RF_s$	**3.5****

*Reduction Factor for long-term siltation plugging and maintenance to be provided by civil engineer
**Total Reduction Factor to be confirmed by civil engineer.

Per the requirements of the Los Angeles County testing guidelines (2021), subsurface materials shall have a design infiltration rate equal to or greater than 0.3 inches per hour. The test procedure, site variability considerations and long-term siltation plugging and maintenance (RF_t, RF_v and RF_s) result in a total reduction factor of 3.5 (to be confirmed by the civil engineer). When total reduction factor presented in Table 7 is applied to the measured infiltration rates presented in Table 1, only one of the three design infiltration rates have a possibility of being greater than the minimum infiltration rate required by the County of Los Angeles for infiltration. Results of infiltration testing are provided in Appendix D.

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

- <u>Due to the fine-grained nature of the soils in the upper 20 to 30 feet below existing grade, we recommend that prior to the installation of any infiltration facilities a series of 12 to 18-inch</u>

diameter borings be drilled to approximately 35 feet below existing grade and backfilled with clean well sand to a minimum of 5 feet above the bottom of the proposed infiltration facility bottom. Above this depth the borings can be backfilled with cuttings as these soils will be removed during excavation of the infiltration facility. The clean well sand should be saturated with water during placement to ensure consolidation.

- We recommend the design of any infiltration system include at least one redundancy or overflow system. It may be prudent to provide an overflow system directly connected to the storm drain system in order to prevent failure of the infiltration system, either as a result of lower than anticipated infiltration and/or very high flow volumes.
- Water discharge from any infiltration systems should not occur within the zone of influence of foundation footings (column and load bearing wall locations). From a geotechnical perspective we recommend a minimum infiltration system setback of 15 feet from the structural improvements.
- An adequate setback distance between any infiltration facility and adjacent property lines should be maintained.
- We recommend the design of any infiltration system include at least one redundancy or overflow system. It may be prudent to provide an overflow system directly connected to the storm drain system in order to prevent failure of the infiltration system, either as a result of lower than anticipated infiltration and/or very high flow volumes.
- The infiltration values provided are based on clean water and this requires the removal of trash, debris, soil particles, etc., and on-going maintenance. Over time, siltation and plugging may reduce the infiltration rate and subsequent effectiveness of the infiltration system. It should be noted that methods to prevent this shall be the responsibility of the infiltration designer and are not the purview of the geotechnical consultant. If adequate measures cannot be incorporated into the design and maintenance of the system, then the infiltration rates may need to be further reduced. These and other factors should be considered in selecting a design infiltration rate.
- Any designed infiltration system will require routine periodic maintenance.
- Contamination and environmental suitability of the site for infiltration was not evaluated by us and should be evaluated by others (environmental consultant). We only addressed the geotechnical issues associated with stormwater infiltration.

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

4.9 Control of Surface Water and Drainage Control

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. If required, 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.

4.10 *Geotechnical Plan Review*

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

4.11 *Geotechnical Observation and Testing*

The recommendations provided in this report are based on limited subsurface observations and geotechnical analysis. The interpolated subsurface conditions should be checked in the field during construction by a representative of LGC Geotechnical. Geotechnical observation and testing is required per Section 1705 of the 2022 California Building Code (CBC).

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

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

5.0 LIMITATIONS

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

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

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the other consultants (at a minimum the civil engineer, structural engineer, landscape architect) and incorporated into their plans. The contractor should properly implement the recommendations during construction and notify the owner if they consider any of the recommendations presented herein to be unsafe, or unsuitable.

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

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

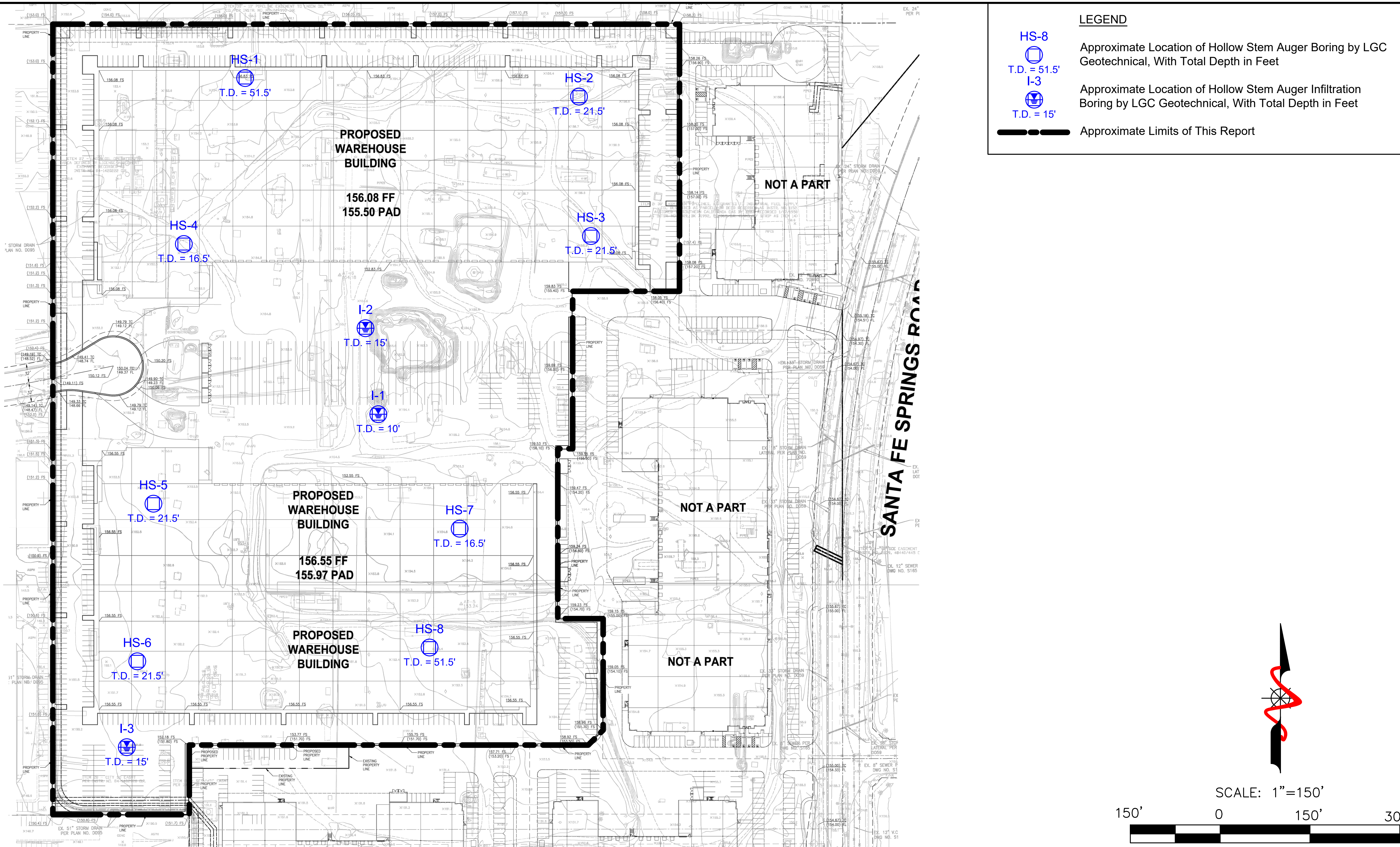

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FIGURE 2
Boring Location Map

PROJECT NAME	EPD - 12400 Hawkins Street (Site 1)
PROJECT NO.	23221-01
ENG. / GEOL.	RLD
SCALE	1" = 150'
DATE	February 2024

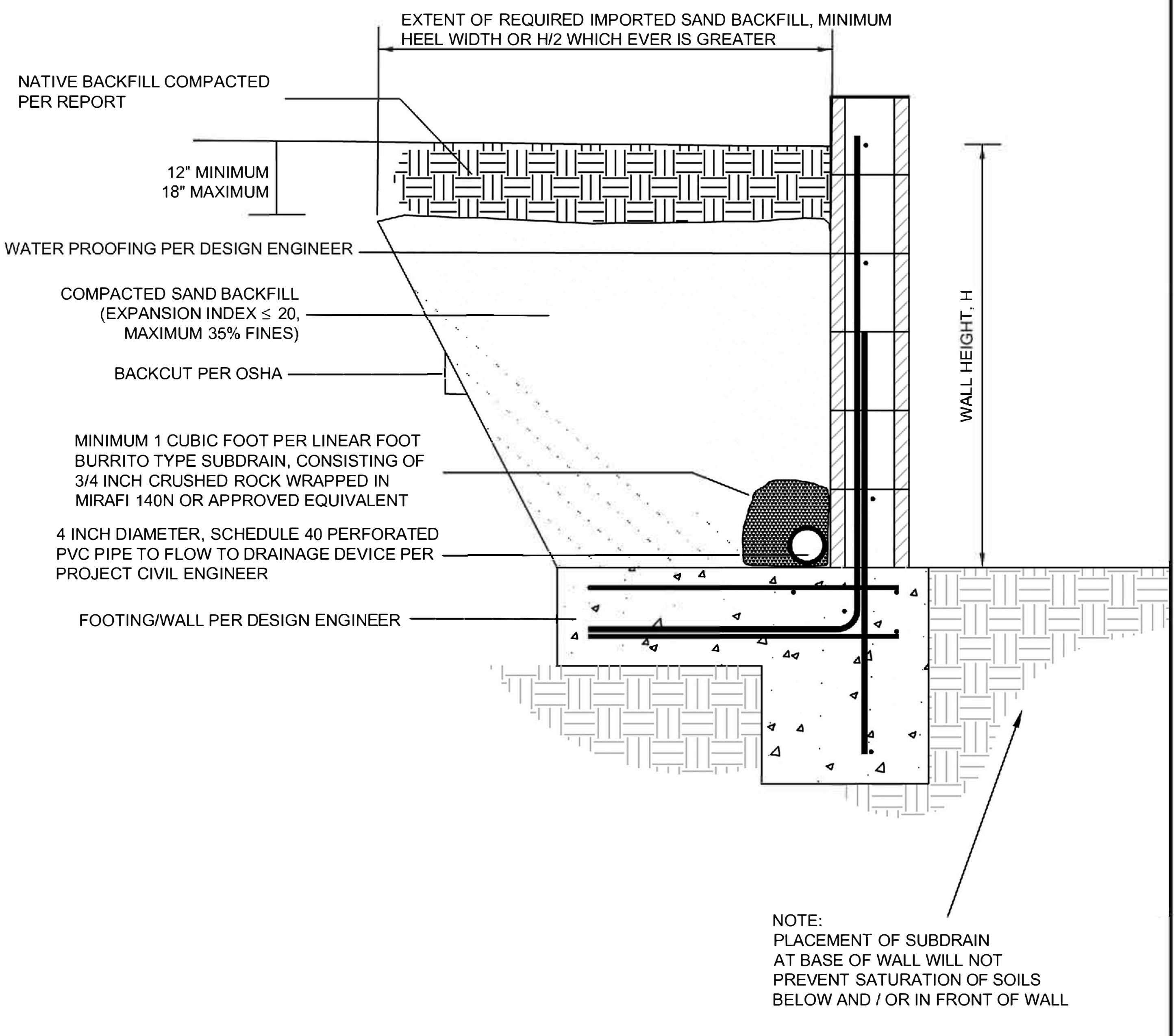

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FIGURE 3

Retaining Wall Backfill Detail

PROJECT NAME	EPD - 12400 Hawkins Street (Site 1)
PROJECT NO.	23221-01
ENG. / GEOL.	RLD
SCALE	Not to Scale
DATE	February 2024

Appendix A
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APPENDIX A

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Appendix B
Boring & Geotechnical Trench Logs

Geotechnical Boring Log Borehole HS-1

Date: 12/18/23	**Drilling Company:** 2R Drilling
Project Name: EPD - Santa Fe Springs (Site 1)	**Type of Rig:** Truck Mounted
Project Number: 23221-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~154' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By RNP
Sampled By RNP
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**Artificial Fill - Undocumented (afu)**	
		B-1				6.5	SM	@ 0' - Silty SAND: brown, slightly moist	
150	5		R-1	17 22 30	119.3	12.4	ML	@ 5' - Sandy SILT: dark brown, moist, hard	
			R-2	7 9 12	108.5	13.7	CL	@ 7.5' - Lean CLAY: yellowish brown, moist, very stiff	CN
145	10		R-3	6 16 25	106.9	11.6	ML	@ 10' - Sandy SILT: dark brown, moist, hard	
140								**Quaternary Older Alluvium (Qoa)**	
	15		SPT-1	7 10 15		15.4		@ 15' - Sandy SILT: olive, very moist, hard	-#200
135	20		R-4	13 23 28	96.0	3.7	SM	@ 20' - Silty SAND: gray, dry, dense	
130	25		SPT-2	5 8 14		27.5	CL	@ 25' - CLAY: blueish gray, very moist, very stiff	
125	30								

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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: 10/20/2022

Geotechnical Boring Log Borehole HS-1

Date: 12/18/23	**Drilling Company:** 2R Drilling
Project Name: EPD - Santa Fe Springs (Site 1)	**Type of Rig:** Truck Mounted
Project Number: 23221-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~154' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By RNP
Sampled By RNP
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		R-5	13 16 20	103.8	10.5	SM	@ 30' - Silty SAND: grayish olive, moist, medium dense	-#200 AL
120									
	35		SPT-3	12 30 31		4.4		@ 35' - Silty SAND: gray, dry, very dense	
115									
	40		R-6	25 40 50/5"	97.7	3.0		@ 40' - Silty SAND: gray, dry, very dense	
110									
	45		SPT-4	9 15 22		26.6	ML	@ 45' - SILT with Sand: dark olive gray, very moist, hard	-#200
105									
	50		R-7	30 50/5"	101.1	2.2	SM	@ 50' - Silty SAND: gray, dry, very dense	
100								Total Depth = 50.9' No Groundwater Encountered Caving after removing augers = 34' (from surface) Backfilled with Cuttings on 12/18/23	
	55								
95									
	60								

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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-2

Date: 12/18/23	**Drilling Company:** 2R Drilling
Project Name: EPD - Santa Fe Springs (Site 1)	**Type of Rig:** Truck Mounted
Project Number: 23221-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~157' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0	B-1				9.5	ML	**Artificial Fill - Undocumented (afu)** @0' - Sandy SILT: dark brown, moist	EI RV
155									
	5		R-1	8 17 21	122.1	10.9		@ 5' - Sandy SILT: very dark brown with brown mottling, moist, hard	
150			R-2	9 12 19	118.0	13.8		@ 7.5' - Sandy SILT: dark gray to brown mottled, moist, very stiff	
	10		R-3	9 16 24	118.3	10.6		**Quaternary Older Alluvium (Qoa)** @ 10' - Sandy SILT: dark gray, moist, hard	
145									
	15		R-4	16 28 34	98.8	22.9		@ 15' - SILT: dark gray, very moist, hard	
140									
	20		SPT-1	5 9 14		24.8	CL	@ 20' - CLAY: brown, very moist, very stiff	
135								Total Depth = 21.5' No Groundwater Encountered Caving after removing augers = 13' (from surface) Backfilled with Cuttings on 12/18/23	
	25								
130									
	30								

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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: 10/20/2022

Geotechnical Boring Log Borehole HS-3

Date: 12/18/23	**Drilling Company:** 2R Drilling
Project Name: EPD - Santa Fe Springs (Site 1)	**Type of Rig:** Truck Mounted
Project Number: 23221-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~157' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**Artificial Fill - Undocumented (afu)**	
		B-1				8.6	SC	@ 0' - Clayey SAND: brown, moist	
155									
	5							**Quaternary Older Alluvium (Qoa)**	
			R-1	4 9 22	115.8	8.9	ML	@ 5' - Sandy SILT: dark brown, slightly moist, very stiff, some pinhole porosity	
150									
			R-2	13 20 22	119.3	11.6		@ 7.5' - Sandy SILT: dark brown, moist, hard, some pinhole porosity	
	10		R-3	12 21 30	119.2	13.7		@ 10' - Sandy SILT: brown, moist, hard	
145									
	15		SPT-1	3 5 8		20.4		@ 15' - SILT: gray, very moist, very stiff	
140									
	20		R-4	13 21 30	105.2	5.4	SM	@ 20' - Sandy SILT: brown, slightly moist, dense	
135								Total Depth = 21.5' No Groundwater Encountered Caving after removing augers = 13' (from surface) Backfilled with Cuttings on 12/18/23	
	25								
130									
	30								

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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: 10/20/2022

Geotechnical Boring Log Borehole HS-4

Date: 12/18/23	**Drilling Company:** 2R Drilling
Project Name: EPD - Santa Fe Springs (Site 1)	**Type of Rig:** Truck Mounted
Project Number: 23221-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~152' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**Artificial Fill - Undocumented (afu)**	
150						12.0	CL	@0' - Sandy CLAY: brown, moist	
	5	B-1	R-1	4 6 14	94.6	25.9	ML	@ 5' - Sandy SILT: dark brown, very moist, very stiff	
145								**Quaternary Older Alluvium (Qoa)**	
			R-2	11 25 37	115.9	15.8		@ 7.5' - Sandy SILT: dark brown, very moist, hard	
	10		R-3	8 8 10	116.3	13.4	SM	@ 10' - Silty SAND: dark gray, very moist, medium dense	
140									
	15		R-4	5 10 17	91.4	30.8	ML	@ 15' - SILT: grayish green, very moist, very stiff	
135								Total Depth = 16.5' No Groundwater Encountered Caving after removing augers = 10.5' (from surface) Backfilled with Cuttings on 12/18/23	
	20								
130									
	25								
125									
	30								

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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: 10/20/2022

Geotechnical Boring Log Borehole HS-5

Date: 12/18/23	**Drilling Company:** 2R Drilling
Project Name: EPD - Santa Fe Springs (Site 1)	**Type of Rig:** Truck Mounted
Project Number: 23221-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~153' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							**Artificial Fill - Undocumented (afu)**	
						6.6	ML	@ 0' - Sandy SILT and Gravel: dark brown, slightly moist	DS EI MD
150									
								Quaternary Older Alluvium (Qoa)	
	5	B-1	R-1	10 14 27	113.0	8.5		@ 5' - Sandy SILT: dark brown, slightly moist, hard, some pinhole porosity	
145			R-2	19 30 43	123.3	11.6		@ 7.5' - Sandy SILT: dark brown, moist, hard	
	10		R-3	14 28 35	88.2	22.7	CL	@ 10' - Sandy CLAY: dark brown, very moist, hard	
140									
	15		SPT-1	3 5 8		24.2		@ 15' - CLAY: gray, very moist, very stiff	
135									
	20		R-4	8 15 23	98.6	22.7	ML	@ 20' - SILT: olive gray, very moist, hard	
130								Total Depth = 21.5' No Groundwater Encountered Caving after removing augers = 14.5' (from surface) Backfilled with Cuttings on 12/18/23	
	25								
125									
	30								

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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: 12/18/23	**Drilling Company:** 2R Drilling
Project Name: EPD - Santa Fe Springs (Site 1)	**Type of Rig:** Truck Mounted
Project Number: 23221-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~152' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0	B-1				6.3	SM	**Artificial Fill - Undocumented (afu)** @ 0' - Silty SAND: brown, slightly moist	
150									
	5		R-1	10 20 37	116.7	11.5	CL/ML	**Quaternary Older Alluvium (Qoa)** @ 5' - CLAY/SILT: brown, slightly moist to moist, hard	
145			R-2	15 20 23	121.8	12.0		@ 7.5' - Sandy CLAY/Sandy SILT: brown, slightly moist to moist, hard	
	10		R-3	12 15 15	109.6	13.5	ML	@ 10' - Sandy SILT: brown, moist, very stiff	
140									
	15		R-4	20 27 45	109.6	2.7	SM	@ 15' - Silty SAND: gray, dry, very dense	
135									
	20		SPT-1	5 19 29		1.9	SM	@ 20' - Silty SAND: gray, dry, very dense	
130								Total Depth = 21.5' No Groundwater Encountered Caving after removing augers = 13' (from surface) Backfilled with Cuttings on 12/18/23	
	25								
125									
	30								

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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: 10/20/2022

Geotechnical Boring Log Borehole HS-7

Date: 12/18/23	**Drilling Company:** 2R Drilling
Project Name: EPD - Santa Fe Springs (Site 1)	**Type of Rig:** Truck Mounted
Project Number: 23221-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~155' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0					7.4	ML	**Artificial Fill - Undocumented (afu)** @ 0' - Sandy SILT: brown, slightly moist	
150	5	B-1	R-1	6 7 15	110.2	7.5	ML	**Quaternary Older Alluvium (Qoa)** @ 5' - Sandy SILT: brown, slightly moist, very stiff, some pinhole porosity	
			R-2	15 18 20	114.4	13.9	CL/ML	@ 7.5' - Sandy CLAY/Sandy SILT: brown, moist, hard	
145	10		R-3	5 6 6	104.2	11.6	ML	@ 10' - Sandy SILT: dark brown, moist, stiff	
140	15		R-4	15 24 34	105.2	9.6	ML	@ 15' - Sandy SILT: bluish gray, slightly moist, hard	
135	20							Total Depth = 16.5' No Groundwater Encountered Caving after removing augers = 9' (from surface) Backfilled with Cuttings on 12/18/23	
130	25								
125	30								

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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-8

Date: 12/19/23	**Drilling Company:** 2R Drilling
Project Name: EPD - Santa Fe Springs (Site 1)	**Type of Rig:** Truck Mounted
Project Number: 23221-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~154' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By RNP
Sampled By RNP
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0	B-1						**Artificial Fill - Undocumented (afu)**	
						8.8	ML	@ 0' - Sandy SILT: brown, slightly moist	
150									
	5							**Quaternary Older Alluvium (Qoa)**	
			R-1	14 20 34	119.2	11.3	CL/ML	@ 5' - CLAY/SILT: brown, slightly moist to moist, hard, some pinhole porosity	
			R-2	11 19 20	116.3	11.3		@ 7.5' - CLAY/SILT: brown, slightly moist to moist, hard	
145									
	10		R-3	5 5 14	110.0	17.6	CL-ML	@ 10' - Silty CLAY: brown, moist to very moist, very stiff	CN
140									
	15		SPT-1	5 10 11		15.6	CL	@ 15' - CLAY: gray, moist, very stiff	
135									
	20		R-4	15 30 50/5"	100.6	3.5	SM	@ 20' - Silty SAND: gray, dry, very dense	
130									
	25		SPT-2	9 20 24		2.5	SM	@ 25' - Silty SAND: gray, dry, very dense	
125									
	30								

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- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole HS-8

Date: 12/19/23	**Drilling Company:** 2R Drilling
Project Name: EPD - Santa Fe Springs (Site 1)	**Type of Rig:** Truck Mounted
Project Number: 23221-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~154' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By RNP
Sampled By RNP
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		R-5	34 50/5"	111.8	1.9	SP-SM	@ 30' - Poorly-Graded SAND with Silt: gray, dry, very dense	
120	35		SPT-3	3 9 17		23.5	CL	@ 35' - Sandy CLAY: grayish olive, very moist, hard	-#200 AL
115	40		R-6	15 35 50/5"	109.5	12.5	ML	@ 40' - Sandy SILT: dark gray, moist, hard	
110	45		SPT-4	12 25 26		2.6	SM	@ 45' - Silty SAND: gray, dry, very dense	
105	50		R-7	30 50/5"	104.3	3.6	SM	@ 50' - Silty SAND: gray, dry, very dense	
100	55							Total Depth = 50.9' No Groundwater Encountered Caving after removing augers = 34' (from surface) Backfilled with Cuttings on 12/19/23	
95	60								

LGC Geotechnical, Inc.

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- EI EXPANSION INDEX
- CN CONSOLIDATION
- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole I-1

Date: 12/18/23
Drilling Company: 2R Drilling
Project Name: EPD - Santa Fe Springs (Site 1)
Type of Rig: Truck Mounted
Project Number: 23221-01
Drop: 30"
Hole Diameter: 8"
Elevation of Top of Hole: ~154' MSL
Drive Weight: 140 pounds
Hole Location: See Geotechnical Map

Logged By RNP
Sampled By RNP
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0		B-1					@0' - Topsoil	
150	5		SPT-1	3 5 7		11.9	CL	@ 5' - CLAY: brown, slightly moist, very stiff	
145			SPT-2	5 10 13		14.8		@ 8' - CLAY: brown, moist, very stiff	
	10							Total Depth = 10' No Groundwater Encountered 3" Perforated Pipe with Filter Sock and Gravel installed Pipe Removed and Backfilled with Cuttings on 12/19/2023	
140	15								
135	20								
130	25								
125	30								

Geotechnical, Inc.

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TEST TYPES:
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- MD MAXIMUM DENSITY
- SA SIEVE ANALYSIS
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- EI EXPANSION INDEX
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- CR CORROSION
- AL ATTERBERG LIMITS
- CO COLLAPSE/SWELL
- RV R-VALUE
- -#200 % PASSING # 200 SIEVE

Geotechnical Boring Log Borehole I-2

Date: 12/18/23	**Drilling Company:** 2R Drilling
Project Name: EPD - Santa Fe Springs (Site 1)	**Type of Rig:** Truck Mounted
Project Number: 23221-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~155' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@0' - Topsoil	
150	5	B-1	SPT-1	5 13 15		10.7	CL	@ 5' - CLAY: brown, slightly moist, hard	
145	10								
			SPT-2	6 15 19		9.8	ML	@ 13' - Sandy SILT: brown, slightly moist, hard	
140	15							Total Depth = 15' No Groundwater Encountered 3" Perforated Pipe with Filter Sock and Gravel installed Pipe Removed and Backfilled with Cuttings on 12/19/2023	
135	20								
130	25								
125	30								

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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: 10/20/2022

Geotechnical Boring Log Borehole I-3

Date: 12/19/23	**Drilling Company:** 2R Drilling
Project Name: EPD - Santa Fe Springs (Site 1)	**Type of Rig:** Truck Mounted
Project Number: 23221-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~151' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By RLD

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
150	0	B-1						@0' - Topsoil	
145	5		SPT-1	8 10 15		10.2	CL	@ 5' - CLAY: brown, slightly moist, hard	
140	10								
			SPT-2	4 7 9		11.6	ML	@ 13' - Sandy SILT: brown, moist, very stiff	
135	15							Total Depth = 15' No Groundwater Encountered 3" Perforated Pipe with Filter Sock and Gravel installed Pipe Removed and Backfilled with Cuttings on 12/20/2023	
130	20								
125	25								
	30								

Geotechnical, Inc.

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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: 10/20/2022

Appendix C
Laboratory Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

The laboratory testing program was formulated towards providing data relating to the relevant engineering properties of the soils with respect to residential construction. 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 relatively undisturbed samples obtained from the test borings and/or trenches. The results of these tests are presented in the boring logs. Where applicable, only moisture content was determined from undisturbed or disturbed samples.

Expansion Index: The expansion potential of selected samples was evaluated by the Expansion Index Test, Standard ASTM D4829. Specimens are molded under a given compactive energy to approximately the optimum moisture content and approximately 50 percent saturation or approximately 90 percent relative compaction. The prepared 1-inch-thick by 4-inch-diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with tap water until volumetric equilibrium is reached. The results of these tests are presented in the table below.

Sample Location	Expansion Index	Expansion Potential*
HS-2 @ 0-5 feet	12	Very Low
HS-5 @ 0-5 feet	15	Very Low

* ASTM D4829

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 and dried and then sieved on a U.S. Standard brass sieve set in accordance with ASTM D6913 (sieve).

Sample Location	Description	% Passing # 200 Sieve
HS-1 @ 15 feet	Sandy Silt	59
HS-1 @ 30 feet	Silty Sand	15
HS-1 @ 45 feet	Silt with Sand	78
HS-8 @ 35 feet	Sandy Clay	63

APPENDIX C (Cont'd)

Laboratory Testing Procedures and Test Results

Atterberg Limits: The liquid and plastic limits ("Atterberg Limits") were determined in accordance with ASTM Test Method D4318 for engineering classification of fine-grained material and presented in the table below. The USCS soil classification indicated in the table below is based on the portion of sample passing the No. 40 sieve and may not necessarily be representative of the entire sample. The plot is provided in this Appendix.

Sample Location	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	USCS Soil Classification
HS-1 @ 30 ft	NP	NP	NP	NP
HS-8 @ 35 ft	31	23	8	CL

Direct Shear: One direct shear test was performed on a remolded sample, which was soaked for a minimum of 24 hours prior to testing. The samples were tested under various normal loads using a motor-driven, strain-controlled, direct-shear testing apparatus (ASTM D3080). The plot is provided in this Appendix.

Consolidation: Two consolidation tests were performed per ASTM D2435. A sample (2.4 inches in diameter and 1 inch in height) was placed in a consolidometer and increasing loads were applied. The sample was 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 ration of the amount of vertical compression to the original sample height. The consolidation pressure curves are provided in this Appendix.

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

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-5 @ 0-5 feet	Dark Brown Sandy Silt with Gravel	123.0	10.0
HS-5 @ 0-5 feet	Dark Brown Sandy Silt with Gravel Correction (20% Gravel)	130.0	8.0

Laboratory Testing Procedures and Test Results

R-Value: The resistance R-value was determined by the ASTM D2844 for base, subbase, and basement soils. The samples were prepared and exudation pressure and R-value were determined. The graphically determined R-values at exudation pressure of 300 psi are reported in this appendix. These results were used for pavement design purposes.

Chloride Content: Chloride content was tested in accordance with Caltrans Test Method (CTM) 422. The results are presented below.

Sample Location	Chloride Content, ppm
HS-2 @ 0-5 feet	160

Soluble Sulfates: The soluble sulfate contents of selected samples were determined by standard geochemical methods (CTM 417). The soluble sulfate content is used to determine the appropriate cement type and maximum water-cement ratios. The test results are presented in the table below.

Sample Location	Sulfate Content (ppm)	Sulfate Exposure Class *
HS-2 @ 0-5 feet	74	S0

*Based on ACI 318R-14, Table 19.3.1.1

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-2 @ 0-5 feet	7.82	1048

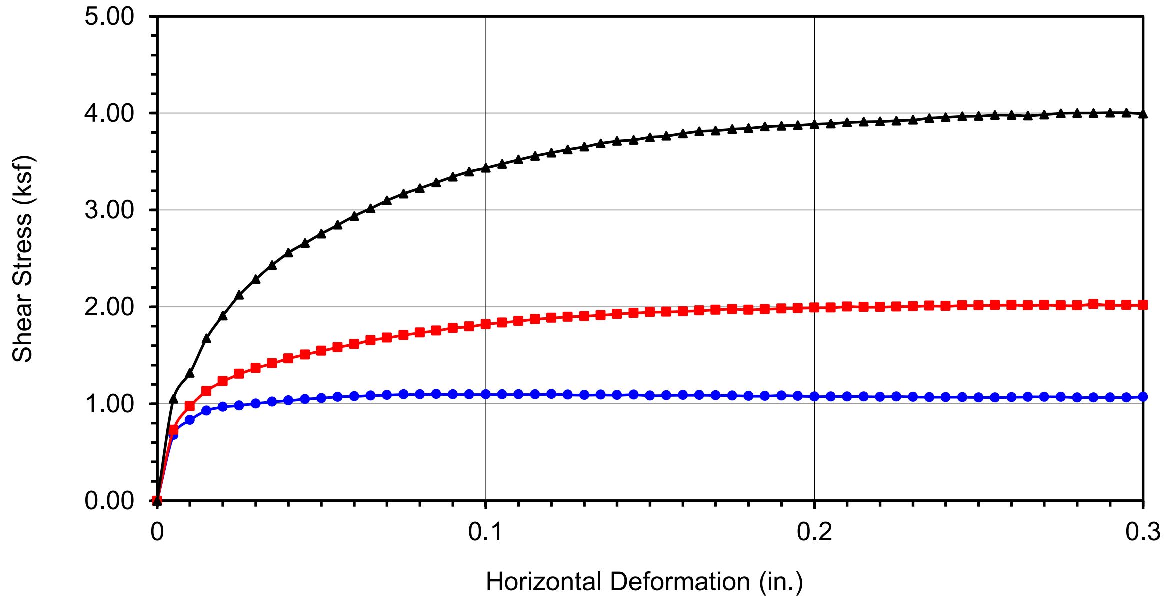
DIRECT SHEAR TEST

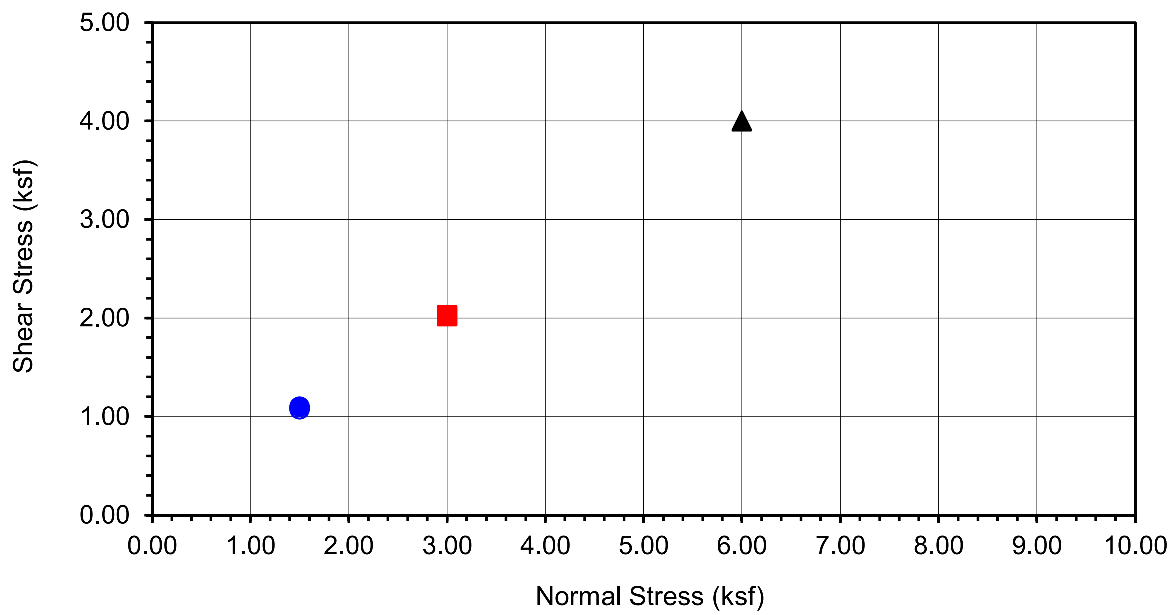
Consolidated Drained - ASTM D 3080

Project Name:	Telegraph Rd Santa Fe Springs Site 1	Tested By:	G. Bathala	Date:	01/09/24
Project No.:	23221-01	Checked By:	J. Ward	Date:	01/15/24
Boring No.:	HS-5	Sample Type:	90% Remold		
Sample No.:	B-1	Depth (ft.):	0-5		

Soil Identification: Dark brown sandy silt with gravel s(ML)g

Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	192.27	191.52	189.83
Weight of Ring(gm):	45.27	44.47	42.54
Before Shearing			
Weight of Wet Sample+Cont.(gm):	159.16	159.16	159.16
Weight of Dry Sample+Cont.(gm):	151.04	151.04	151.04
Weight of Container(gm):	68.52	68.52	68.52
Vertical Rdg.(in): Initial	0.2315	0.2560	0.0000
Vertical Rdg.(in): Final	0.2470	0.2777	-0.0321
After Shearing			
Weight of Wet Sample+Cont.(gm):	215.84	216.74	191.05
Weight of Dry Sample+Cont.(gm):	194.45	195.60	171.85
Weight of Container(gm):	61.83	63.20	39.45
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43





Boring No.	**HS-5**
Sample No.	**B-1**
Depth (ft)	**0-5**

Sample Type:

90% Remold

Soil Identification:
Dark brown sandy silt with gravel s(ML)g

Normal Stress (kip/ft²)	1.500	3.000	6.000
Peak Shear Stress (kip/ft²)	● 1.100	■ 2.028	▲ 4.005
Shear Stress @ End of Test (ksf)	○ 1.072	□ 2.018	△ 3.996
Deformation Rate (in./min.)	0.0017	0.0017	0.0017
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	9.84	9.84	9.84
Dry Density (pcf)	111.3	111.3	111.5
Saturation (%)	51.6	51.7	51.9
Soil Height Before Shearing (in.)	0.9845	0.9783	0.9679
Final Moisture Content (%)	16.1	16.0	14.5

DIRECT SHEAR TEST RESULTS

Consolidated Drained - ASTM D 3080

Project No.: 23221-01

Telegraph Rd Santa Fe Springs_Site 1

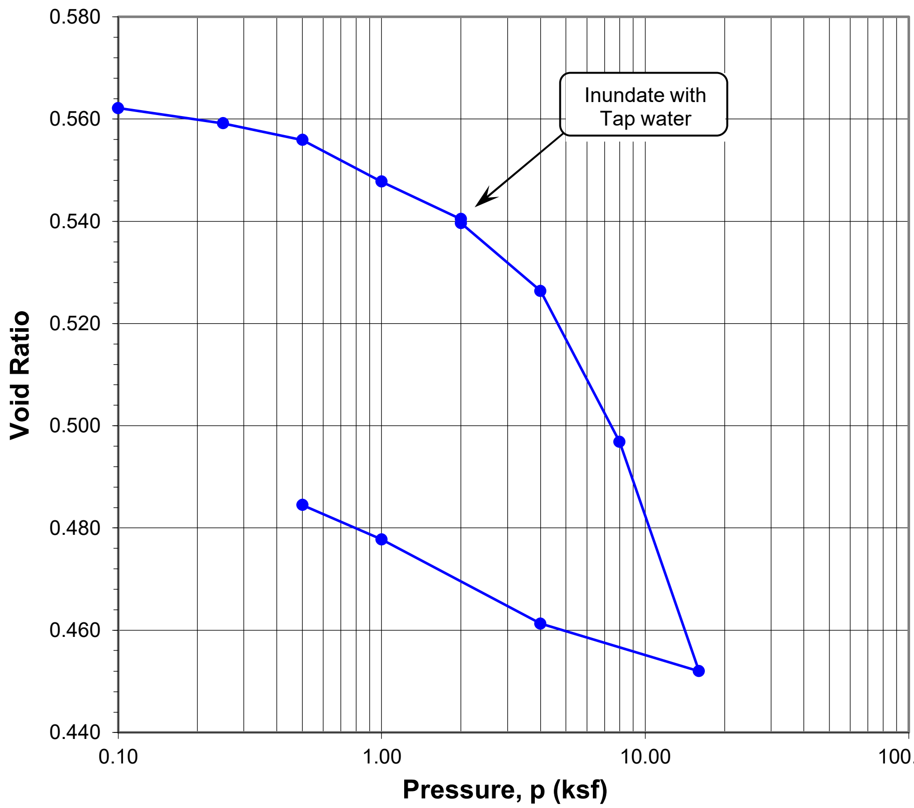
01-24

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Telegraph Rd Santa Fe Springs_Site 1	Tested By:	GB/JD	Date:	01/04/24
Project No.:	23221-01	Checked By:	J. Ward	Date:	01/15/24
Boring No.:	HS-1	Depth (ft.):	7.5		
Sample No.:	R-2	Sample Type:	Ring		
Soil Identification:	Yellowish brown lean clay (CL)				

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	189.96
Weight of Ring (g):	42.50
Height after consol. (in.):	0.9502
Before Test	
Wt. of Wet Sample+Cont. (g):	190.22
Wt. of Dry Sample+Cont. (g):	173.98
Weight of Container (g):	55.14
Initial Moisture Content (%)	13.7
Initial Dry Density (pcf)	107.9
Initial Saturation (%):	66
Initial Vertical Reading (in.)	0.1418
After Test	
Wt. of Wet Sample+Cont. (g):	248.84
Wt. of Dry Sample+Cont. (g):	225.72
Weight of Container (g):	55.52
Final Moisture Content (%)	18.10
Final Dry Density (pcf):	111.8
Final Saturation (%):	96
Final Vertical Reading (in.)	0.1960
Specific Gravity (assumed):	2.70
Water Density (pcf):	62.43

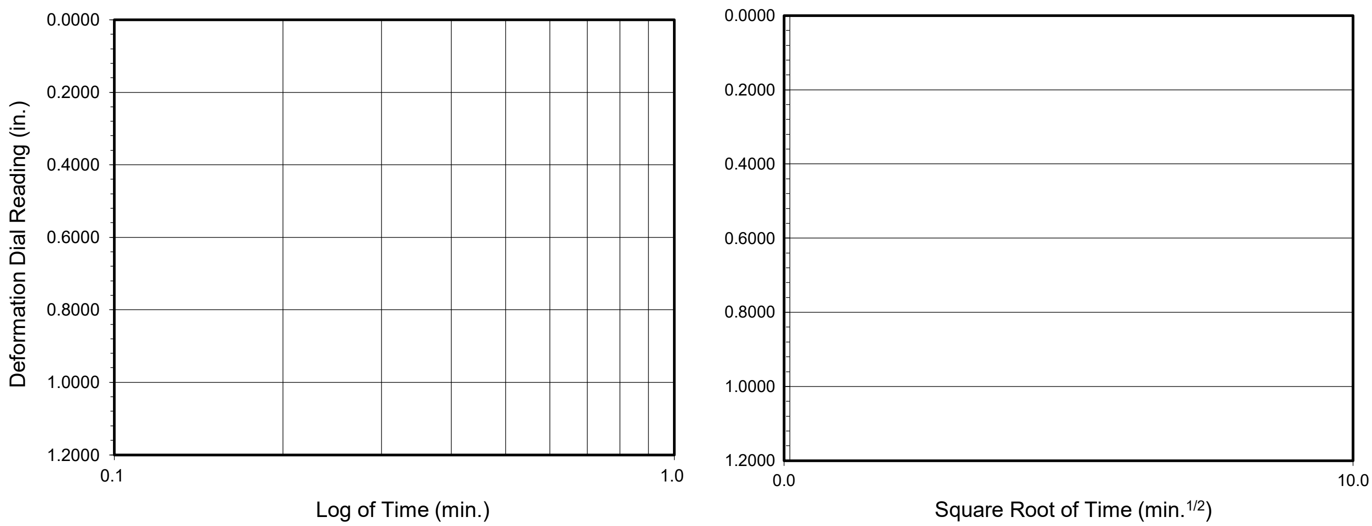


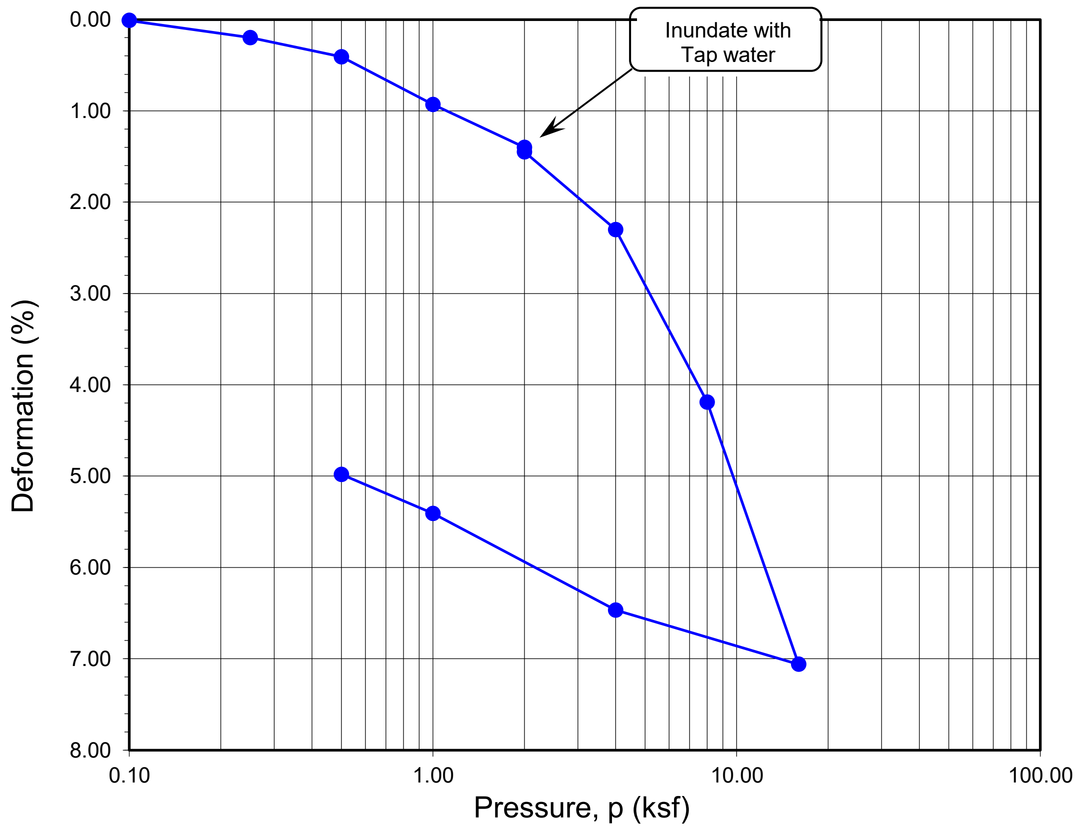
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma-tion (%)
0.10	0.1419	0.9999	0.00	0.01	0.562	0.01
0.25	0.1444	0.9974	0.06	0.26	0.559	0.20
0.50	0.1471	0.9947	0.12	0.53	0.556	0.41
1.00	0.1531	0.9887	0.20	1.13	0.548	0.93
2.00	0.1588	0.9830	0.30	1.70	0.540	1.40
2.00	0.1593	0.9825	0.30	1.75	0.540	1.45
4.00	0.1689	0.9729	0.41	2.71	0.526	2.30
8.00	0.1892	0.9526	0.55	4.74	0.497	4.19
16.00	0.2196	0.9222	0.72	7.78	0.452	7.06
4.00	0.2125	0.9294	0.60	7.07	0.461	6.47
1.00	0.2008	0.9410	0.49	5.90	0.478	5.41
0.50	0.1960	0.9458	0.44	5.42	0.485	4.98

Time Readings

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)

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-1	**R-2**	**7.5**	**13.7**	**18.1**	**107.9**	**111.8**	**0.562**	**0.485**	**66**	**96**

Soil Identification: Yellowish brown lean clay (CL)

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 23221-01

Telegraph Rd Santa Fe Springs_Site 1

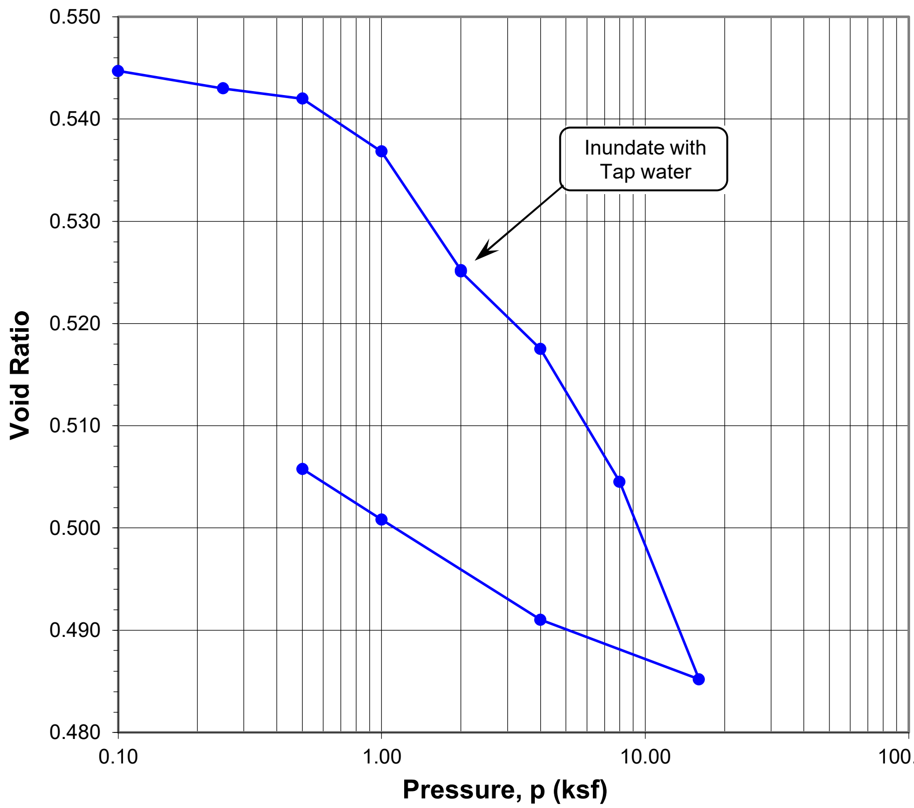
01-24

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS

ASTM D 2435

Project Name:	Telegraph Rd Santa Fe Springs_Site 1	Tested By: GB/JD	Date: 01/04/24
Project No.:	23221-01	Checked By: J. Ward	Date: 01/15/24
Boring No.:	HS-8	Depth (ft.): 10.0	
Sample No.:	R-3	Sample Type: Ring	
Soil Identification:	Brown silty clay (CL-ML)		

Sample Diameter (in.):	2.415
Sample Thickness (in.):	1.000
Weight of Sample + ring (g):	199.00
Weight of Ring (g):	44.70
Height after consol. (in.):	0.9745
Before Test	
Wt. of Wet Sample+Cont. (g):	196.85
Wt. of Dry Sample+Cont. (g):	176.96
Weight of Container (g):	64.16
Initial Moisture Content (%)	17.6
Initial Dry Density (pcf)	109.1
Initial Saturation (%):	87
Initial Vertical Reading (in.)	0.1420
After Test	
Wt. of Wet Sample+Cont. (g):	252.00
Wt. of Dry Sample+Cont. (g):	229.21
Weight of Container (g):	52.21
Final Moisture Content (%)	17.23
Final Dry Density (pcf):	112.9
Final Saturation (%):	94
Final Vertical Reading (in.)	0.1759
Specific Gravity (assumed):	2.70
Water Density (pcf):	62.43

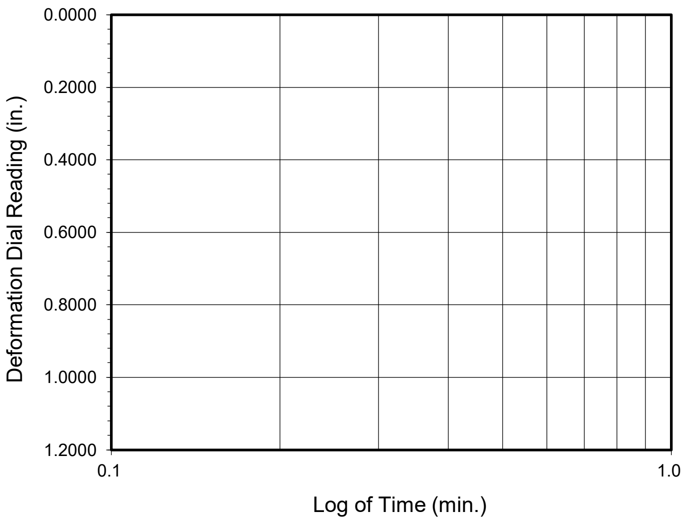


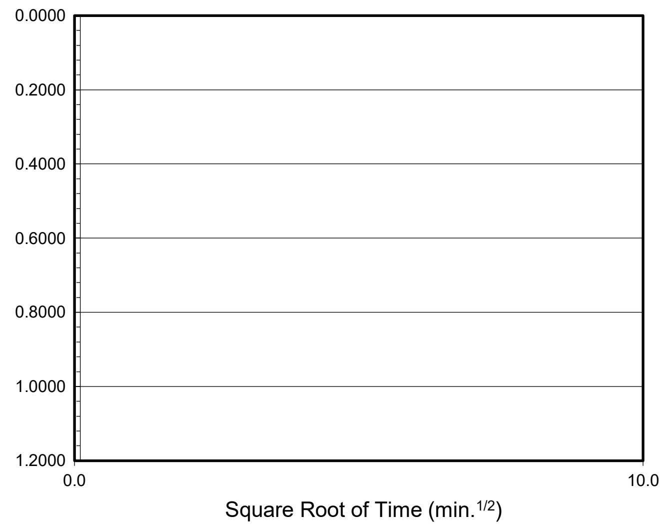
Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.10	0.1423	0.9997	0.00	0.03	0.545	0.03
0.25	0.1448	0.9972	0.14	0.28	0.543	0.14
0.50	0.1471	0.9950	0.30	0.51	0.542	0.21
1.00	0.1523	0.9897	0.49	1.03	0.537	0.54
2.00	0.1614	0.9806	0.65	1.94	0.525	1.29
2.00	0.1615	0.9805	0.65	1.95	0.525	1.30
4.00	0.1680	0.9740	0.81	2.60	0.518	1.79
8.00	0.1778	0.9642	0.95	3.58	0.505	2.63
16.00	0.1917	0.9503	1.09	4.97	0.485	3.88
4.00	0.1870	0.9551	0.99	4.50	0.491	3.51
1.00	0.1796	0.9624	0.89	3.76	0.501	2.87
0.50	0.1759	0.9661	0.84	3.39	0.506	2.55

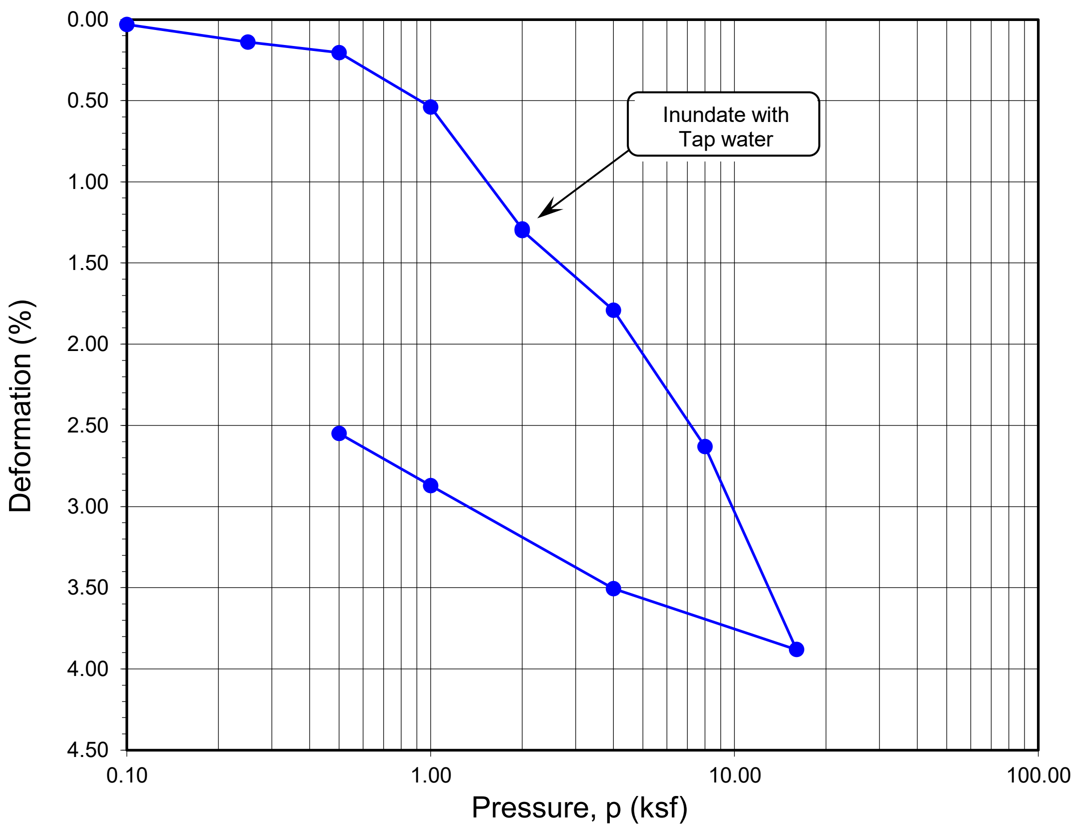
Time Readings

Date	Time	Elapsed Time (min)	Square Root of Time	Dial Rdgs. (in.)

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-8	**R-3**	**10**	**17.6**	**17.2**	**109.1**	**112.9**	**0.545**	**0.506**	**87**	**94**

Soil Identification: Brown silty clay (CL-ML)

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS
ASTM D 2435

Project No.: 23221-01

Telegraph Rd Santa Fe Springs_Site 1

01-24

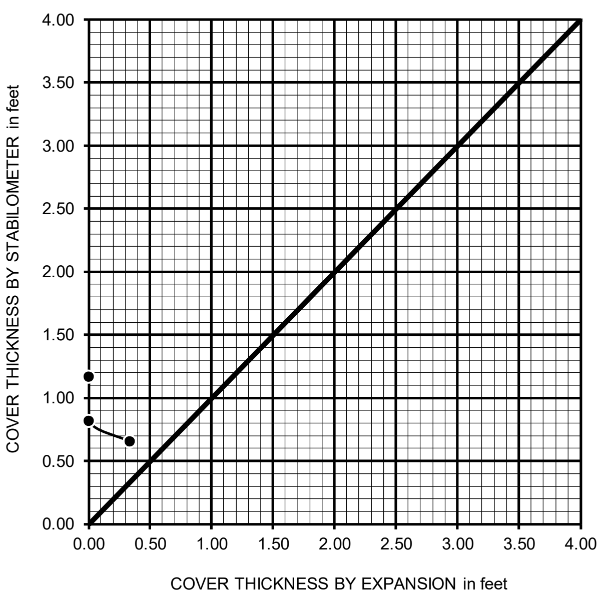
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	Telegraph Rd Santa Fe Springs_Site 1	PROJECT NUMBER:	23221-01
BORING NUMBER:	HS-2	DEPTH (FT.):	0-5
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Dark brown sandy silt s(ML)	DATE COMPLETED:	1/9/2024

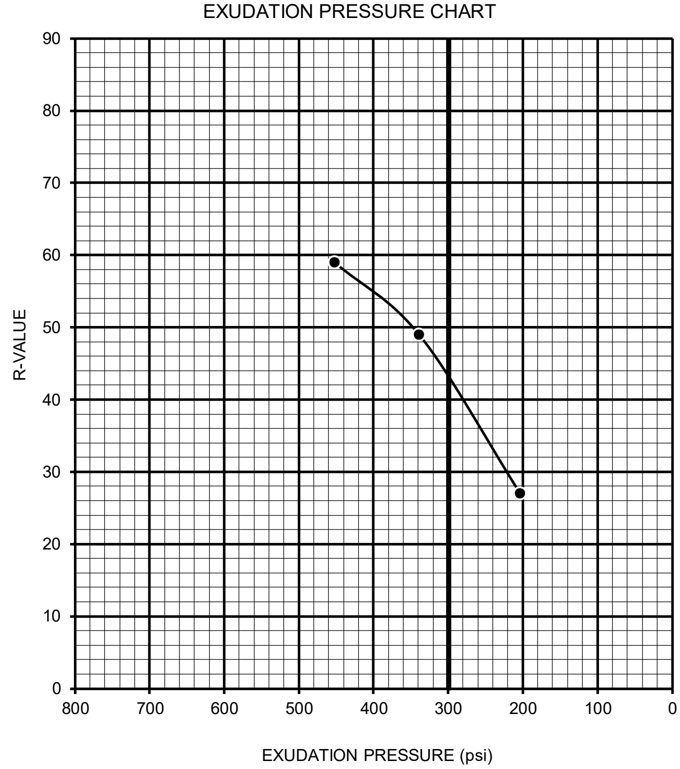
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	11.0	11.6	12.5
HEIGHT OF SAMPLE, Inches	2.46	2.48	2.54
DRY DENSITY, pcf	124.7	123.6	121.3
COMPACTOR PRESSURE, psi	180	120	70
EXUDATION PRESSURE, psi	452	339	204
EXPANSION, Inches x 10exp-4	10	0	0
STABILITY Ph 2,000 lbs (160 psi)	45	56	94
TURNS DISPLACEMENT	4.48	4.75	4.85
R-VALUE UNCORRECTED	59	49	27
R-VALUE CORRECTED	59	49	27

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.	0.66	0.82	1.17
EXPANSION PRESSURE THICKNESS, ft.	0.33	0.00	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	64
R-VALUE BY EXUDATION:	43
EQUILIBRIUM R-VALUE:	43

Appendix D
Infiltration Results

Infiltration Test Data Sheet

LGC Geotechnical, Inc

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

Project Name: EPD - Santa Fe Springs (Site 1)
Project Number: 23221-01
Date: 12/19/2023
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	14:22	14:52	30.0	6.44	6.48	0.04	

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	14:54	15:24	30.0	6.48	6.54	0.06	7.72	0.1
2	15:26	15:56	30.0	6.45	6.50	0.05	7.78	0.1
3	15:58	16:28	30.0	6.43	6.46	0.03	7.83	0.0
4	16:30	17:00	30.0	6.48	6.49	0.01	7.72	0.0
5	17:00	17:30	30.0	6.47	6.48	0.01	7.74	0.0
6	17:46	18:16	30.0	6.45	6.46	0.01	7.78	0.0
7								
8								
9								
10								
11								
12								

Measured Infiltration Rate	**0.0**
Reduction Factor	See Report
Design Infiltration Rate	**See Report**

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: EPD - Santa Fe Springs (Site 1)
Project Number: 23221-01
Date: 12/19/2023
Location: I-2

Test hole dimensions (if circular)	
Boring Depth (feet)*:	15
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	14:25	14:55	30.0	11.58	13.51	1.93	

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	14:57	15:27	30.0	12.53	13.57	1.04	5.52	1.6
2	15:29	15:59	30.0	12.77	14.00	1.23	5.02	2.1
3	16:01	16:31	30.0	12.41	13.37	0.96	5.77	1.4
4	16:33	17:03	30.0	12.43	13.23	0.80	5.73	1.2
5	17:05	17:35	30.0	12.60	13.38	0.78	5.38	1.2
6	17:51	18:21	30.0	12.55	13.31	0.76	5.48	1.2
7								
8								
9								
10								
11								
12								

Measured Infiltration Rate	**1.2**
Reduction Factor	See Report
Design Infiltration Rate	**See Report**

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: EPD - Santa Fe Springs (Site 1)
Project Number: 23221-01
Date: 12/19/2023
Location: I-3

Test hole dimensions (if circular)	
Boring Depth (feet)*:	15
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	14:30	15:00	30.0	11.91	12.46	0.55	

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	15:02	15:32	30.0	12.11	12.49	0.38	6.40	0.5
2	15:34	16:04	30.0	12.32	12.76	0.44	5.96	0.6
3	16:06	16:36	30.0	12.15	12.52	0.37	6.32	0.5
4	16:38	17:08	30.0	12.06	12.47	0.41	6.51	0.5
5	17:10	17:40	30.0	12.19	12.57	0.38	6.23	0.5
6	17:55	18:25	30.0	12.23	12.62	0.39	6.15	0.5
7								
8								
9								
10								
11								
12								

Measured Infiltration Rate	**0.5**
Reduction Factor	See Report
Design Infiltration Rate	**See Report**

Sketch:

Notes:

Based on Guidelines from: LA County dated 06/2021

Spreadsheet Revised on: 6/22/2023

LGC Geotechnical, Inc.

Appendix E
General Earthwork and Grading Specifications for Rough Grading

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 sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

4.5 Compaction Testing

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

4.6 Frequency of Compaction Testing

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

4.7 Compaction Test Locations

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

5.0 Subdrain Installation

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

6.0 Excavation

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

7.0 Trench Backfills

7.1 The Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

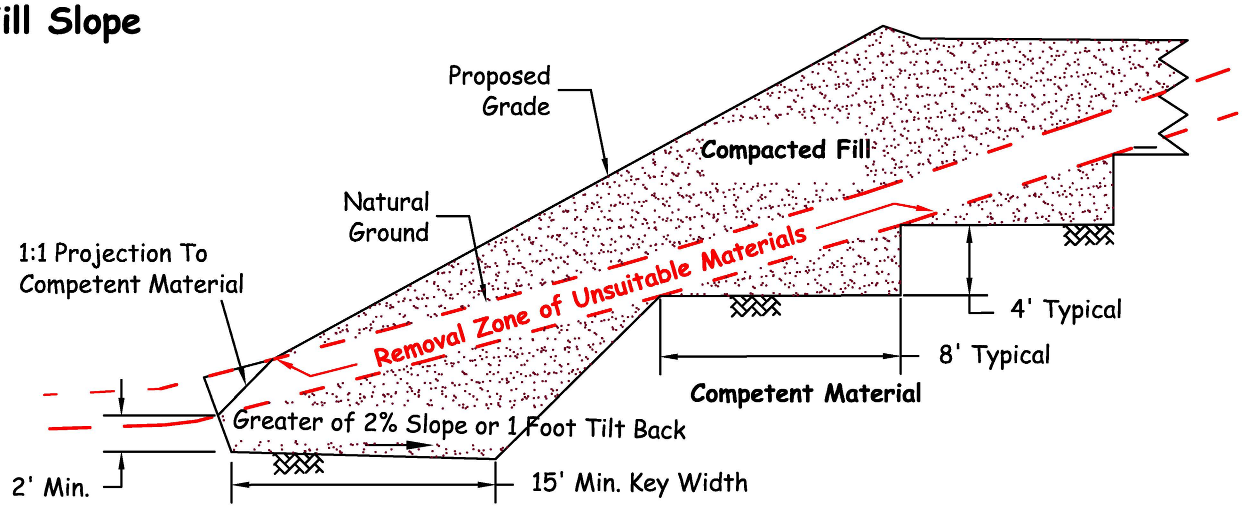
7.2 All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of maximum from 1 foot above the top of the conduit to the surface.

7.3 The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

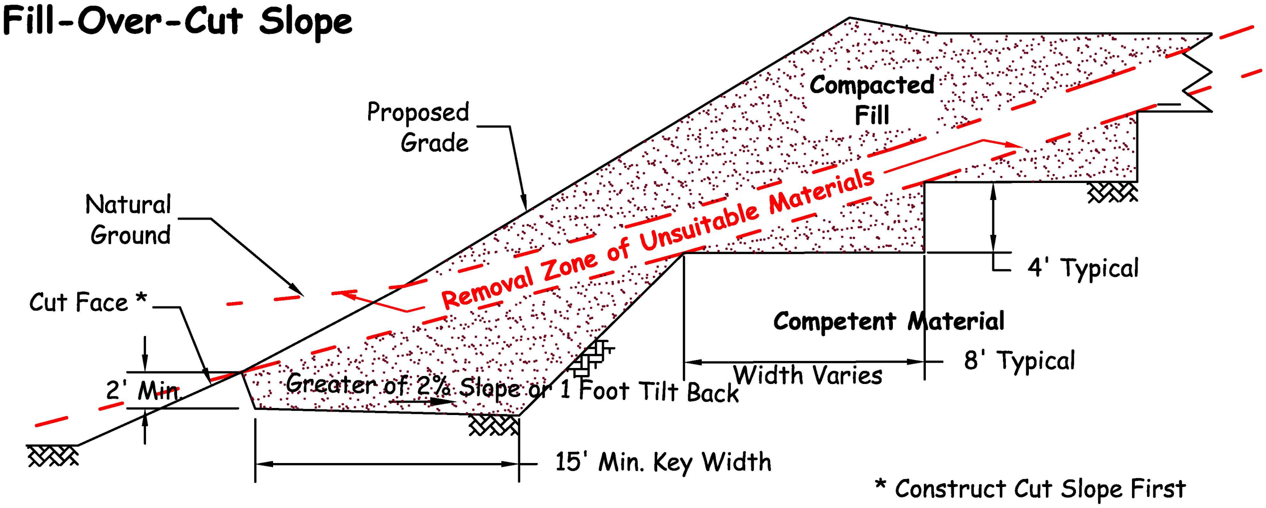
7.4 The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

7.5 Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

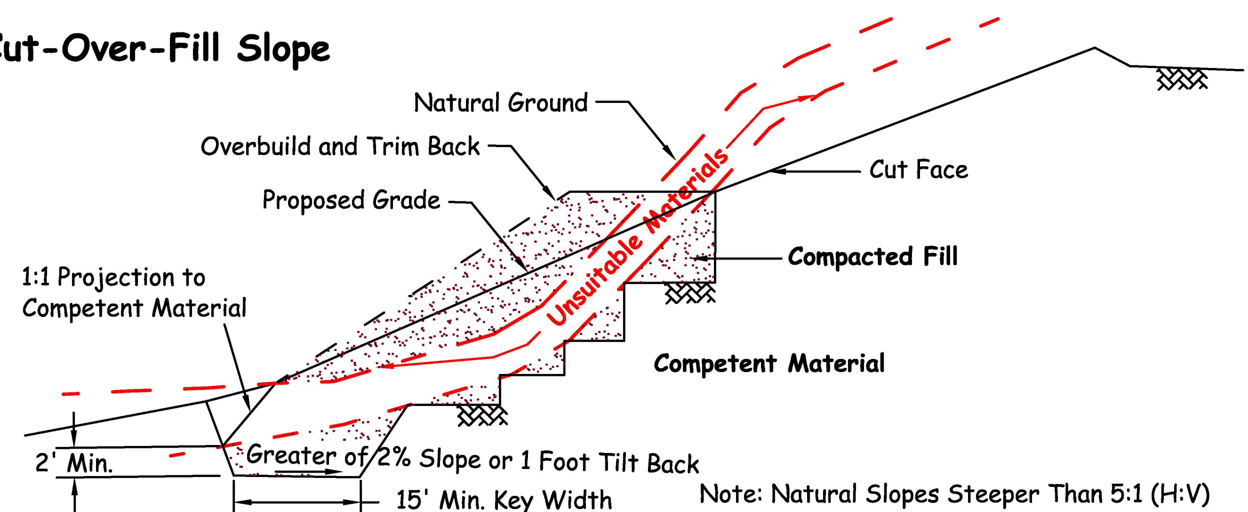
Fill Slope



Fill-Over-Cut Slope



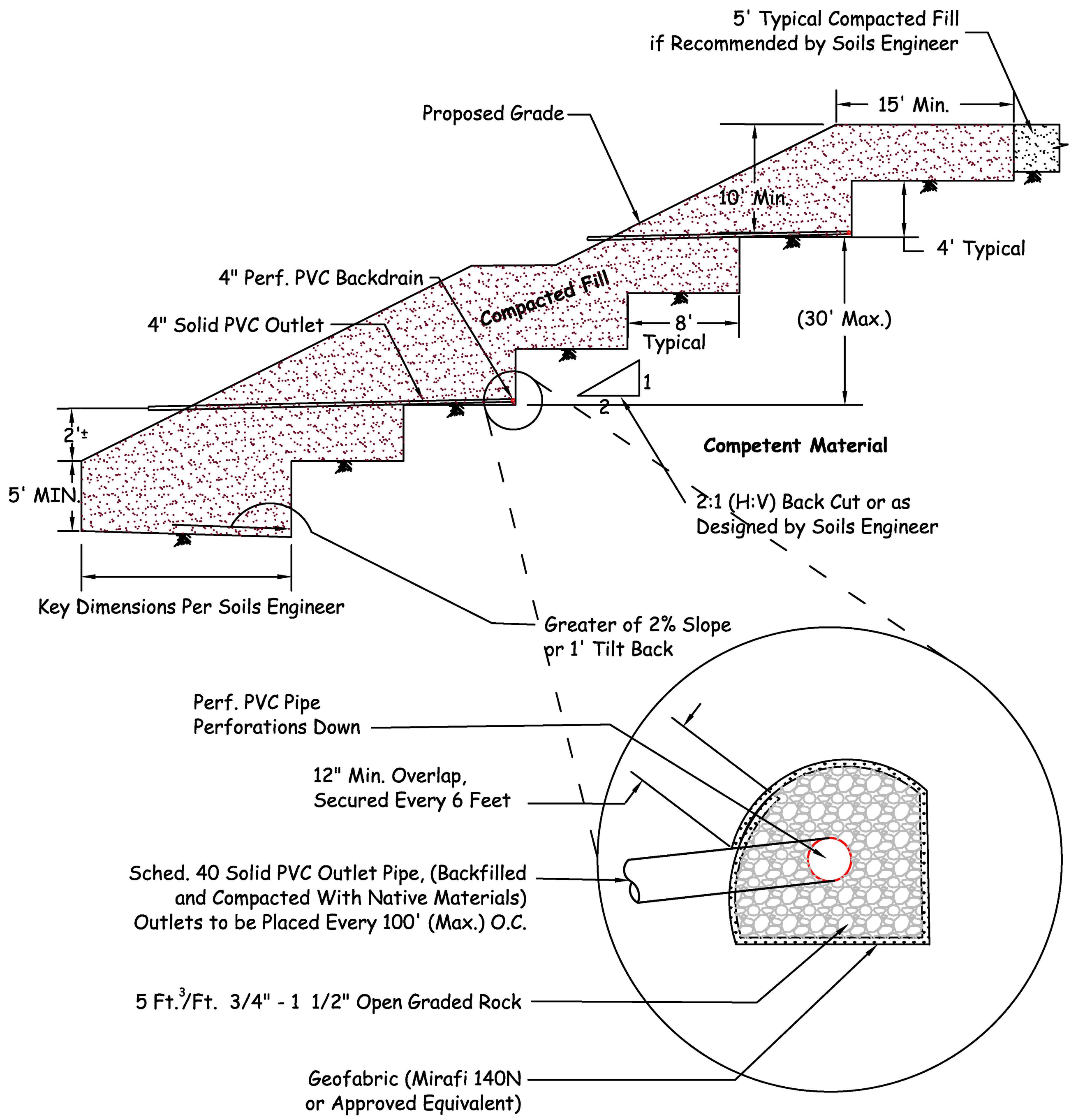
Cut-Over-Fill Slope



Note: Natural Slopes Steeper Than 5:1 (H:V) Must Be Benched.

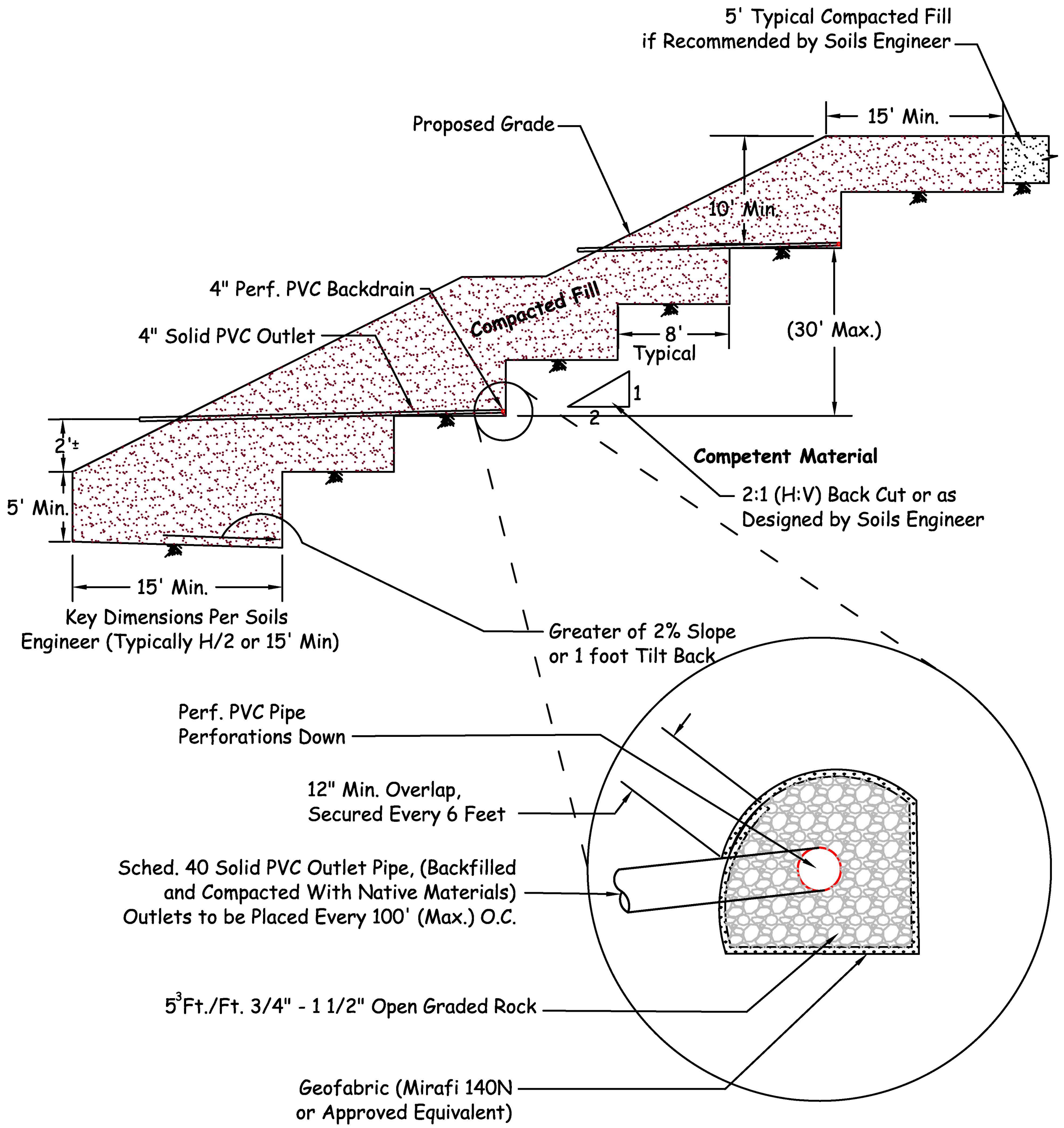
LGC Geotechnical, Inc.

KEYING AND BENCHING



LGC Geotechnical, Inc.

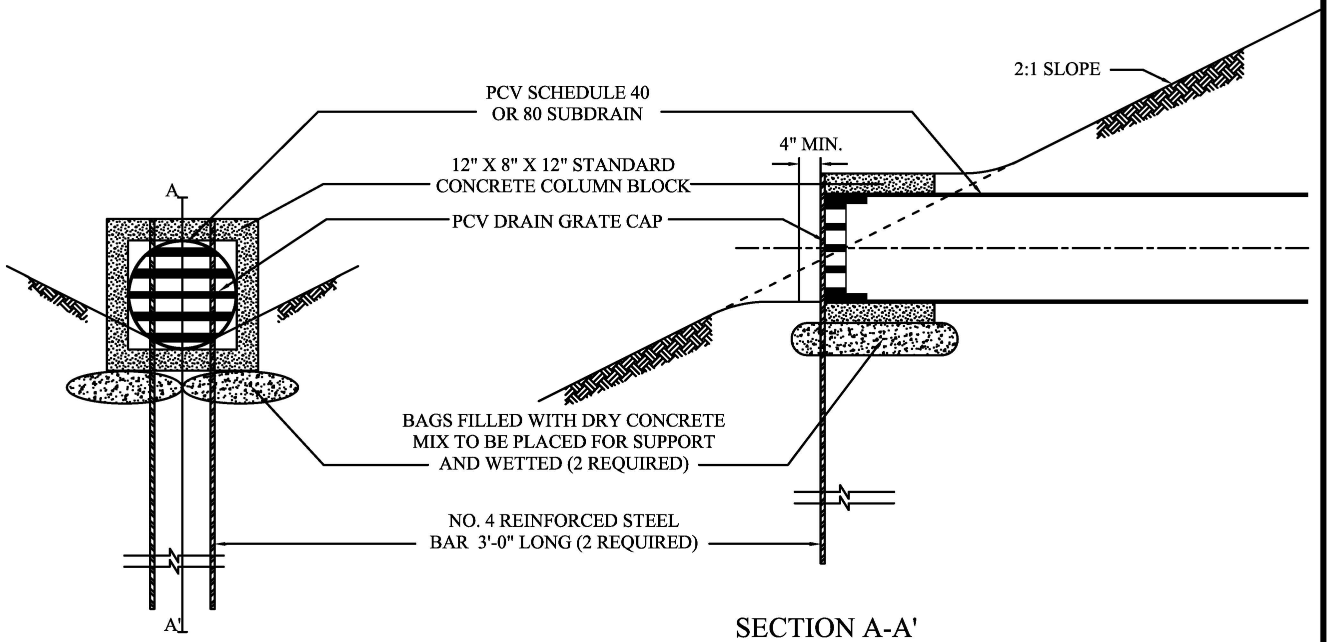
TYPICAL BUTTRESS DETAIL



LGC Geotechnical, Inc.

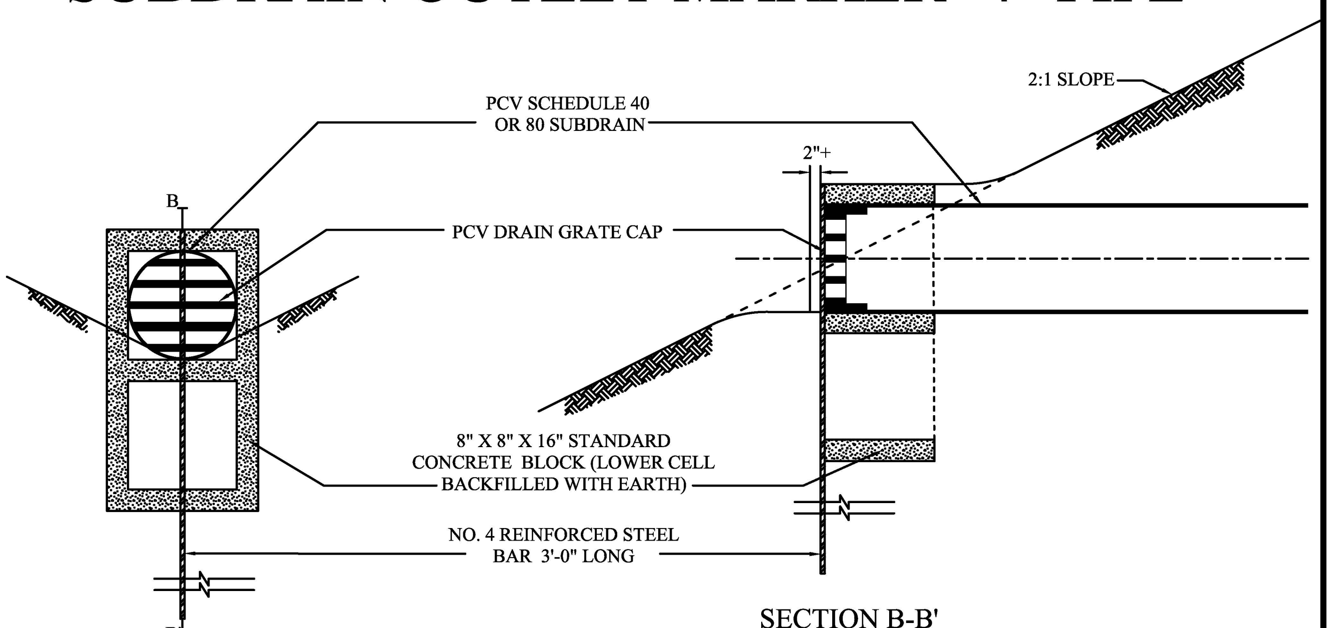
TYPICAL STABILIZATION FILL DETAIL

SUBDRAIN OUTLET MARKER -6" & 8" PIPE



SECTION A-A'

SUBDRAIN OUTLET MARKER -4" PIPE



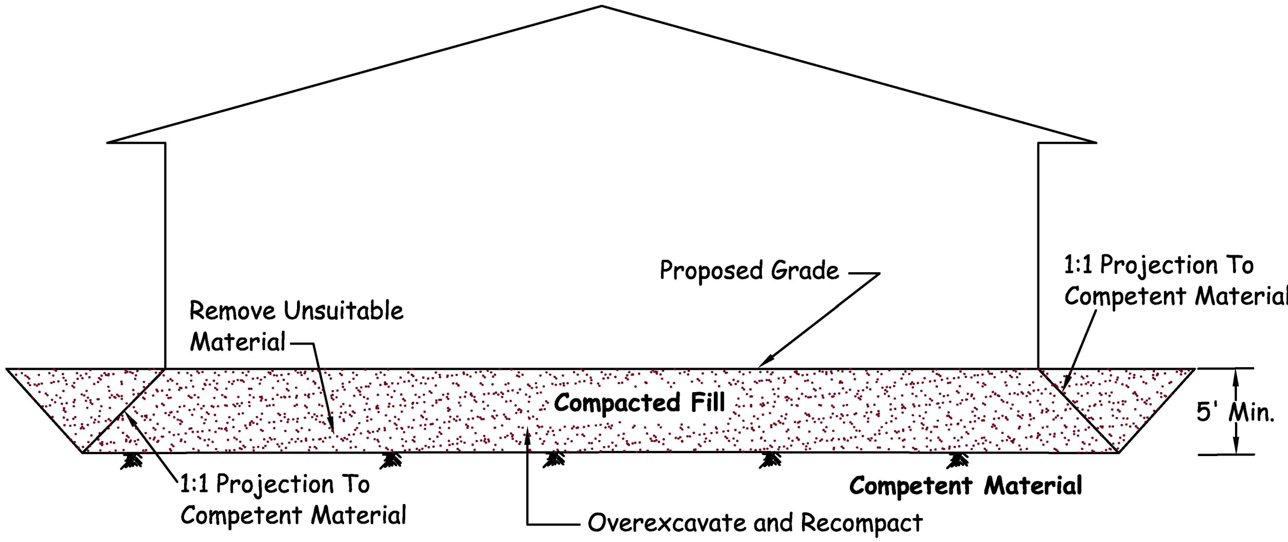
SECTION B-B'

NOT TO SCALE

LGC Geotechnical, Inc.

SUBDRAIN OUTLET MARKER DETAIL

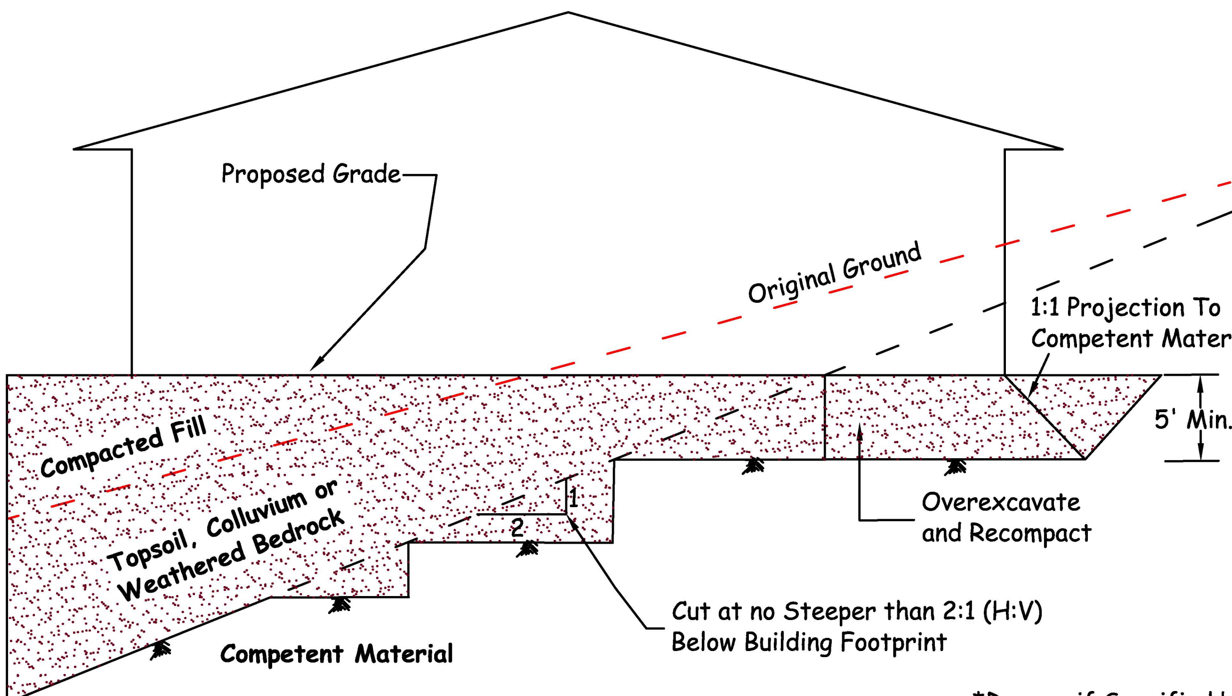
Cut Lot
(Exposing Unsuitable Soils at Design Grade)



Note 1: Removal Bottom Should be Graded With Minimum 2% Fall Towards Street or Other Suitable Area (as Determined by Soils Engineer) to Avoid Ponding Below Building

Note 2: Where Design Cut Lots are Excavated Entirely Into Competent Material, Overexcavation May Still be Required for Hard-Rock Conditions or for Materials With Variable Expansion Characteristics.

Cut/Fill Transition Lot



*Deeper if Specified by Soils Engineer

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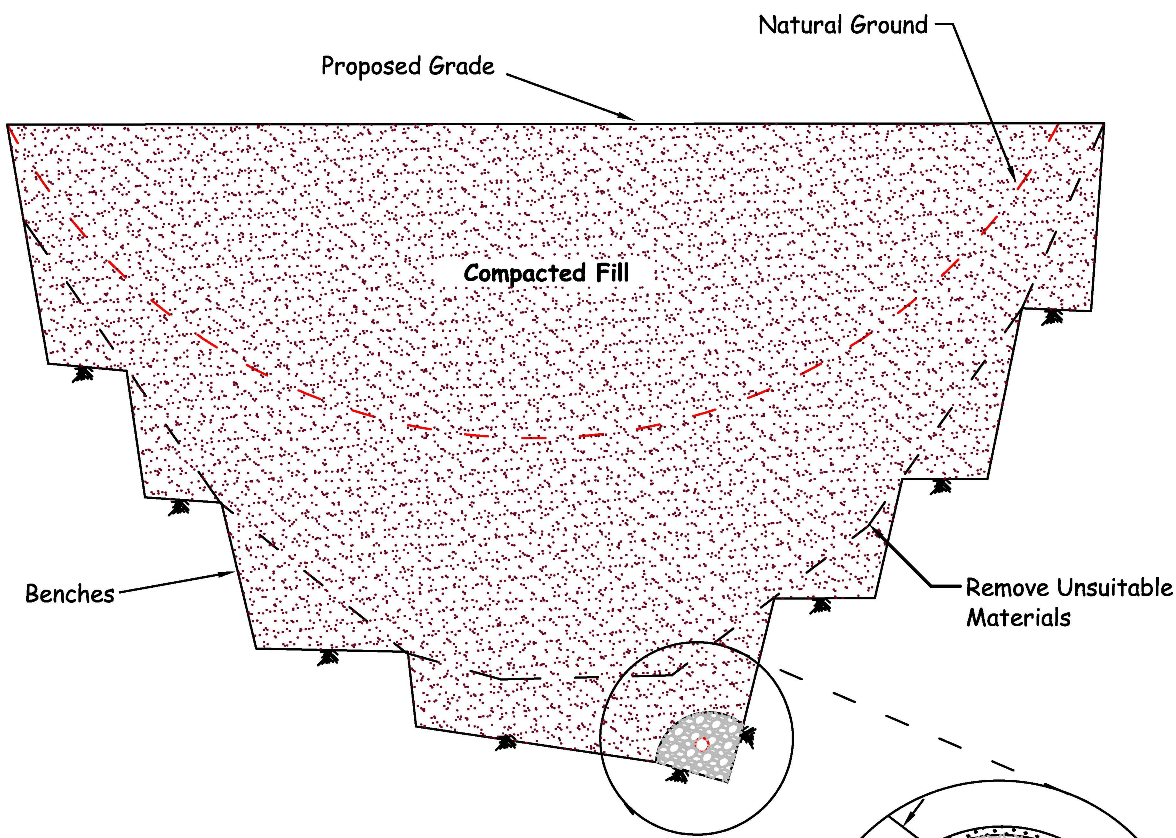
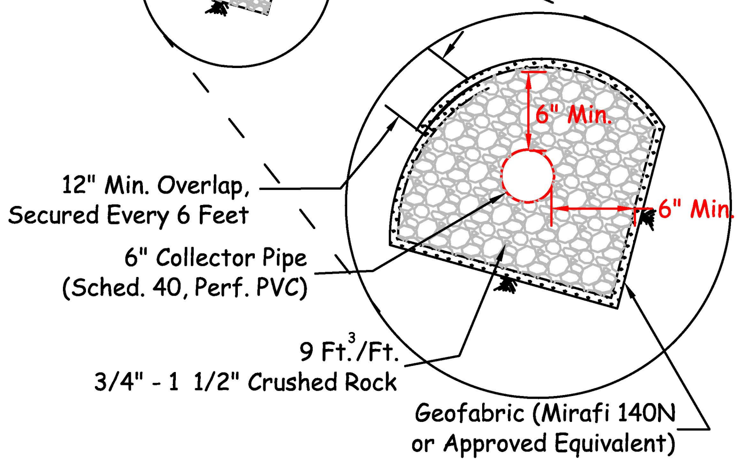
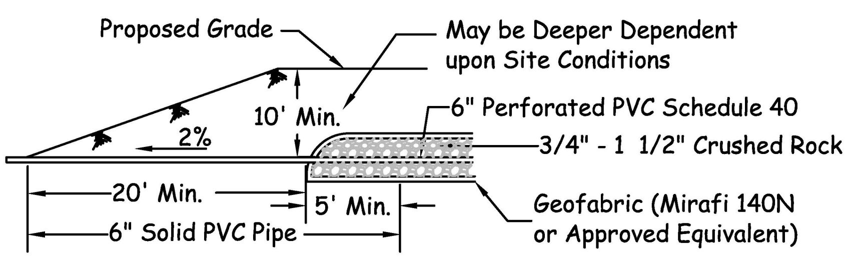
CUT AND TRANSITION LOT OVEREXCAVATION DETAIL





Notes:
1) Continuous Runs in Excess of 500' Shall Use 8" Diameter Pipe.
2) Final 20' of Pipe at Outlet Shall be Solid and Backfilled with Fine-grained Material.

Proposed Outlet Detail



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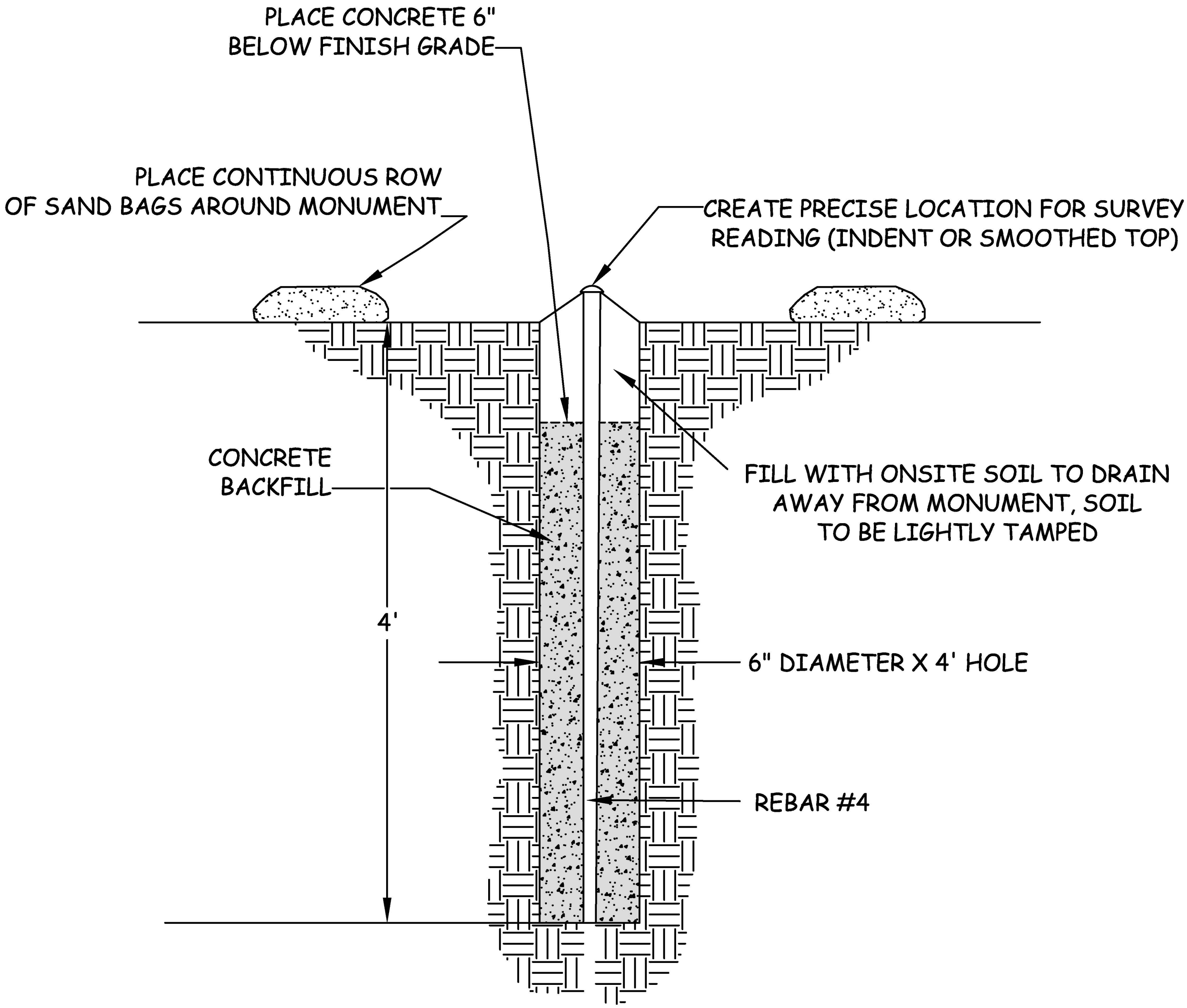
CANYON SUBDRAINS



NO CONSTRUCTION EQUIPMENT WITHIN 25 FEET
OF ANY INSTALLED SETTLEMENT MONUMENTS

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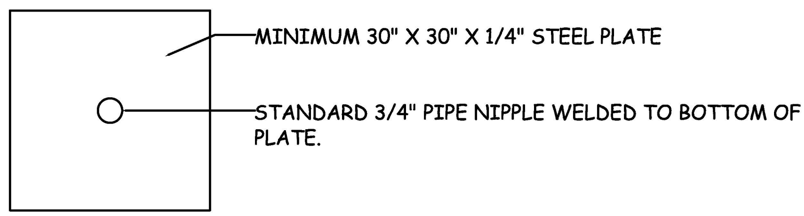
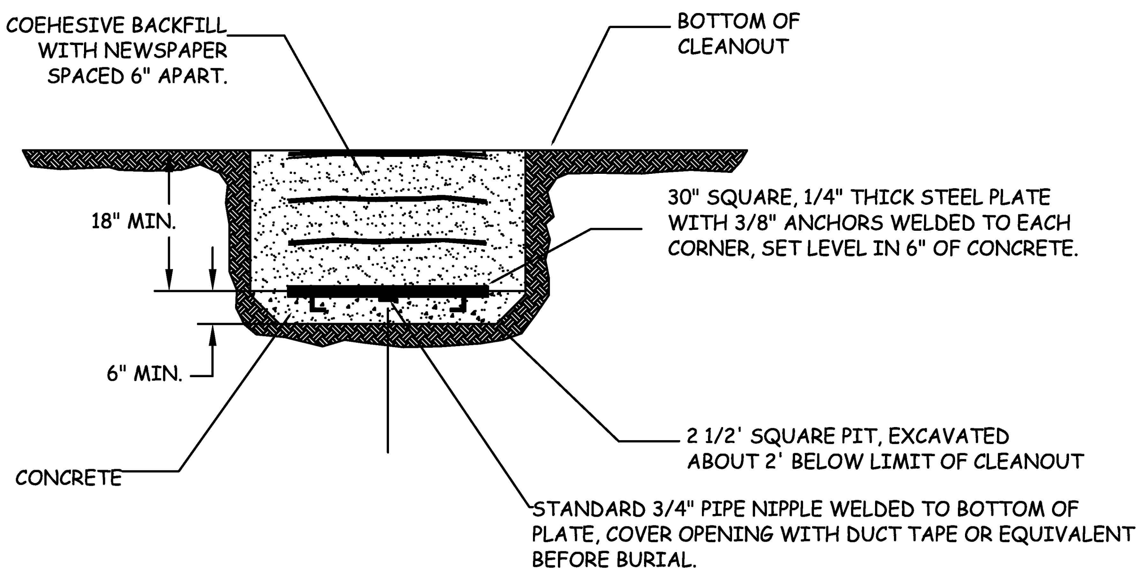
TYPICAL SURFACE SETTLEMENT MONUMENT

TOP VIEW





1. SURVEY FOR HORIZONTAL AND VERTICAL LOCATION TO NEAREST .01 INCH PRIOR TO BACKFILL USING KNOW LOCATIONS THAT WILL REMAIN INTACT DURING THE DURATION OF THE MONITORING PROGRAM. KNOW POINTS EXPLICITELY NOT ALLOWED ARE THOSE LOCATED ON FILL OR THAT WILL BE DESTROYED DURING GRADING.
2. IN THE EVENT OF DAMAGE TO SETTLEMENT PLATE DURING GRADING, CONTRACTOR SHALL IMMEDIATELY NOTIFY THE GEOTECHNICAL ENGINEER AND SHALL BE RESPONSIBLE FOR RESTORING THE SETTLEMENT PLATES TO WORKING ORDER.
3. DRILL TO RECOVER AND ATTACH RISER PIPE.

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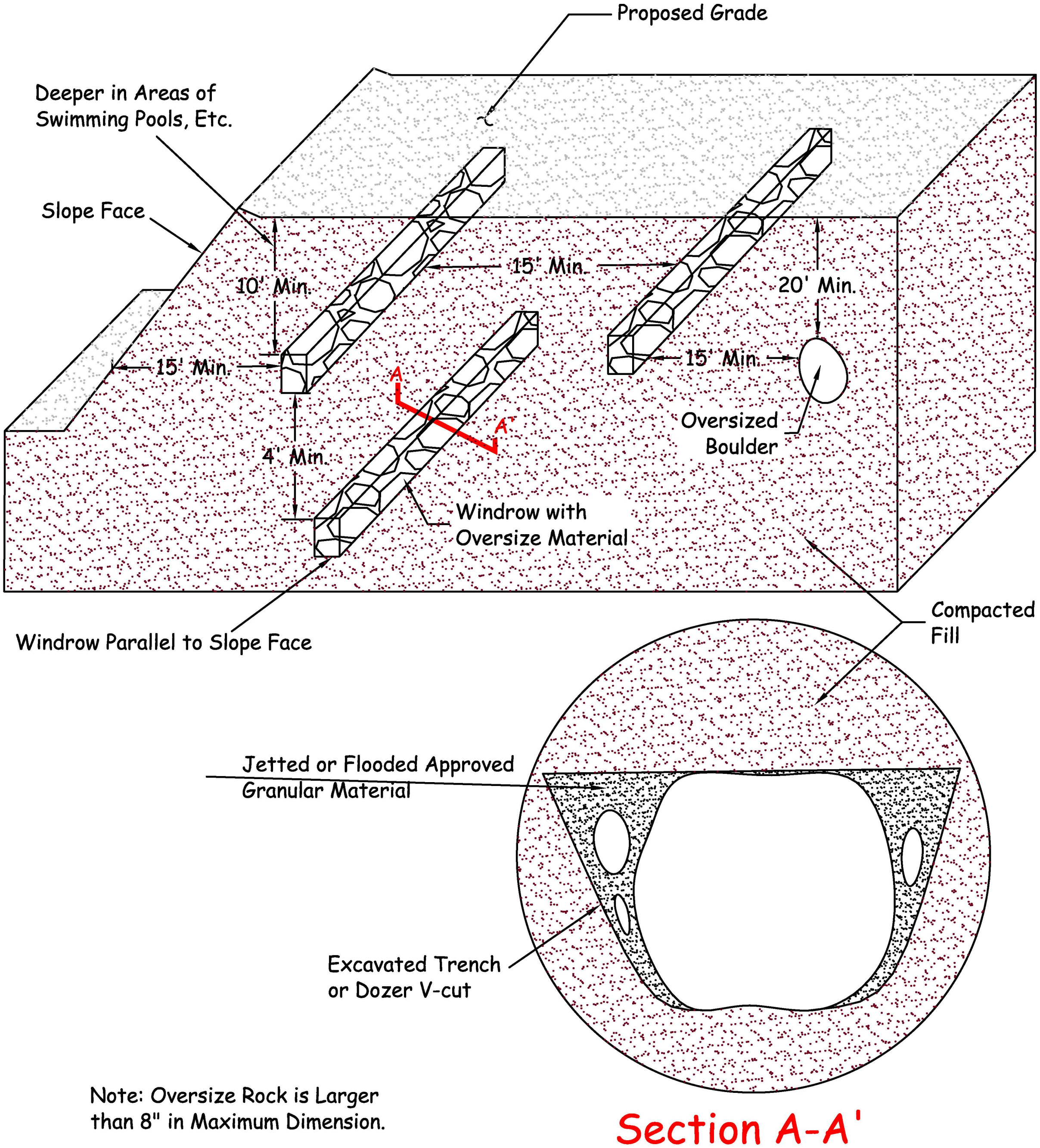
TYPICAL SETTLEMENT PLATE AND RISER



Note: Oversize Rock is Larger than 8" in Maximum Dimension.

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OVERSIZE ROCK DISPOSAL DETAIL