LGC Geotechnical, Inc.

July 1, 2024 Project No. 23174-01

Roman Catholic Bishop of Orange
13280 Chapman Avenue
Garden Grove, CA 92840

Subject: ***Preliminary Geotechnical Subsurface Evaluation, Proposed Church Development, 14010 Remington, Irvine, California***

In accordance with your request, LGC Geotechnical, Inc. has performed a geotechnical subsurface evaluation for the proposed church development located at 14010 Remington in the City of Irvine, 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 church project.

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,

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1.0 INTRODUCTION

1.1 Purpose and Scope of Services

This report presents the results of our geotechnical subsurface evaluation for the proposed church development located at 14010 Remington in the City of Irvine, California (see Site Location Map, Figure 1). The purpose of our work was to collect subsurface data in order to confirm that the site can be developed from a geotechnical perspective. Our scope of services included:

- Review of pertinent readily available geotechnical information and geologic maps (Appendix A).
- Subsurface evaluation including excavation, sampling, and logging of four small-diameter hollow stem borings.
- Infiltration testing at two of the small-diameter hollow-stem borings.
- Laboratory testing of representative samples obtained during our subsurface evaluation (Appendix C).
- Geotechnical analysis and evaluation of the data obtained, including:
  - Suitability of the site for the proposed development from a geotechnical standpoint;
  - Description of the site geology, and subsurface soil and groundwater conditions;
  - Evaluation of the seismic conditions at the site, including seismic design criteria based on the 2022 California Building Code (CBC); and
  - Recommendations for remedial grading operations and site preparation.
- Preparation of this report presenting our findings, conclusions and recommendations with respect to the proposed site development.

1.2 Background/Project Description

The approximately 3-acre site is bound to the north by Trabuco Road, to the west by Remington, to the south by a parking lot for the associated church and open space and east by a residential development. Review of historical aerial photographs suggests the site was undeveloped farmland prior to 1980. By 1985, Roosevelt was constructed but the site remained undeveloped until 2000. The church and associated parking lot was completed by 2002. The site has essentially remained unchanged since then.

The proposed development will consist of a new parish hall, plaza, church, rectory, parking lot and other associated improvements. We understand that the proposed church development will have a basement approximately 9 feet deep while the other structures will be at grade with relatively light building loads (column and wall loads assumed to be a maximum of approximately 30 kips and 3 kips per lineal foot, respectively). At this time, we do not have any design cut/fill information, but we assume maximum design cuts will be a maximum of approximately 10 feet (for basement) and design fills will be less than 5 feet.

The recommendations provided herein are based upon the estimated structural loading, potential grade changes, and expected 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 (including grading, foundation, retaining/basement walls, etc.) and any changes to the assumed structural loads when they become available, in order to either confirm or modify the recommendations provided herein.

Note that an evaluation of the adjacent proposed residential site (LGC Geotechnical, 2024) was conducted at a similar time to the subject proposed church site. The limits of these sites can be found in Figure 2 - Boring Location Map. The laboratory test results from the adjacent residential evaluation were also included within this report.

1.3 ***Subsurface Evaluation***

LGC Geotechnical performed a subsurface geotechnical evaluation of the site consisting of the excavation of 4 hollow-stem auger borings.

Four hollow-stem borings (HS-5, HS-6, I-3, and I-4) were drilled to depths ranging from approximately 10 to 70 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 track-mounted drill rig equipped with 8-inch-diameter hollow-stem augers. 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.

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

1.4 ***Field Percolation Testing***

Two falling head field percolation tests (I-3 and I-4) were performed in the approximate locations indicated on our Geotechnical Map (Figure 2). Estimation of infiltration rates for the site was accomplished in general accordance with the guidelines set forth by the County of Orange County (2013). A 3-inch diameter perforated PVC pipe with filter sock was placed in the borehole, and the annulus was backfilled with gravel, including placement of approximately 2 inches of gravel at the bottom of the borehole. The infiltration wells were pre-soaked the day prior to testing. During the pre-test, if the water level drops more than 6 inches in 25 minutes for two consecutive readings, the test procedure for coarse-grained soils was followed. If the water level did not drop 6 inches in one or both pre-test readings, the procedure for fine-grained soils was followed. The procedure for coarse-grained soils requires performing the test for one hour and taking one reading every 10 minutes from a fixed reference point. The procedure for fine-grained soils requires performing the test for six hours and taking one reading every 30 minutes from a fixed reference point.

The pre-tests indicated the procedure for the fine-grained soils should be followed. The calculated infiltration is normalized relative to the three-dimensional flow that occurs within the field test to a one-dimensional flow out of the bottom of the boring only (i.e., "Porchet Method"). The measured infiltration rates (for feasibility purposes only) are provided in Table 1 below. Please note that infiltration is discussed in Section 4.8 and field data is provided in Appendix D.

Please note that the values provided in Table 1 do not include reduction factors associated with the test procedure, site variability, and long-term siltation plugging that are used to calculate the design infiltration rate.

TABLE 1

Summary of Field Infiltration Testing

Infiltration Test No.	Approx. Depth Below Existing Grade (ft)	Measured Infiltration Rate* (in./hr.)
I-3	10	0.02
I-4	10	0.01

*Measured Infiltration Rates Do Not Include Factor of Safety.

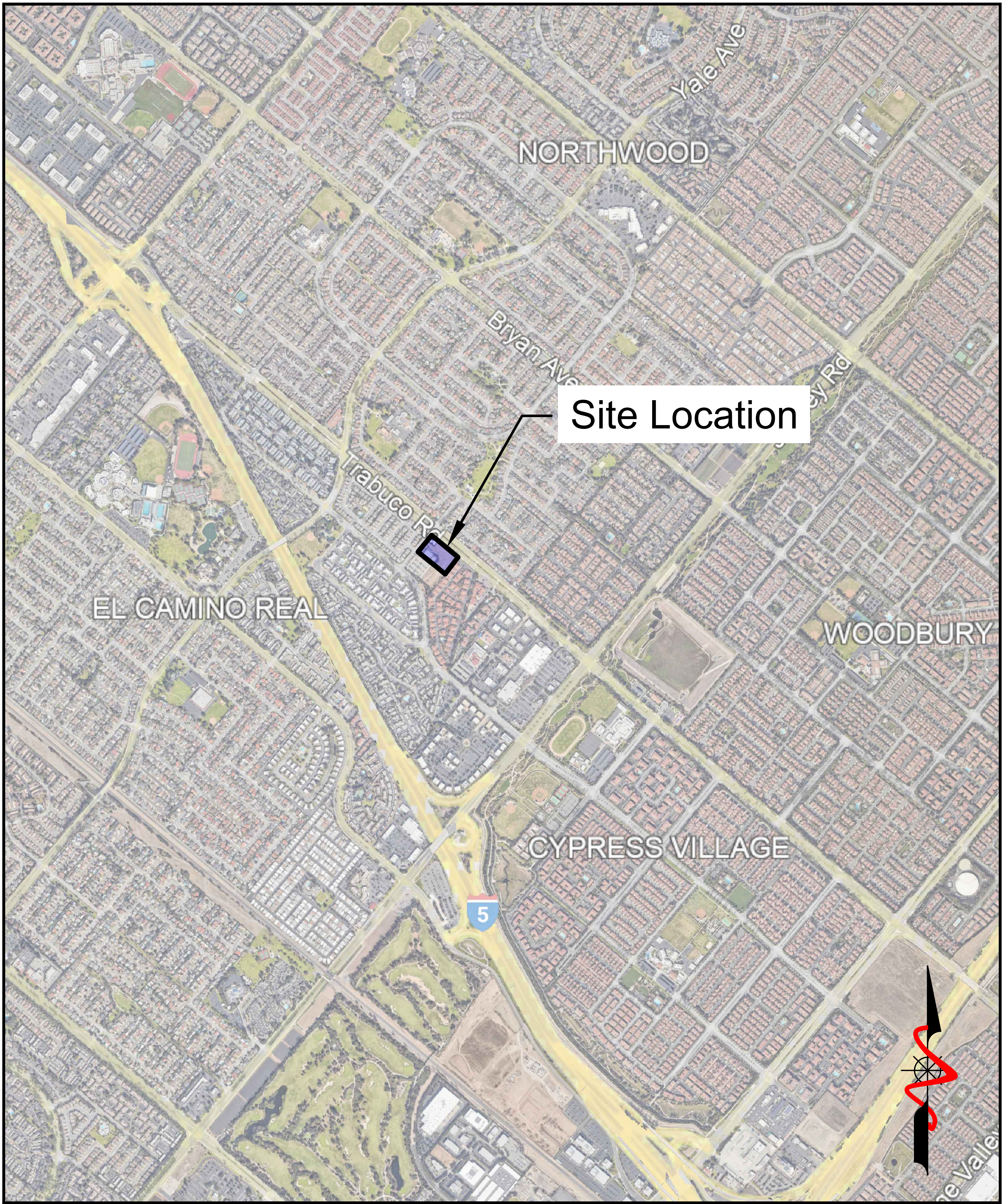
1.5 Laboratory Testing

Laboratory testing was performed on representative soil samples obtained from our subsurface evaluation. Laboratory testing included in-situ moisture and density tests, fines content, expansion index, laboratory compaction, collapse and corrosion testing.

The following is a summary of the laboratory test results.

- Dry density of the samples collected ranged from approximately 80 pounds per cubic foot (pcf) to 106 pcf, with an average of approximately 94 pcf. Field moisture contents ranged from approximately 1 percent to 24 percent, with an average of approximately 13 percent.
- Two samples were tested for fines content indicated a fines content (passing No. 200 sieve) ranging from 48 to 68 percent. According to the Unified Soils Classification System (USCS), the tested samples are classified as "coarse" and "fine-grained" soil.
- Two Expansion Index (EI) tests were performed. The result indicated an EI value ranging from 112 to 135, corresponding to "High" to "Very High" expansion potential.
- Two laboratory compaction tests of near surface samples indicated a maximum dry density ranging from 115.0 to 117.5 with an optimum moisture content ranging from 12.5 to 14.0 percent.
- Six swell/collapse tests were performed. The plots are provided in Appendix C.
- Corrosion testing indicated soluble sulfate content of 95 parts per million (ppm), chloride content of 60 ppm, and pH value of 7.90, and minimum resistivity of 720 ohm-cm.

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



LGC Geotechnical, Inc.

FIGURE 1
Site Location Map

PROJECT NAME	Our Lady of Peace, Irvine
PROJECT NO.	23174-01
ENG. / GEOL.	DJB
SCALE	Not to Scale
DATE	July 2024

2.0 GEOTECHNICAL CONDITIONS

2.1 Regional Geology

The subject site is located along the southeastern margin of the Los Angeles Basin, a large structural depression within the Peninsular Ranges geomorphic province of California. The southeastern portion of Los Angeles Basin includes the Tustin Plain, a shallow alluvial portion of the coastal plain. The site is located in the eastern portion of the Tustin Plain in the Irvine subbasin, generally composed of approximately 200 to 700 feet of unconsolidated to semi-consolidated Pliocene, Quaternary, and Holocene-age alluvial sediments. Soils within this zone consist predominantly of interbedded discontinuous lenses of clay, silt, sand and gravel.

Regional geologic mapping and local topographic expressions do not indicate the presence of large-scale landslides within or adjacent to the project area.

2.2 Site Geology and Generalized Subsurface Conditions

Based on regional mapping (Morton, 2006) and our observations of the site, the subject site is underlain by Young Alluvial Fan Deposits (Qyf). In portions of the site, the young alluvial deposits are overlain by a thin veneer of older artificial fill that was placed during the previous development of the site. The depth of this older artificial fill was found to range from approximately 1 to 2 feet below existing grade during our evaluation but may be found at a deeper depth during construction, especially beneath the footprint of the existing church building. As indicated in our field explorations, site soils generally consisted of medium dense to dense clayey sands/silty sands and stiff to hard sandy clays.

It should be noted that borings are only representative of the location and time where/when they are performed, and varying subsurface conditions may exist outside of the performed locations. In addition, subsurface conditions can change over time. The soil descriptions provided should not be construed to mean that the subsurface profile is uniform, and that soil is homogenous within the project area. Descriptions of the subsurface conditions are presented on the boring and geotechnical trench logs located in Appendix B.

2.3 Groundwater

Groundwater was encountered at a depth of approximately 54 feet below existing grade. Historic high groundwater is estimated to be greater than 40 feet below existing ground surface (CGS, 2001).

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

2.4 Faulting and Seismic Hazards

Prompted by damaging earthquakes in California, State legislation and policies concerning the classification and land-use criteria associated with faults have been developed. Their purpose was to prevent the construction of urban developments across the trace of active faults, resulting in the Alquist-Priolo Earthquake Fault Zoning Act. Earthquake Fault Zones have been delineated along the traces of active faults 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 mitigate the hazards associated with active faulting by identifying the location of active faults and allowing for a setback from zones of previous ground rupture.

The subject site is not located within an Alquist-Priolo Earthquake Fault Zone and no faults were identified on the site during our site evaluation. 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 San Joaquin Hills, Whittier, Elsinore, Newport-Inglewood, among others (CGS, 2018). A discussion of these secondary effects is provided in the following sections.

2.4.1 Liquefaction and Dynamic Settlement

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

Based on our review of the State of California Seismic Hazard Zone for liquefaction potential (CDMG, 2001), the subject site is not located within a liquefaction hazard zone. Based on our evaluation, site soils are generally not susceptible to liquefaction due to the fine-grained nature of the on-site soils and low probability of groundwater in the upper 50 feet. Therefore, liquefaction potential is considered low.

2.4.2 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 site being relatively flat, low liquefaction potential and the lack of an adjacent "free face" to drive lateral spreading, the potential for lateral spreading is considered very low.

2.5 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 33.700290 degrees north and longitude -117.768356 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.

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)	1.258g	From SEAOC, 2024
S_1 (Risk-Targeted Spectral Accelerations for 1-Second Periods)	0.450g	From SEAOC, 2024
F_a (per Table 1613.2.3(1))	1.0	For Simplified Design Procedure of Section 12.14 of ASCE 7, F_a shall be taken as 1.4 (Section 12.14.8.1)
F_v (per Table 1613.2.3(2))	1.850	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{MS} for Site Class D [Note: $S_{MS} = F_aS_S$]	1.258g	-
S_{M1} for Site Class D [Note: $S_{M1} = F_vS_1$]	0.833g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
S_{DS} for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	0.839g	-
S_{D1} for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.555g	Value is only applicable per requirements/exceptions per Section 11.4.8 of ASCE 7
C_{RS} (Mapped Risk Coefficient at 0.2 sec)	0.940	ASCE 7 Chapter 22
C_{R1} (Mapped Risk Coefficient at 1 sec)	0.932	ASCE 7 Chapter 22
*Since site soils are Site Class D and S1 is greater than or equal to 0.2, the seismic response coefficient Cs is determined by Eq. 12.8-2 for values of T ≤ 1.5Ts and taken equal to 1.5 times the value calculated in accordance with either Eq. 12.8-3 for TL ≥ T > Ts, or Eq. 12.8-4 for T > TL. Refer to ASCE 7-16. Site Class F modified to Site Class D, seismic parameters only applicable for structure period ≤ 0.5 second, refer to discussion above.		

A deaggregation of the PGA based on a 2,475-year average return period (MCE) indicates that an earthquake magnitude of 6.55 at a distance of approximately 14.75 km from the site would contribute the most to this ground motion. A deaggregation of the PGA based on a 475-year average return period (Design Earthquake) indicates that an earthquake magnitude of 6.56 at a distance of approximately 21.37 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 0.576g (SEAOC, 2024). The design PGA is equal to 0.384g (2/3 of PGA_M).

2.6 <u>Oversized Material</u>

Oversized materials (material larger than 8 inches in maximum dimension) are not anticipated to be encountered during site grading based on our subsurface evaluation. If encountered, recommendations are provided for appropriate handling of oversized materials in Appendix E.

2.7 <u>Expansion Potential</u>

Based on the results of laboratory testing from our evaluation and from nearby sites, finished grade soils are anticipated to have a "High to Very High" expansion potential. Final expansion potential of site soils should be determined at the completion of grading.

2.8 <u>Soils Susceptible to Hydro-Collapse</u>

Soils that are typically susceptible to hydro-collapse (or collapsible soils) are predominately sand and silt held in a loose honeycomb structure. This relatively loose honeycomb structure is held together by small amounts clay or calcium carbonate acting as a temporary (soluble) cementing agent. If the soil remains dry the soil maintains its structure, however the addition of water to the soil will greatly weaken the honeycomb structure and the soil can subsequently experience immediate collapses. This collapse results in rapid soil settlement and potential damage to any improvements which are located without the zone of influence of the collapsible soils.

Laboratory testing for hydro-collapse is typically performed per American Society for Testing and Materials (ASTM) Test Method D5333 or ASTM D 2435. The two most common categorizations of hydro-collapse potential are ASTM D5333 and the Naval Facilities Engineering Command (NFEC, 1986) are shown in Table 3A and 3B below. As indicated in Table 3B soils with collapse potential above 5 percent are considered "trouble".

TABLE 3A

Classification of Soil Collapsibility per ASTM D 5333

Collapse Index (Ie)	Collapse (%)	Collapse Potential
0	0	None
0.001 - 0.02	0.1 - 2	Slight
0.021 - 0.60	2.1 - 6	Moderate
0.60 - 0.10	6 - 10	Moderately Severe
> 0.10	> 10	Severe

TABLE 3B

Classification of Soil Collapsibility per NFEC, 1986

Collapse Potential (%)	Severity of Problem
0-1	No Problem
1-5	Moderate Trouble
5-10	Trouble
10-20	Severe Trouble
20	Very Severe Trouble

A summary of the onsite soils collapse potential are provided in Table 4 on the following page. The measured collapse potential is up to 7.5 percent, with three samples indicating collapse potential of 5 percent or greater.

TABLE 4

Summary of Hydro-Collapse Laboratory Test Results

Boring	**Laboratory Measured Collapse (%)**
HS-2 @ 10 feet	7.5
HS-2 @ 15 feet	1.7
HS-3 @ 10 feet	6.2
HS-5 @ 10 feet	5.1

3.0 <u>CONCLUSIONS</u>

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 development. A summary of our geotechnical conclusions are as follows:

- In general, borings excavated at the subject site indicate primarily native soils consisting of clayey sand, silty sand and sandy clay to the maximum explored depth of approximately 70 feet below existing grade. Native alluvial materials were found locally overlain by a relatively thin veneer of old artificial fill associated with previous development of the site. The near-surface loose and compressible soils are not suitable for the planned improvements in their present condition (refer to Section 4.1).
- Groundwater was encountered at a depth of approximately 54 feet below existing grade. Historic high groundwater is estimated to be greater than 40 feet below existing ground surface (CGS, 2001).
- The subject study area is not located within a mapped State of California Earthquake Fault 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 mapped liquefaction hazard zone. Based on our evaluation, site soils are generally not susceptible to liquefaction due to the fine-grained nature of the on-site soils and low probability of groundwater in the upper 50 feet. Therefore, liquefaction potential is considered low.
- Based on the results of preliminary laboratory testing, site soils are anticipated to have “High to Very High” expansion potential. Final design expansion potential must be determined at the completion of grading.
- Excavations into the existing site soils should be feasible with heavy construction equipment in good working order. We anticipate that soils 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.
- Oversized materials (greater than 8 inches in maximum dimension) are not likely to be encountered during site grading. If encountered, recommendations are provided for appropriate handling of oversized materials in Appendix E.
- Due to their high clay content and expansion potential, the onsite soils are not suitable for backfill of site retaining/basement walls. Therefore, import of sandy soils meeting project recommendations will be required.
- Field testing resulted in a measured infiltration rate ranging from 0.01 to 0.02 inches per hour. The measured infiltration rates do not include a factor of safety. Discussion regarding infiltration is provided in Section 4.8.
- Some of the site soils are susceptible to hydro-collapse. To mitigate this condition, the soils most susceptible to collapse should be temporarily removed and recompacted as fill during the grading operations.

- Pre-soaking of the subgrade for building slabs (and flatwork) will be required due to site expansive soils. The duration and process varies greatly based on the chosen method and is also dependent on factors such as soil type and weather conditions. Time duration for presoaking from completion of rough grading to trenching of foundations should be accounted for in the construction schedule (typically 2 to 3 weeks).

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

Rough grading shall include remedial earthwork grading and placement of engineered compacted fill to design grades. Geotechnical recommendations for precise grading and construction of the proposed new improvements will be provided, as necessary.

We recommend that earthwork onsite be performed in accordance with the following recommendations, future grading plan review report(s), the 2022 CBC/City of Irvine 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, areas to be developed should undergo complete demolition, removal of all vegetation, and clearing of pavements, existing utilities, foundation and slab elements from the site. Vegetation, debris from previous land use and excessive organic material should be removed and properly disposed of offsite. Holes resulting from removals of buried obstructions, which extend below proposed remedial and/or finish grades, should be replaced with suitable compacted fill material. If the demolition contractor removes subsurface utilities below the proposed remedial grading depth we recommend the excavations either be left open until grading operations begin or properly compacted to the depth of remedial grading.

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.

4.1.2 Removal Depths and Limits

In order to provide a relatively uniform bearing condition for the planned improvements, we recommend the near-surface potentially compressible soils and soils highly susceptible to collapse be temporarily removed and recompacted as engineered fill. If any undocumented artificial fill is encountered within the influence of the proposed building pads, the soils should be removed to competent native materials.

In order to promote more uniform soil conditions, soils shall be temporarily removed and recompacted to a minimum depth of approximately 12 feet below existing grade or 3 feet below the bottom of proposed foundations, whichever is deeper. Additionally, existing undocumented fill and unsuitable topsoil encountered within the building footprints should be temporarily removed and recompacted as compacted fill. Where space is available, the envelope for removal and recompaction should extend laterally a minimum distance equal to the depth of removal and recompaction below finish grade or 5 feet beyond the edges of the proposed building improvements, whichever is larger.

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). Isolated lenses of 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.

Where proposed improvements will be adjacent to property lines, the potential for impacting existing offsite improvements may be reduced by performing "ABC" slot cuts while performing earthwork removal and recompaction. "ABC" slot cuts are defined as excavations perpendicular to sensitive property boundaries that are divided into multiple "slots" of equal width. If slots are labeled A, B, C, A, B, C, etc., then all "A" slots can be excavated at the same time but must be backfilled before all "B" slots can be excavated, etc. Any given slot should be backfilled immediately with properly compacted fill to finish grade prior to excavation of the adjacent two slots. Please note some isolated sand lenses which are potentially susceptible to caving are present at the site. Recommendations for slot cut dimensions should be evaluated during grading. Protection of the existing offsite improvements during grading is the responsibility of the contractor.

Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a minimum distance equivalent to a 1:1(horizontal to vertical) 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 will be provided.

4.1.4 Removal Bottoms and Subgrade Preparation

In general, removal bottoms, over-excavation bottoms and 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 recommendations.

Removal bottoms, over-excavation 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 significant 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/basement wall backfill) should consist of clean, granular soils of Medium expansion potential (expansion index 90 or less based on ASTM D4829). Import for retaining/basement 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/basement wall backfill should consist of imported sandy soils with a maximum of 35 percent fines (passing the No. 200 sieve) per 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.

Aggregate base should conform to the requirements of Section 200-2 of the most recent version of the Standard Specifications for Public Works Construction ("Greenbook") for untreated base materials and/or City of Irvine requirements.

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 2 to 4 inches in maximum dimension) and well blended into fill soils with essentially not resulting voids. Demolition material placed in fills must be free of construction debris (wood, organics, etc.) and reinforcing steel. If you elect to incorporate asphalt concrete fragments into the fill materials, approval from an environmental viewpoint and/or local agency may be required and is not the purview of the geotechnical consultant. From our previous experience, we recommend that asphalt concrete fragments be exported or limited to fill areas within planned street or parking 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 will be required prior to reusing the materials in compacted fills. Soils are also present that will require additional moisture in order to achieve the required compaction.

The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 8 inches in 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.

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).

4.1.7 Trench and Retaining/Basement Wall Backfill and Compaction

The onsite materials may generally be suitable as trench backfill provided the soils are screened of organic matter and rocks greater than 6 inches in diameter. Trench backfill should be compacted in uniform lifts (generally not exceeding 12 inches in compacted thickness) by mechanical means to at least 90 percent relative compaction (per ASTM Test Method D1557). A representative from LGC Geotechnical should observe and test the backfill to verify compliance with the project recommendations.

Some manufacturers have restrictions for the type of soil in contact with pipes. Manufacturer specifications for soils surrounding the pipe should be reviewed and approved by the pipe designer for use prior to construction. The use of open-graded rock as backfill must include means of reducing migration of fines of adjacent materials into the void spaces. This can be done by entirely wrapping the open-graded rock in a filter fabric or introducing an excavatable flowable fill to the rock.

Compactive efforts must be made for all backfill soil types except for flowable fill. There must be room between the trench side walls and the springline of the pipe for mechanical compaction equipment to get adequate compaction of the soil under the haunches of the pipe. If mechanical compaction may result in damage to pipes, clean sand having a Sand Equivalent (SE) greater than or equal to 20 (per California Test Method [CTM] 217) can be used to shade the pipes. Sand backfill should be densified by jetting or flooding and then tamped to ensure adequate compaction. Sand grains should be from a natural source with rounded shape. Manufactured sand from crushed rock or recycled material is not suitable for jetting/flooding as the grains are typically angular in shape and do not densify well enough with these methods.

Retaining/basement wall backfill should consist of approved imported sandy soils as outlined in preceding Section 4.1.5. The onsite soils are not suitable for backfill of site retaining/basement walls based on their fines content and expansion potential. Therefore, import of sandy soils meeting project recommendations will be required. For the lateral limits of select sandy backfill refer to Figures 3 and 4. Retaining/basement 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/basement wall backfill materials should not be permitted.

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

4.1.8 Shrinkage and Subsidence

Allowance in the earthwork volumes budget should be made for an estimated 5 to 15 percent reduction in volume of near-surface (upper approximate 12 feet) soils. It should be stressed that these values are only estimates and that an actual shrinkage factor would be extremely difficult to predetermine. The effective change in volume of onsite soils will depend primarily on the type of compaction equipment, method of compaction used onsite by the contractor, and accuracy of the topographic survey.

Due to the combined variability in topographic surveys, inability to precisely model the removals and variability of on-site near-surface conditions, it is our opinion that the site will not balance at the end of grading. If importing/exporting a large volume of soils is not considered feasible or economical, we recommend a balance area be designated onsite that can fluctuate up or down based on the actual volume of soil. We recommend a "balance" area that can accommodate on the order of 5 percent (plus or minus) of the total grading volume be considered.

4.2 Preliminary Foundation Recommendations

Provided that the remedial grading recommendations provided herein are implemented, the site may be considered suitable for the support of the proposed structures using either a post-tensioned foundation or alternative rigid foundation system designed to resist the impacts of expansive soils. Site soils are anticipated to be of High to Very High expansion potential (EI of greater than 90 per ASTM D4829). This must be verified based on as-graded conditions. Please note that the following foundation recommendations are preliminary and must be confirmed by LGC Geotechnical at the completion of project plans (i.e., foundation, grading and site layout plans) as well as completion of earthwork. Recommended soil bearing and estimated static settlement are provided in Section 4.3.

4.2.1 Provisional Post-Tensioned Foundation Design Parameters

The geotechnical parameters provided herein may be used for post-tensioned slab foundations with a deepened perimeter footing or a post-tensioned mat slab. These parameters have been determined in general accordance with the Post-Tensioning Institute (PTI) Standard Requirements for Design of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, referenced in Chapter 18 of the 2022 CBC. In utilizing these parameters, the foundation engineer should design the foundation system in accordance with the allowable deflection criteria of applicable codes and the requirements of the structural designer/architect. Other types of stiff slabs may be used in place of the CBC post-tensioned slab design provided that, in the opinion of the foundation structural designer, the alternative type of slab is at least as stiff and strong as that designed by the CBC/PTI method.

Our design parameters are based on our experience with similar projects, test results onsite, and the anticipated nature of the soil (with respect to expansion potential).

Please note that implementation of our recommendations will not eliminate foundation movement (and related distress) should the moisture content of the subgrade soils fluctuate. It is the intent of these recommendations to help maintain the integrity of the proposed structures and reduce (not eliminate) movement, based upon the anticipated site soil conditions. Should future owners and/or property maintenance personnel not properly maintain the areas surrounding the foundation, for example by overwatering, then we anticipate for highly expansive soils the maximum differential movement of the perimeter of the foundation to the center of the foundation to be on the order of a couple of inches. Soils of lower expansion potential are anticipated to show less movement.

TABLE 5

Preliminary Post-Tensioned Foundation Design Parameters

Parameter	PT Slab with Perimeter Footing	PT Mat with Thickened Edge
Expansion Potential	Very High	Very High
Thornthwaite Moisture Index	-20	-20
Constant Soil Suction	PF 3.9	PF 3.9
Center Lift Edge moisture variation distance, e_m Center lift, y_m	 6.1 feet 1.0 inch	 6.1 feet 1.2 inch
Edge Lift Edge moisture variation distance, e_m Edge lift, y_m	 3.4 feet 2.5 inches	 3.4 feet 2.5 inches
Modulus of Subgrade Reaction, k (assuming presoaking as indicated below)	100 pci	100 pci
Minimum perimeter footing/thickened edge embedment below finish grade	30 inches	6 inches
Minimum Slab Thickness	6 inches[2]	10 inches[2]
Presoak	140% of Opt. 30 inches	140% of Opt. 30 inches

1. Expansion index is for preliminary design purposes. Further evaluation is needed at the completion of grading.
2. Recommendations for foundation reinforcement and slab thickness are ultimately the purview of the foundation engineer/structural engineer based upon geotechnical criteria and structural engineering considerations.
3. Recommendations for vapor retarders below slabs are also the purview of the foundation engineer/structural engineer and should be provided in accordance with applicable code requirements.

4.2.2 Provisional Conventional Foundation Design Parameters

Conventional foundations may be designed in accordance with the Wire Reinforcement Institute (WRI) procedure for slab-on-ground foundations per Section 1808 of the 2022 CBC to resist expansive soils. The following preliminary soil parameters may be used:

- Effective Plasticity Index: 45
- Climatic Rating: Cw = 15
- Reinforcement: Per structural designer
- Minimum Footing Depth: 30 inches below lowest adjacent grade.
- Moisture-condition (presoak) slab subgrade to 140% of optimum moisture content to a minimum depth of 30 inches prior to trenching.

The recommended moisture content should be maintained up to the time of concrete placement.

4.2.3 Shallow Foundation Maintenance

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

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

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

4.2.4 Slab Underlayment Guidelines

The following is for informational purposes only since slab underlayment (e.g., moisture retarder, sand or gravel layers for concrete curing and/or capillary break) is unrelated to the geotechnical performance of the foundation and thereby not the purview of the geotechnical consultant. Post-construction moisture migration should be expected below the foundation. The foundation engineer/architect should determine whether the use of a capillary break (sand or gravel layer), in conjunction with the vapor retarder, is necessary or required by code. Sand layer thickness and location (above and/or below vapor retarder) should also be determined by the foundation engineer/architect.

4.3 Soil Bearing and Lateral Resistance

Subterranean Basement at least 9 feet below Grade: Provided our earthwork recommendations are implemented, an allowable soil bearing pressure of 3,000 pounds per square foot (psf) may be used for the design of footings having a minimum width of 18 inches and minimum embedment of 18 inches below the lowest adjacent ground surface. This value may be increased by 300 psf for each additional foot of embedment and 150 psf for each additional foot of foundation width to a maximum of 4,000 psf.

At-Grade Structures: Provided our earthwork recommendations are implemented, an allowable soil bearing pressure of 1,500 pounds per square foot (psf) may be used for the design of footings having a minimum width of 12 inches and minimum embedment of 12 inches below lowest adjacent ground surface. This value may be increased by 300 psf for each additional foot of embedment and 150 psf for each additional foot of foundation width to a maximum value of 2,500 psf.

A mat foundation a minimum of 6 inches below lowest adjacent grade may be designed for an allowable soil bearing pressure of 1,200 psf. These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Bearing values indicated 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).

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 1-inch or less. Differential settlement may be taken as half of the total settlement (i.e., ½-inch over a horizontal span of 40 feet).

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.25 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 200 psf per foot of depth (or pcf) to a maximum of 2,000 psf may be used for the sides of footings poured against properly compacted fill. Allowable passive pressure may be increased to 265 pcf (maximum of 2,650 psf) for short duration seismic loading. This passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Frictional resistance and passive pressure may be used in combination without reduction. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt. 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. The structural designer should incorporate appropriate factors of safety and/or load factors in their design.

4.4 Lateral Earth Pressures and Retaining/Basement Wall Design Considerations

The following may be used for design of site retaining/basement walls. It is our understanding that basement walls up to approximately 9 feet are proposed beneath the church building. Lateral earth pressures are provided as equivalent fluid unit weights, in psf per foot of depth (or pcf). These values do not contain an appreciable factor of safety, so the retaining/basement 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 in Table 6 below, for approved imported free draining, clean granular (sandy) soils with a maximum of 35 percent fines (passing the No. 200 sieve per ASTM D-421/422) and a "Very Low" expansion potential (EI of 20 or less per ASTM D4829). The site soils are not suitable for retaining/basement wall backfill due to their fines content and expansion index. Import of soils meeting the criteria outlined above will need to be used for retaining/basement wall backfill soil. <u>The wall designer should clearly indicate on the retaining/basement wall plans the required imported sandy soil backfill criteria.</u> These preliminary findings should be confirmed during grading.

TABLE 6

Lateral Earth Pressures – Approved Imported Sandy Soils

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. This would include basement walls and 90-degree corners of retaining walls. Such walls should be designed for "at-rest." The equivalent fluid pressure values assume free-draining conditions and a drainage system will be installed and maintained to prevent the build-up of hydrostatic pressures. If conditions other than those assumed above are anticipated, the equivalent fluid pressure values should be provided on an individual-case basis by the geotechnical engineer.

Retaining/basement wall structures should be provided with appropriate drainage and appropriately waterproofed. To reduce, but not eliminate, subsurface water issues for soils in front of the retaining or potential water intrusion into the basement, the perforated subdrain pipe should be located as low as possible behind the retaining/basement wall. The outlet pipe should be sloped to drain to a suitable outlet. In general, we do not recommend retaining/basement wall outlet pipes be connected to area drains. If subdrains are connected to area drains, special care should be taken to maintain these drains. Typical conventional retaining/basement wall drainage is shown on Figures 3 and 4, respectively. It should be noted that the recommended subdrain does not provide protection against seepage through the face of the wall and/or efflorescence. Waterproofing and subdrain outlet systems are not the purview of the geotechnical consultant.

Surcharge loading effects from any adjacent structures should be evaluated by the retaining/basement wall designer. In general, structural loads within a 1:1 (horizontal to vertical) upward projection from the bottom of the proposed retaining/basement wall footing will surcharge the proposed retaining/basement wall. In addition to the recommended earth pressure, retaining/basement walls adjacent to streets should be designed to resist a uniform lateral pressure of 85 pounds per square foot (psf) due to normal street vehicle traffic, if applicable. Uniform lateral surcharges may be estimated using the applicable coefficient of lateral earth pressure using a rectangular distribution. A factor of 0.45 and 0.3 may be used for at-rest and active conditions, respectively. The retaining/basement wall designer should contact the geotechnical consultant for any required geotechnical input in estimating surcharge loads.

The retaining wall/basement wall designer may use a seismic lateral earth pressure increment of 10 pcf. This increment should be applied in addition to the provide static lateral earth pressure using a triangular distribution with the resultant acting at H/3 in relation to the base of the retaining structure (where H is the retained height). 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 and the Naval Civil Engineering Laboratory (Lew, et al, 2010 & NCEL, 1993).

Soil bearing and lateral resistance (friction coefficient and passive resistance) are provided in Section 4.3. Earthwork considerations (temporary backcuts, backfill, compaction, etc.) for retaining/basement walls are provided in Section 4.1 (Site Earthwork) and the subsequent earthwork related sub-sections.

4.5 <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 a soluble sulfate content value of 95 ppm (less than 0.01 percent), a chloride content of 60 ppm, pH of 7.90, and a minimum resistivity of 720 ohm-centimeters. Based on Caltrans Corrosion Guidelines (2021), soils are considered corrosive if the pH is 5.5 or less, or the chloride concentration is 500 ppm or greater, or the sulfate concentration is 2,000 ppm (0.2 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 severely 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 our laboratory test results of representative site soil samples and our experience, onsite soils should be considered as having a severity categorization of “moderate” and are designated class “S1” 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 “S1” 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.

4.6 <u>Preliminary Asphalt Concrete Pavement Sections</u>

For the purposes of these preliminary recommendations, we have selected a preliminary design R-value of 5 and calculated pavement sections for Traffic Indices of 5.0, 5.5 and 6.0. R-value testing of the street subgrade will need to be performed to confirm our preliminary testing results/assumptions once the streets have been graded to finish subgrade elevations and the final Traffic Index is determined by the Civil Engineer or City Engineer. We are not responsible for selecting a design Traffic Index. It is our understanding that the City of Irvine requires a minimum pavement section of 4.2 inches of asphalt over 6 inches of base.

<u>TABLE 7</u>

<u>Preliminary Pavement Sections</u>

Assumed Traffic Index	5.0	5.5	6.0
R -Value Subgrade	5	5	5
AC Thickness	4.2	4.2 inches	4.2 inches
AB Thickness	8.0 inches	10.0 inches	12.5 inches

Due to anticipated construction traffic prior to the completion of the project, we recommend that the total thickness (base course and capping course) of AC be placed at essentially the same time. Construction traffic loading on only the base course of the AC will increase the potential for pavement distress. It should be noted that construction traffic such as concrete trucks will likely exceed traffic loading after completion of construction.

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

4.7 Nonstructural Concrete Flatwork

Nonstructural concrete flatwork (such as walkways, private drives, patio slabs, 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 may be designed in accordance with the minimum guidelines outlined in Table 8 on the following page. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

TABLE 8

Nonstructural Concrete Flatwork Guidelines

	Sidewalks (≤6 feet wide)	Patios/Entryways / Walkways (adjacent to Buildings or flatwork >6 feet wide)	City Sidewalk Curb and Gutters
Minimum Thickness (in.)	4 (nominal)	5 (full)	City/Agency Standard
Presaturation	140% of optimum m/c to 12 inches	140% of optimum m/c to 24 inches	City/Agency Standard
Reinforcement	—	No. 3 at 24 inches on centers	City/Agency Standard
Thickened Edge (in.)	—	—	City/Agency Standard
Crack Control Joints	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	City/Agency Standard
Maximum Joint Spacing	5 feet	6 feet	City/Agency Standard

4.8 Subsurface Water Infiltration

Recent regulatory changes have occurred that mandate that storm water be infiltrated below grade into subsurface soils rather than collected in a conventional storm drain system. Typically, a combination of methods are implemented to reduce surface water runoff and increase infiltration including; permeable pavements/pavers for roadways and walkways, directing surface water runoff to grass-lined swales, retention areas, and/or drywells, etc.

It should be noted that collecting and concentrating surface water for the purpose of intentionally infiltrating below grade, conflicts with the geotechnical engineering objective of directing surface water away from slopes, structures and other improvements. The geotechnical stability and integrity of a site is reliant upon appropriately handling surface water. In general, the vast majority of geotechnical distress issues are directly related to improper drainage. In general, distress in the form of movement of improvements could occur as a result of soil saturation and loss of soil support, expansion, internal soil erosion, collapse and/or settlement.

The results of our field infiltration testing indicate the observed 1-D infiltration rate for I-1 and I-2 (not including required factors of safety for design) was 0.01 to 0.02 inches per hour. The design infiltration rate is thereby equal to the Observed Infiltration Rate provided in Table 1 (inches per hour) divided by the design factor of safety. The design factor of safety must be a minimum of 2.0 but may be increased at the discretion of the design engineer (County of Orange, 2013). Per the County of Orange infiltration guidelines (2013), infiltration of stormwater is not required when the factored infiltration rate (measured infiltration rate with safety factors applied) is less than 0.3 inches per hour in the vicinity of the BMPs.

Based on the results of field infiltration testing indicating essentially zero infiltration and soils with high fines content (silts and clays), we strongly recommend against the intentional infiltration of stormwater into the subsurface soils.

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. Where possible, we recommend concrete or asphalt be placed around the outer 5-10 feet of the buildings. 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/basement 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/basement wall backfill and compaction;
- During utility trench backfill and compaction;
- After presoaking building pad and other concrete-flatwork subgrades, and prior to placement of aggregate base or concrete;
- Preparation of pavement subgrade and placement of aggregate base;
- After building and wall footing excavation and prior to placement of steel reinforcement and/or concrete; and
- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

5.0 <u>LIMITATIONS</u>

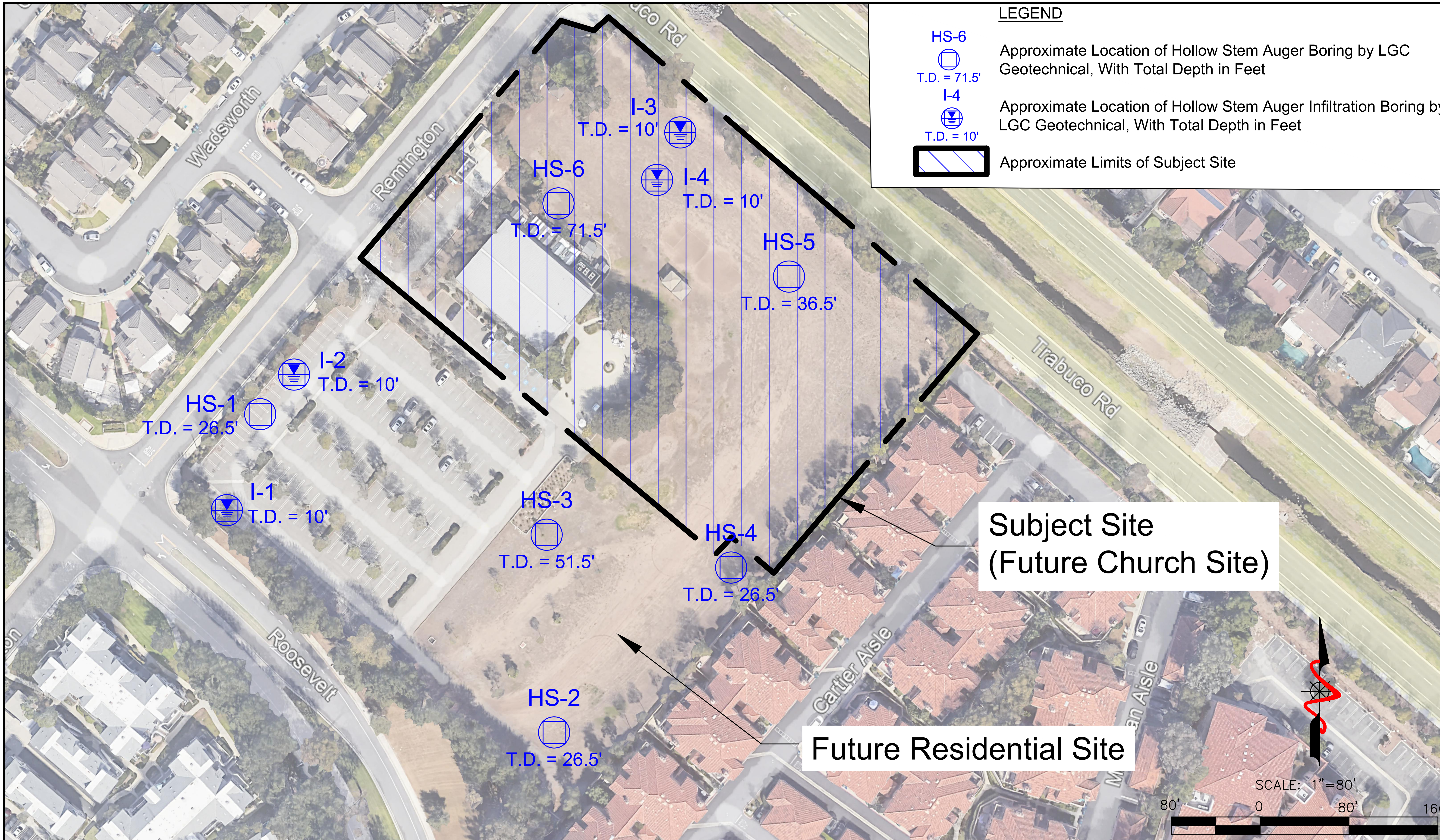
Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable soils engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included 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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131 Calle Iglesia, Ste. 200
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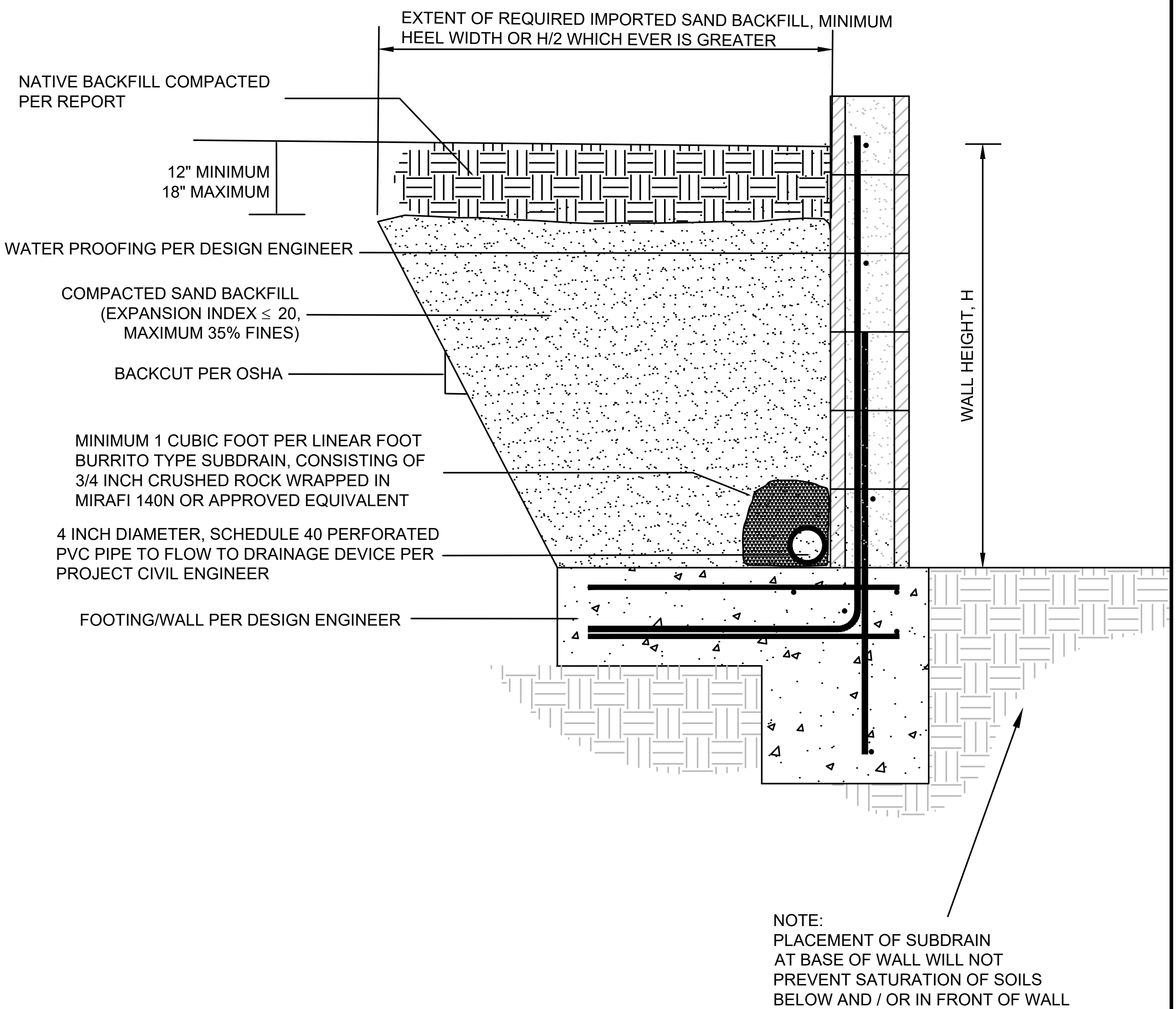
FIGURE 2
Boring Location Map

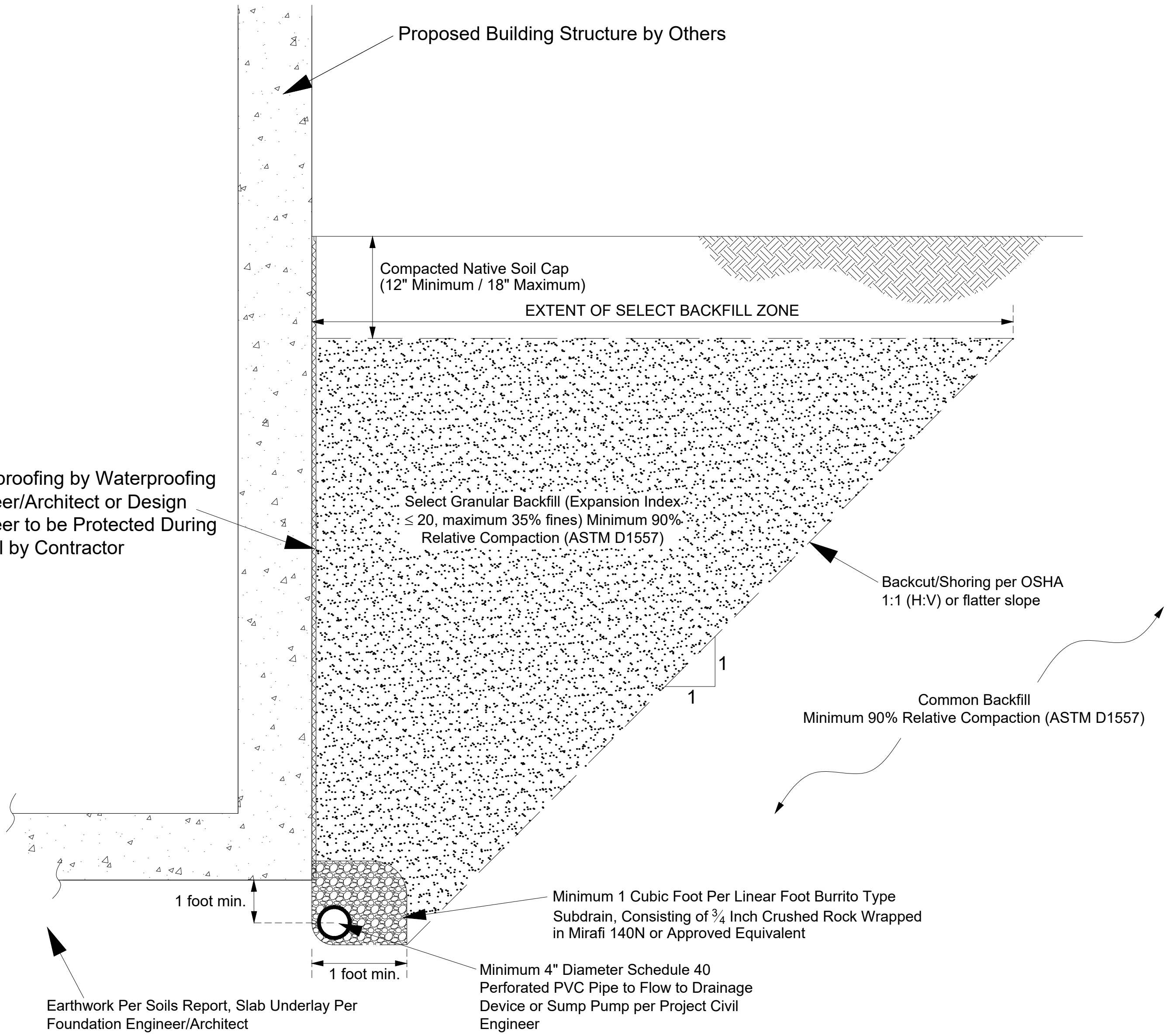
PROJECT NAME	Our Lady of Peace, Irvine
PROJECT NO.	23174-01
ENG. / GEOL.	DJB / BPG
SCALE	1" = 80'
DATE	July 2024



LGC Geotechnical, Inc.

FIGURE 3
Retaining Wall Backfill Detail

PROJECT NAME	Our Lady of Peace, Irvine
PROJECT NO.	23174-01
ENG. / GEOL.	DJB
SCALE	Not to Scale
DATE	July 2024



NOTE : Granular backfill may be substituted with 3/4 inch crushed rock wrapped in filter fabric. Crushed rock must be compacted in lifts. Geotechnical drainage detail will not prevent water seepage into structure, it is only to prevent build up of water pressure against wall.

LGC Geotechnical, Inc.

FIGURE 4

Basement Subterranean Structure Backfill/Subdrain Detail

PROJECT NAME	Our Lady of Peace, Irvine
PROJECT NO.	23174-01
ENG. / GEOL.	DJB / BPG
SCALE	Not to Scale
DATE	July 2024

Appendix A
References

APPENDIX A

<u>References</u>

American Concrete Institute, 2019, Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary (ACI 318R-19).

American Society of Civil Engineers (ASCE), 2017, Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-16, Third Printing, 2017.

ASTM International, Annual Book of ASTM Standards, Volume 04.08.

California Building Standards Commission, 2022, California Building Code, California Code of Regulations Title 24, Volumes 1 and 2, dated July 2022.

California Geological Survey (CGS), (Previously California Division of Mines and Geology [CDMG]), 2001, Earthquake Zones of Required Investigation, Tustin Quadrangle, Seismic Hazard Zones, Released: January 17, 2001.

________, 2001, State of California Seismic Hazard Zone Report for the Tustin 7.5-Minute Quadrangle, Orange County, California, Open-File Report 97-20. Revised 2001.

California Department of Conservation, Division of Mines and Geology, 1997, Guidelines for Evaluating and Mitigating Seismic Hazards in California, CDMG Special Publication 117.

________, 2000, Digital Images of Official Maps of Alquist-Priolo Earthquake Fault Zones of California, Southern Region, CDMG CD 2000-03.

California Department of Transportation (Caltrans), 2021, Corrosion Guidelines, Version 3.2, dated May 2021.

________, 2017, Highway Design Manual, Updated November 20, 2017.

California Geological Survey (CGS), 2008, California Geological Society Special Publication 117A: Guidelines for Evaluating and Mitigating Seismic Hazards in California.

________, 2018, Special Publication 42: Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California, Revised 2018.

County of Orange, 2013, Technical Guidance Document (TGD) for the Preparation of Conceptual and/Preliminary and/or Project Water Quality Management Plans (WQMPs).

Harrison Design, 2024, Proposed Site Plan, Our Lady of Peace Korean Catholic Center, dated April 11, 2024.

LGC Geotechnical, Inc., 2024, Preliminary Geotechnical Subsurface Evaluation, Proposed Residential Development, 14010 Remington, Irvine, California, Project No. 23174-01, dated May 28, 2024.

Morton, D.M., F.K. Miller , 2006, Geologic Map of the San Bernardino and Santa Ana 30' x 60' quadrangles, California: U.S. Geological Survey Open-File Report 2006-1217. 42 / 2260331.

Naval Civil Engineering Laboratory (NCEL), 1993, The Seismic Desing of Waterfront Retaining Structures, Technical Report ITL-92-11, NCEL TR-939, January 1993.

Structural Engineers Association of California (SEAOC), 2024, U.S. Seismic Design Maps, Retrieved June 26, 2024, from https://seismicmaps.org/.

United States Geological Survey (USGS), 2014, Unified Hazard Tool, Dynamic: Conterminous U.S. 2014 (v4.2.0), Retrieved June 26, 2024, from https://earthquake.usgs.gov/hazards/interactive/.

Appendix B
Boring Logs

Geotechnical Boring Log Borehole HS-5

Date: 2/23/24	**Drilling Company:** 2R Drilling
Project Name: Our Lady of Peace, Irvine	**Type of Rig:** Limited Access Rig
Project Number: 23174-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: 161' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 2

Logged By RNP
Sampled By RNP
Checked By BPP

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
160	0							@0' - Grass	
								Young Axial Channel Deposits (Qya):	
			R-1	4 13 31	100.4	20.9	CL	@ 2.5' - Sandy CLAY: dark brown, very moist, hard, traces of roots	
155	5	B-1	R-2	11 11 11	79.5	9.3	SM	@ 5' - Silty SAND: pale brown, moist, medium dense	
			R-3	13 21 24	118.6	9.8	SC	@ 7.5' - Clayey SAND: brown, moist, dense	
150	10		R-4	13 12 10	81.6	9.8	CL-ML	@ 10' - Silty CLAY: olive brown, slightly moist, very stiff	CO
145	15		SPT-1	4 4 4		7.9	SM	@ 15' - Silty SAND: brown, moist, medium dense	
140	20		R-5	8 17 18	120.1	11.8	SC	@ 20' - Clayey SAND: dark brown, moist, medium dense	
135	25		SPT-2	3 5 7		28.0	CL	@ 25' - Sandy CLAY: dark brown, very moist, very stiff	
	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: 2/23/24	**Drilling Company:** 2R Drilling
Project Name: Our Lady of Peace, Irvine	**Type of Rig:** Limited Access Rig
Project Number: 23174-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~161' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 2

Logged By RNP
Sampled By RNP
Checked By BPP

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
130	30		R-6	8 13 18	108.8	5.4	SC	@ 30' - Clayey SAND: brown, slightly moist, medium dense	
125	35		SPT-3	4 6 7		12.1		@ 35' - Clayey SAND: brown, moist, medium dense	
120	40							Total Depth = 36.5' No Groundwater Encountered Backfilled with Cuttings on 2/23/24	
115	45								
110	50								
105	55								
	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-6

Date: 2/23/24	**Drilling Company:** 2R Drilling
Project Name: Our Lady of Peace, Irvine	**Type of Rig:** Limited Access Rig
Project Number: 23174-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: 158' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 3

Logged By RNP
Sampled By RNP
Checked By BPP

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@0' - Grass	MD EI
		B-1						**Young Axial Channel Deposits (Qya):**	
155			R-1	7 10 11	97.5	24.2	CL	@ 2.5' - Sandy CLAY: brown, very moist, very stiff, scattered rootlets	
	5		R-2	6 11 18	109.1	19.1		@ 5' - Sandy CLAY: brown, very moist, very stiff	
150			R-3	5 12 17	89.3	18.6	SC	@ 7.5' - Clayey SAND: brown, very moist, medium dense	
	10		R-4	12 14 16	112.9	15.7	CL	@ 10' - Sandy CLAY: brown, moist, very stiff	
145									
	15		SPT-1	3 5 6		12.6	SC	@ 15' - Clayey SAND: reddish brown, moist, medium dense	
140									
	20		R-5	11 16 26	108.9	3.9	SM-CL	@ 20' - Silty SAND to Sandy CLAY: brown, slightly moist, dense/hard, traces of gravel	
135									
	25		SPT-2	4 4 6		13.4	SC	@ 25' - Clayey SAND: brown, moist, medium dense	
130									
	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 HS-6

Date: 2/23/24	**Drilling Company:** 2R Drilling
Project Name: Our Lady of Peace, Irvine	**Type of Rig:** Limited Access Rig
Project Number: 23174-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~158' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 2 of 3

Logged By RNP
Sampled By RNP
Checked By BPP

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	30		R-6	8 8 10	88.3	19.7	ML	@ 30' - Sandy SILT: pale brown, very moist, medium dense	
125									
	35		SPT-3	4 5 6		9.6	SC	@ 35' - Clayey SAND: brown, moist, medium dense	
120									
	40		R-7	7 12 20	109.4	13.3		@ 40' - Clayey SAND: brown, moist, medium dense	
115									
	45		SPT-4	5 7 6		12.9		@ 45' - Clayey SAND: brown, moist, medium dense	
110									
	50		R-8	6 7 12	89.9	30.7	CL	@ 50' - Sandy CLAY: brown, very moist, very stiff	
105		▽							
	55		SPT-5	2 2 5		22.0		@ 55' - Sandy CLAY: brown, wet, stiff	
100									
	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-6

Date: 2/23/24	**Drilling Company:** 2R Drilling
Project Name: Our Lady of Peace, Irvine	**Type of Rig:** Limited Access Rig
Project Number: 23174-01	**Drop:** 30" **Hole Diameter:** 6"
Elevation of Top of Hole: ~158' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 3 of 3

Logged By RNP
Sampled By RNP
Checked By BPP

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	60		R-9	8 13 15	92.7	31.3	CL	@ 60' - Sandy CLAY: brown, wet, very stiff	
95									
	65		SPT-6	2 3 6		21.8		@ 65' - Sandy CLAY: brown, wet, stiff	
90									
	70		R-10	9 13 22	102.2	24.0		@ 70' - Sandy CLAY: brown, wet, very stiff	
85								Total Depth = 71.5' Groundwater encountered at 54' Backfilled with Cuttings on 2/23/24	
	75								
80									
	80								
75									
	85								
70									
	90								

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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 I-3

Date: 2/23/24	**Drilling Company:** 2R Drilling
Project Name: Our Lady of Peace, Irvine	**Type of Rig:** Limited Access Rig
Project Number: 23174-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~160' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By BPP

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@0' - Grass	
								Young Axial Channel Deposits (Qya):	
			R-1	6 19 19	87.2	14.4	SC	@ 2.5' - Clayey SAND: brown, very moist, dense, traces of roots	
155	5		R-2	11 18 23	87.5	13.8		@ 5' - Clayey SAND: pale brown, very moist, dense	
			R-3	9 12 17	88.1	11.7	CL	@ 8.5' - Sandy CLAY: brown, moist, very stiff	-#200
150	10							Total Depth = 10' No Groundwater Encountered 3" Perforated Pipe with Filter Sock and Gravel Installed Pipe Removed and Backfilled with Cuttings on 2/26/2024	
145	15								
140	20								
135	25								
130	30								

THIS SUMMARY APPLIES ONLY AT THE LOCATION OF THIS BORING AND AT THE TIME OF DRILLING. SUBSURFACE CONDITIONS MAY DIFFER AT OTHER LOCATIONS AND MAY CHANGE AT THIS LOCATION WITH THE PASSAGE OF TIME. THE DATA PRESENTED IS A SIMPLIFICATION OF THE ACTUAL CONDITIONS ENCOUNTERED. THE DESCRIPTIONS PROVIDED ARE QUALITATIVE FIELD DESCRIPTIONS AND ARE NOT BASED ON QUANTITATIVE ENGINEERING ANALYSIS.

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

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

Geotechnical Boring Log Borehole I-4

Date: 2/23/24	**Drilling Company:** 2R Drilling
Project Name: Our Lady of Peace, Irvine	**Type of Rig:** Limited Access Rig
Project Number: 23174-01	**Drop:** 30" **Hole Diameter:** 8"
Elevation of Top of Hole: ~159' MSL	**Drive Weight:** 140 pounds
Hole Location: See Geotechnical Map	Page 1 of 1

Logged By RNP
Sampled By RNP
Checked By BPP

Elevation (ft)	Depth (ft)	Graphic Log	Sample Number	Blow Count	Dry Density (pcf)	Moisture (%)	USCS Symbol	DESCRIPTION	Type of Test
	0							@0' - Grass	
								<u>Young Axial Channel Deposits (Qya):</u>	
			R-1	6 9 13	103.9	15.0	CL	@ 2.5' - Sandy CLAY: dark brown, moist, very stiff, traces of roots.	
155	5		R-2	12 14 14	74.6	14.6	SC	@ 5' - Clayey SAND: pale brown, very moist, medium dense.	
150			R-3	19 32 41	118.6	12.4	CL	@ 8.5' - Sandy CLAY: dark brown, moist, hard.	
	10							Total Depth = 10' No Groundwater Encountered 3" Perforated Pipe with Filter Sock and Gravel Installed Pipe Removed and Backfilled with Cuttings on 2/26/2024	
145	15								
140	20								
135	25								
130	30								

LGC Geotechnical, Inc.

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

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

Grain Size Distribution/Fines Content: Representative samples were dried, weighed and soaked in water until individual soil particles were separated (per ASTM D421) and then washed on a No. 200 sieve (ASTM D1140). Where applicable, the portion retained on the No. 200 sieve 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
I-1 @ 8.5 feet	Clayey Sand	48
I-3 @ 8.5 feet	Sandy Clay	68

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-3 @ 1-5 feet	112	High
HS-6 @ 1-5 feet	135	Very High

* ASTM D4829

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:

Laboratory Testing Procedures and Test Results

Sample Location	Sample Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
HS-3 @ 1-5 feet	Dark Grayish Brown Clay	117.5	12.5
HS-6 @ 1-5 feet	Dark Brown Sandy Clay	115.0	14.0

Collapse/Swell Potential: Six collapse tests were performed per ASTM D4546. A sample (2.4 inches in diameter and 1-inch in height) was placed in a consolidometer and loaded to the approximate in-situ effective stress. The curve is presented in this Appendix.

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-3 @ 1-5 feet	60

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-3 @ 1-5 feet	95	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-3 @ 1-5 feet	7.90	720

ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS

ASTM D 4546

Project Name:	Shea - Our Lady of Peace, Irvine	Tested By:	G. Bathala	Date:	05/20/24
Project No.:	23174-01	Checked By:	J. Ward	Date:	05/22/24
Boring No.:	HS-1	Sample Type:	Ring		
Sample No.:	R-4	Depth (ft.)	10.0		
Sample Description:	Brown lean clay (CL)				

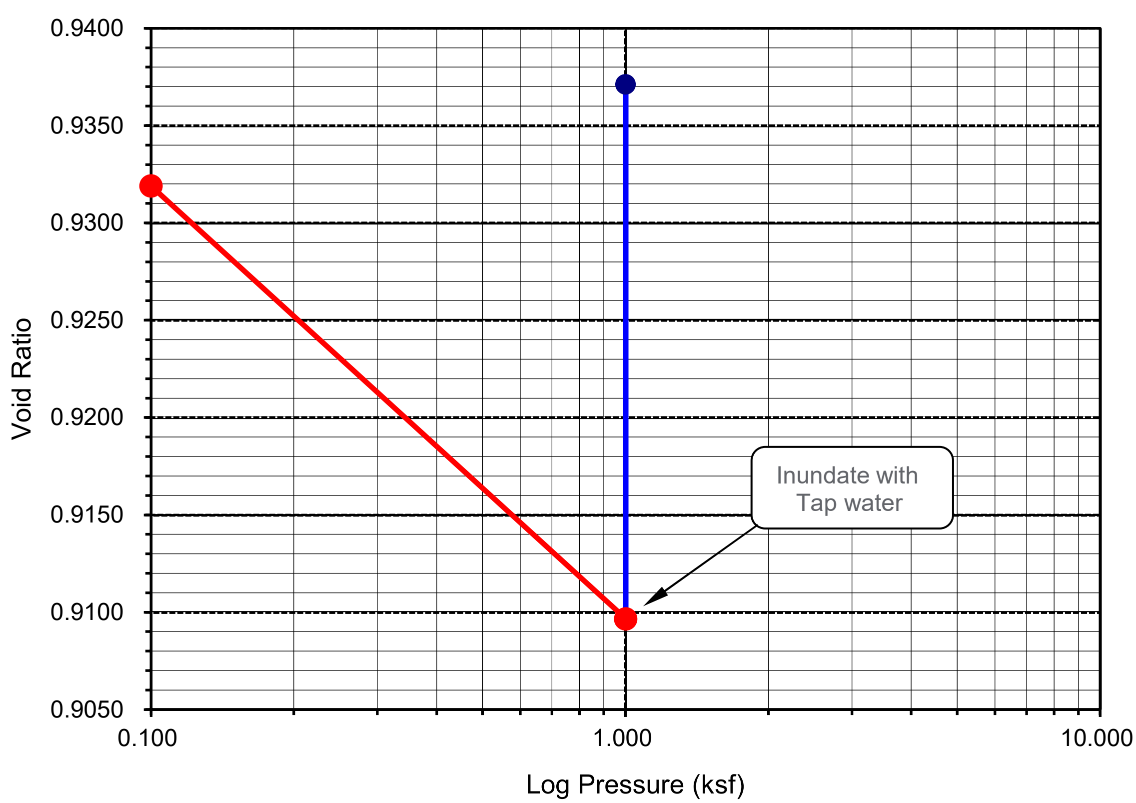
Initial Dry Density (pcf):	87.2
Initial Moisture (%):	22.94
Initial Length (in.):	1.0000
Initial Dial Reading:	0.1187
Diameter(in):	2.415

Final Dry Density (pcf):	87.3
Final Moisture (%) :	31.6
Initial Void ratio:	0.9335
Specific Gravity(assumed):	2.70
Initial Saturation (%)	66.4

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.1195	0.9992	0.00	-0.08	0.9319	-0.08
1.000	0.1325	0.9862	0.15	-1.38	0.9097	-1.23
H2O	0.1183	1.0004	0.15	0.04	0.9371	0.19

Percent Swell (+) / Settlement (-) After Inundation = 1.44

Void Ratio - Log Pressure Curve



ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS
ASTM D 4546

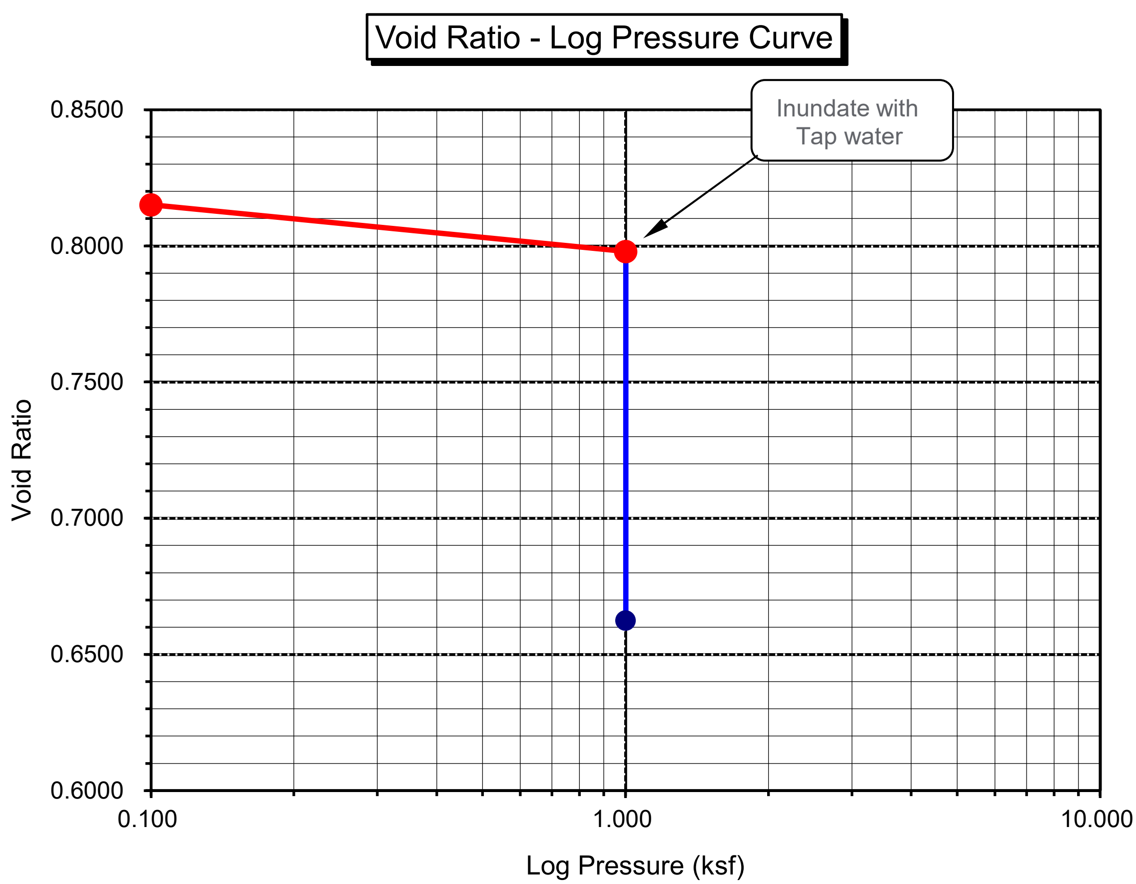
Project Name:	Shea - Our Lady of Peace, Irvine	Tested By:	G. Bathala	Date:	05/21/24
Project No.:	23174-01	Checked By:	J. Ward	Date:	05/23/24
Boring No.:	HS-2	Sample Type:	Ring		
Sample No.:	R-4	Depth (ft.)	10.0		
Sample Description:	Brown silty clay (CL-ML)				

Initial Dry Density (pcf):	92.3
Initial Moisture (%):	7.54
Initial Length (in.):	1.0000
Initial Dial Reading:	0.1122
Diameter(in):	2.415

Final Dry Density (pcf):	101.7
Final Moisture (%) :	20.8
Initial Void ratio:	0.8270
Specific Gravity(assumed):	2.70
Initial Saturation (%)	24.6

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.1187	0.9935	0.00	-0.65	0.8151	-0.65
1.000	0.1296	0.9826	0.15	-1.74	0.7980	-1.59
H2O	0.2038	0.9084	0.15	-9.16	0.6624	-9.01

Percent Swell (+) / Settlement (-) After Inundation = -7.54



ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS

ASTM D 4546

Project Name:	Shea - Our Lady of Peace, Irvine	Tested By:	G. Bathala	Date:	03/13/24
Project No.:	23174-01	Checked By:	J. Ward	Date:	03/21/24
Boring No.:	HS-2	Sample Type:	Ring		
Sample No.:	R-5	Depth (ft.)	15.0		
Sample Description:	Brown silty sand (SM)				

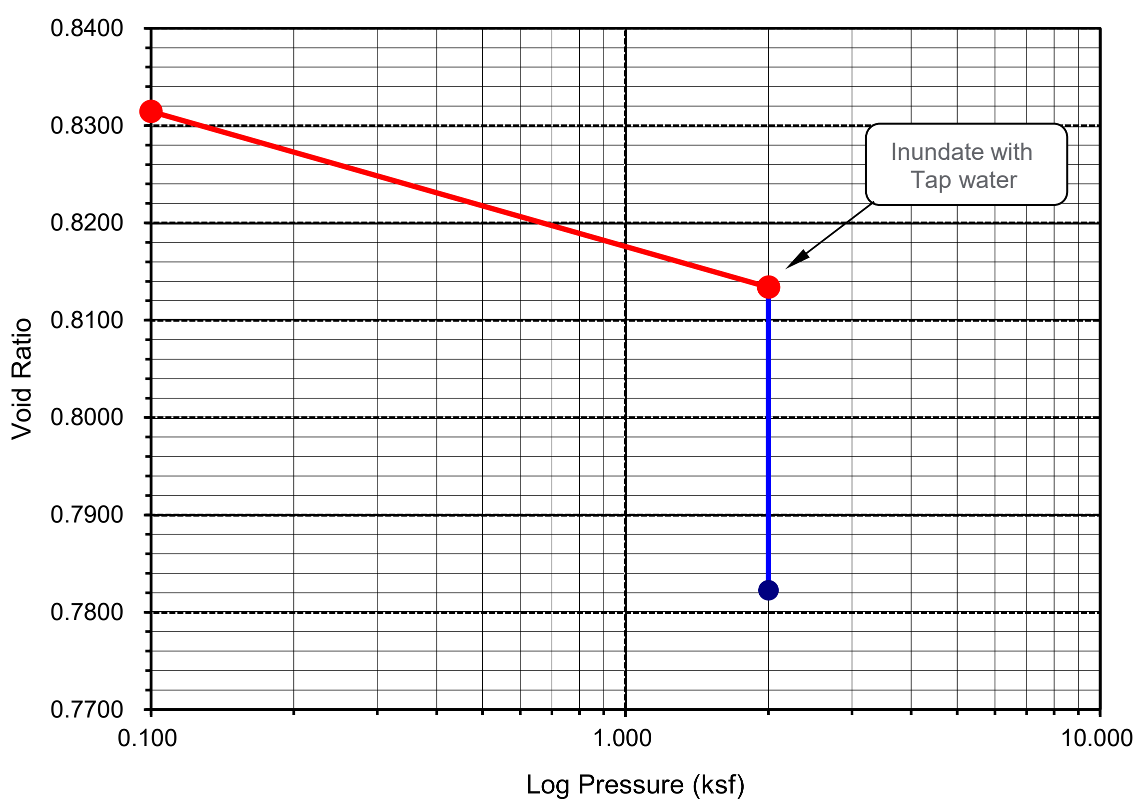
Initial Dry Density (pcf):	92.0
Initial Moisture (%):	8.87
Initial Length (in.):	1.0000
Initial Dial Reading:	0.0680
Diameter(in):	2.415

Final Dry Density (pcf):	95.9
Final Moisture (%) :	22.0
Initial Void ratio:	0.8325
Specific Gravity(assumed):	2.70
Initial Saturation (%)	28.8

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.06855	0.9995	0.00	-0.06	0.8315	-0.06
2.000	0.0849	0.9831	0.65	-1.69	0.8134	-1.04
H2O	0.1019	0.9661	0.65	-3.39	0.7823	-2.74

Percent Swell (+) / Settlement (-) After Inundation = -1.72

Void Ratio - Log Pressure Curve



ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS
ASTM D 4546

Project Name:	Shea - Our Lady of Peace, Irvine	Tested By:	G. Bathala	Date:	05/20/24
Project No.:	23174-01	Checked By:	J. Ward	Date:	05/22/24
Boring No.:	HS-3	Sample Type:	Ring		
Sample No.:	R-4	Depth (ft.)	10.0		
Sample Description:	Brown lean clay with sand (CL)s				

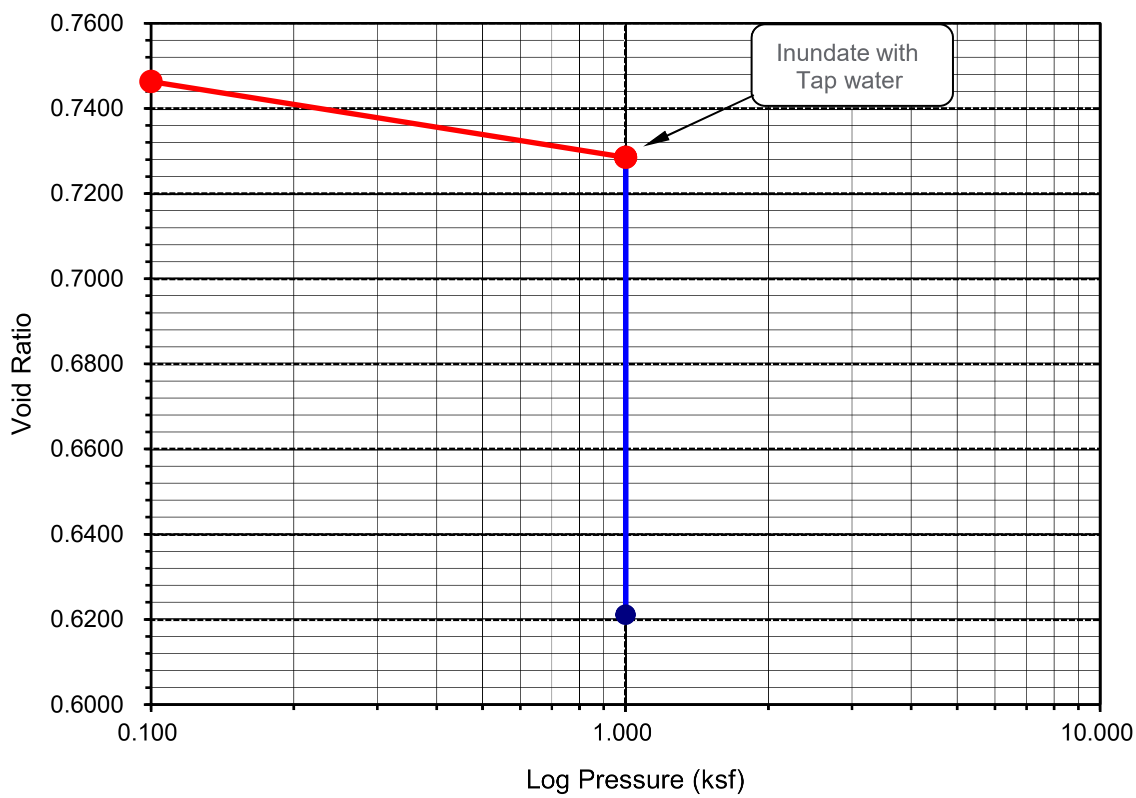
Initial Dry Density (pcf):	96.5
Initial Moisture (%):	5.37
Initial Length (in.):	1.0000
Initial Dial Reading:	0.0967
Diameter(in):	2.415

Final Dry Density (pcf):	104.3
Final Moisture (%) :	19.1
Initial Void ratio:	0.7469
Specific Gravity(assumed):	2.70
Initial Saturation (%)	19.4

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.0970	0.9997	0.00	-0.03	0.7464	-0.03
1.000	0.1087	0.9880	0.15	-1.20	0.7285	-1.05
H2O	0.1702	0.9265	0.15	-7.35	0.6211	-7.20

Percent Swell (+) / Settlement (-) After Inundation = -6.22

Void Ratio - Log Pressure Curve



ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS
ASTM D 4546

Project Name:	Shea - Our Lady of Peace, Irvine	Tested By:	G. Bathala	Date:	05/21/24
Project No.:	23174-01	Checked By:	J. Ward	Date:	05/23/24
Boring No.:	HS-4	Sample Type:	Ring		
Sample No.:	R-4	Depth (ft.)	10.0		
Sample Description:	Brown lean clay (CL)				

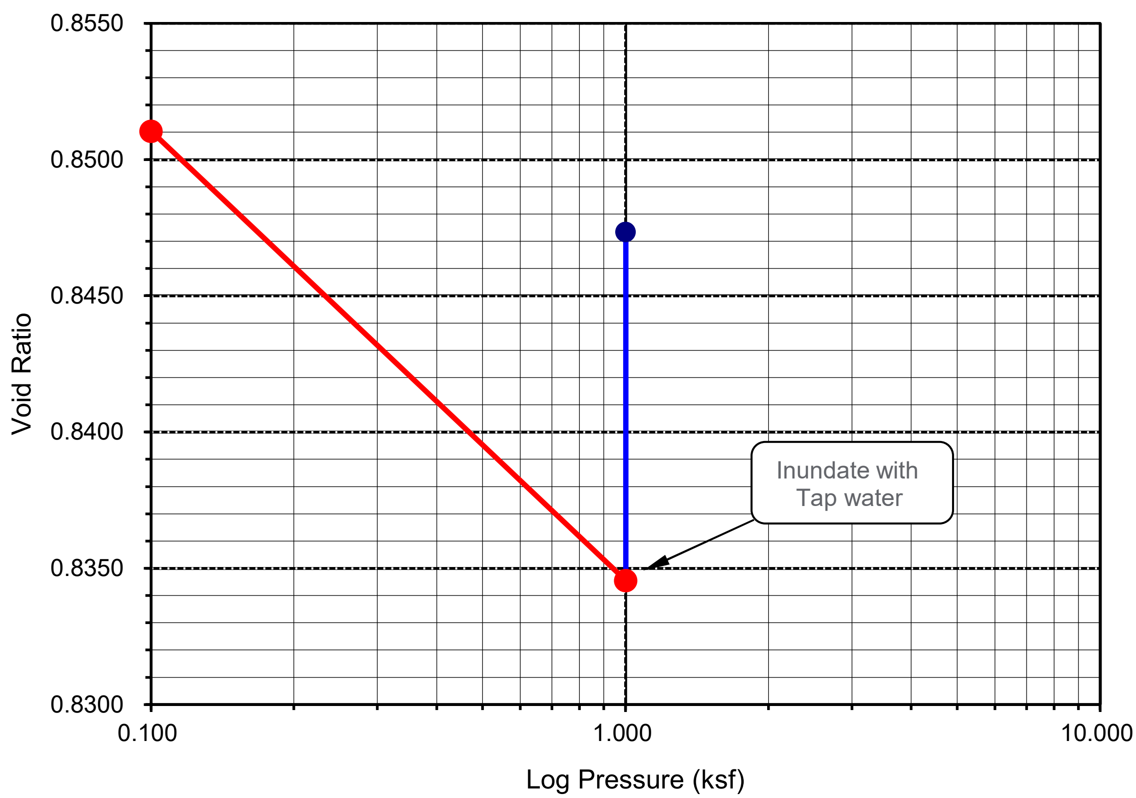
Initial Dry Density (pcf):	91.0
Initial Moisture (%):	13.63
Initial Length (in.):	1.0000
Initial Dial Reading:	0.0961
Diameter(in):	2.415

Final Dry Density (pcf):	91.5
Final Moisture (%) :	24.3
Initial Void ratio:	0.8522
Specific Gravity(assumed):	2.70
Initial Saturation (%)	43.2

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.0967	0.9994	0.00	-0.06	0.8510	-0.06
1.000	0.1071	0.9890	0.15	-1.10	0.8346	-0.95
H2O	0.1002	0.9959	0.15	-0.41	0.8473	-0.26

Percent Swell (+) / Settlement (-) After Inundation = 0.70

Void Ratio - Log Pressure Curve



ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS

ASTM D 4546

Project Name:	Shea - Our Lady of Peace, Irvine	Tested By:	G. Bathala	Date:	03/13/24
Project No.:	23174-01	Checked By:	J. Ward	Date:	03/21/24
Boring No.:	HS-5	Sample Type:	Ring		
Sample No.:	R-4	Depth (ft.)	10.0		
Sample Description:	Olive brown silty clay (CL-ML)				

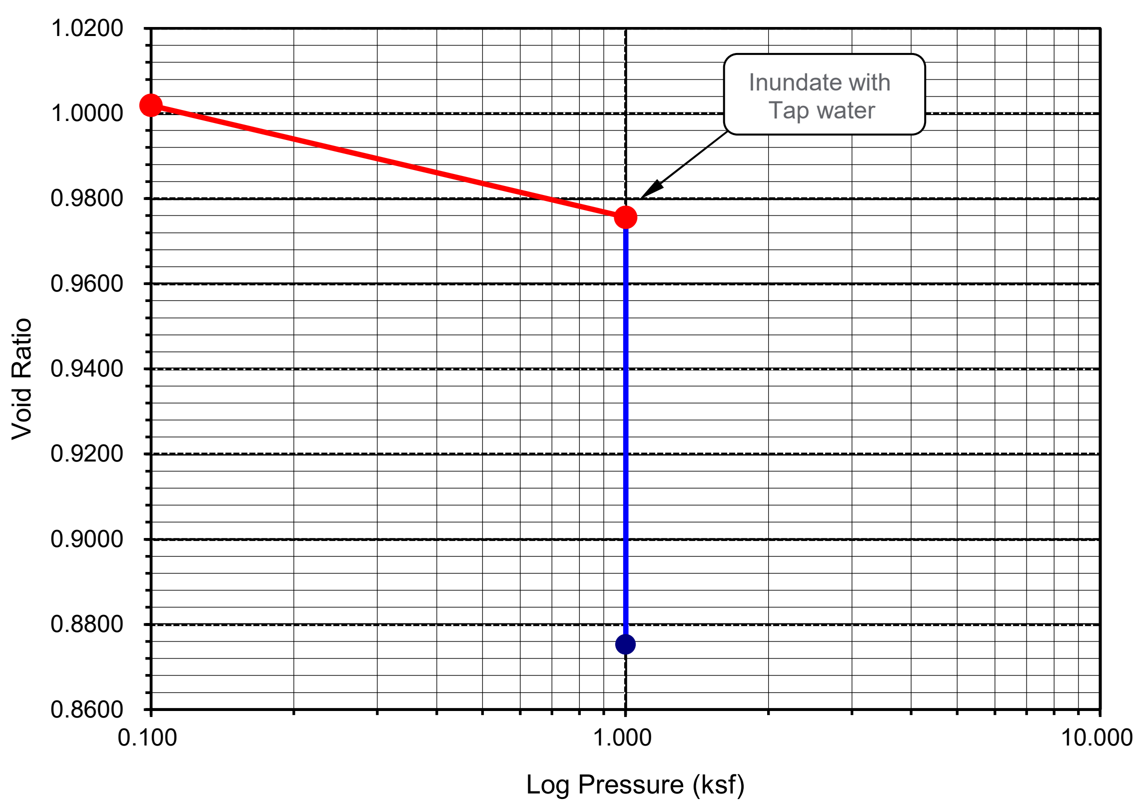
Initial Dry Density (pcf):	84.2
Initial Moisture (%):	12.07
Initial Length (in.):	1.0000
Initial Dial Reading:	0.1519
Diameter(in):	2.415

Final Dry Density (pcf):	90.2
Final Moisture (%) :	25.5
Initial Void ratio:	1.0029
Specific Gravity(assumed):	2.70
Initial Saturation (%)	32.5

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.1524	0.9995	0.00	-0.05	1.0019	-0.05
1.000	0.1670	0.9849	0.15	-1.51	0.9757	-1.36
H2O	0.2171	0.9348	0.15	-6.52	0.8753	-6.37

Percent Swell (+) / Settlement (-) After Inundation = -5.08

Void Ratio - Log Pressure Curve



Appendix D
Infiltration Test Data

Infiltration Test Data Sheet

LGC Geotechnical, Inc

131 Calle Iglesia Suite 200, San Clemente, CA 92672 tel. (949) 369-6141

Project Name: Shea - Our Lady of Peace, Irvine

Project Number: 23174-01

Date: 2/26/2024

Boring Number: I-3

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):	

Minimum test Head (D_o):

(What the sounder tape should read)

Boring Depth - (5 x Boring Radius) 8.4 ft

(Shallow) The value on the sounder tape should be close to this value during testing for **DEEP** testing fill to 4 feet below top of hole

*Pre-Test (Sandy Soil Criteria)**

Trial 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)	Greater Than or Equal to 0.5 feet (yes/no)
1	7:15	7:40	25.0	6.88	6.91	0.03	Yes
2	7:42	8:07	25.0	6.81	6.84	0.03	Yes

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight, and then obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25 inches

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Dt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, DD (feet)	Measured Infiltration Rate(in/hr)
1	8:04	8:34	30.0	6.84	6.87	0.03	0.04
2	8:36	9:06	30.0	6.87	6.89	0.02	0.02
3	9:08	9:38	30.0	6.85	6.87	0.02	0.02
4	9:40	10:10	30.0	6.87	6.9	0.03	0.04
5	10:12	10:42	30.0	6.91	6.94	0.03	0.04
6	10:45	11:15	30.0	6.82	6.85	0.03	0.04
7	11:18	11:48	30.0	6.85	6.87	0.02	0.02
8	11:50	12:20	30.0	6.87	6.89	0.02	0.02
9	12:22	12:52	30.0	6.86	6.88	0.02	0.02
10	12:54	13:24	30.0	6.84	6.86	0.02	0.02
11	13:26	13:56	30.0	6.86	6.88	0.02	0.02
12	13:58	14:28	30.0	6.88	6.90	0.02	0.02
Measured Infiltration Rate (No Factor of Safety)							**0.02**

Sketch:

Notes:

Based on Guidelines from: South Orange County 9/28/2017

Spreadsheet Revised on: 10/30/2019

LGC Geotechnical, Inc.

Infiltration Test Data Sheet

LGC Geotechnical, Inc

131 Calle Iglesia Suite 200, San Clemente, CA 92672 tel. (949) 369-6141

Project Name: Shea - Our Lady of Peace, Irvine
Project Number: 23174-01
Date: 2/26/2024
Boring Number: I-4

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):	

Minimum test Head (D_o):

(What the sounder tape should read) Boring Depth - (5 x Boring Radius) 8.4 ft

(Shallow) The value on the sounder tape should be close to this value during testing for **DEEP** testing fill to 4 feet below top of hole

Pre-Test (Sandy Soil Criteria)*

Trial 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)	Greater Than or Equal to 0.5 feet (yes/no)
1	7:17	7:42	25.0	7.04	7.06	0.02	Yes
2	7:44	8:09	25.0	7.06	7.08	0.02	Yes

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight, and then obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25 inches

Main Test Data

Trial No.	Start Time (24:HR)	Stop Time (24:HR)	Time Interval, Dt (min)	Initial Depth to Water, D_o (feet)	Final Depth to Water, D_f (feet)	Change in Water Level, DD (feet)	Measured Infiltration Rate(in/hr)
1	8:11	8:41	30.0	7.05	7.08	0.03	0.04
2	8:43	9:13	30.0	7.01	7.03	0.02	0.03
3	9:15	9:45	30.0	7.03	7.05	0.02	0.03
4	9:47	10:17	30.0	7.02	7.03	0.01	0.01
5	10:10	10:49	39.0	7.03	7.05	0.02	0.02
6	10:51	11:21	30.0	7.05	7.07	0.02	0.03
7	11:23	11:53	30.0	7.00	7.02	0.02	0.03
8	11:55	12:25	30.0	7.02	7.03	0.01	0.01
9	12:28	12:58	30.0	7.03	7.05	0.02	0.03
10	13:00	13:30	30.0	7.05	7.06	0.01	0.01
11	13:32	14:02	30.0	7.05	7.06	0.01	0.01
12	14:04	14:34	30.0	7.06	7.07	0.01	0.01
						Measured Infiltration Rate (No Factor of Safety)	**0.01**

Sketch:

Notes:

Based on Guidelines from: South Orange County 9/28/2017

Spreadsheet Revised on: 10/30/2019

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.

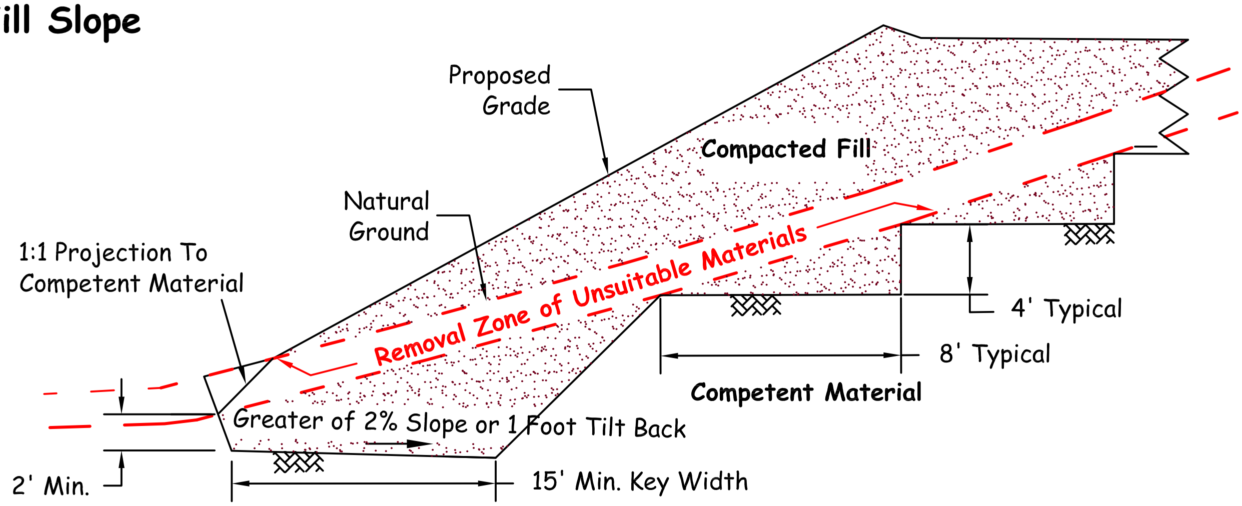
7.2 All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit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

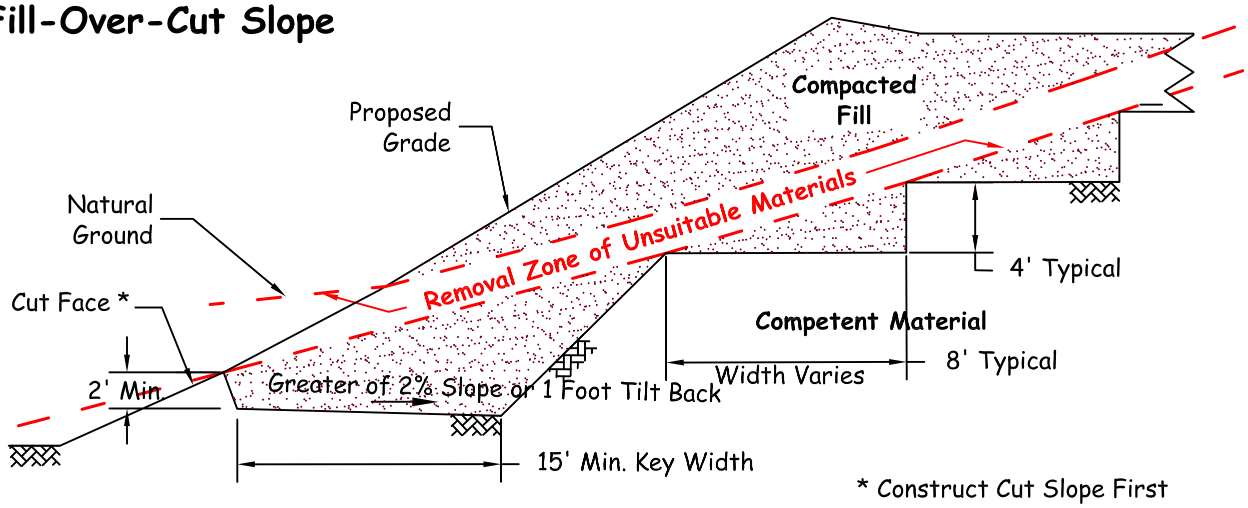
7.4 The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

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.

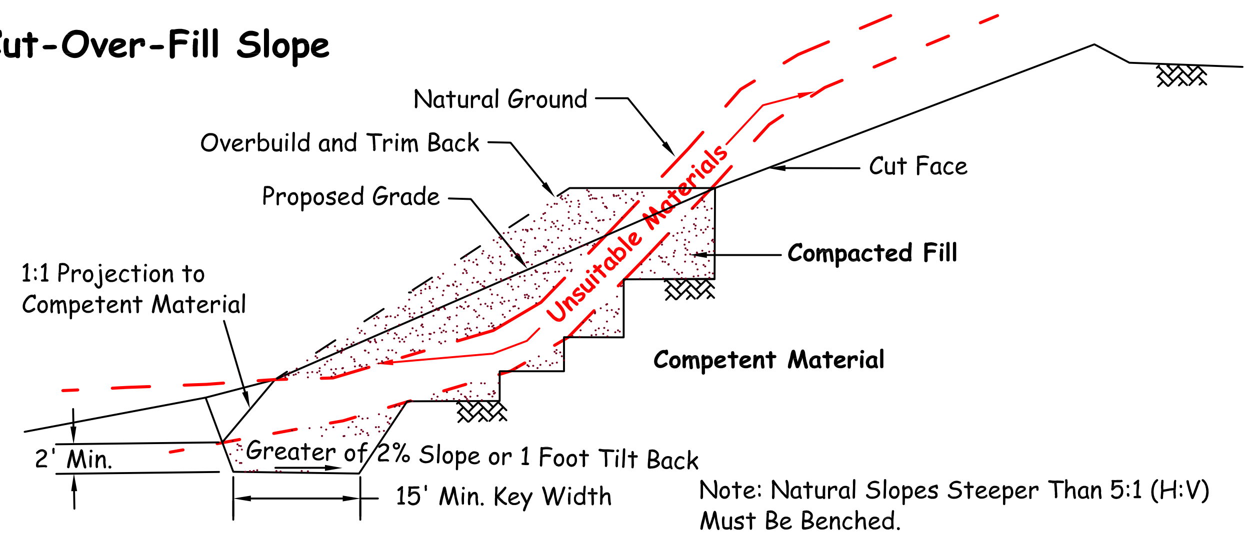
Fill Slope



Fill-Over-Cut Slope

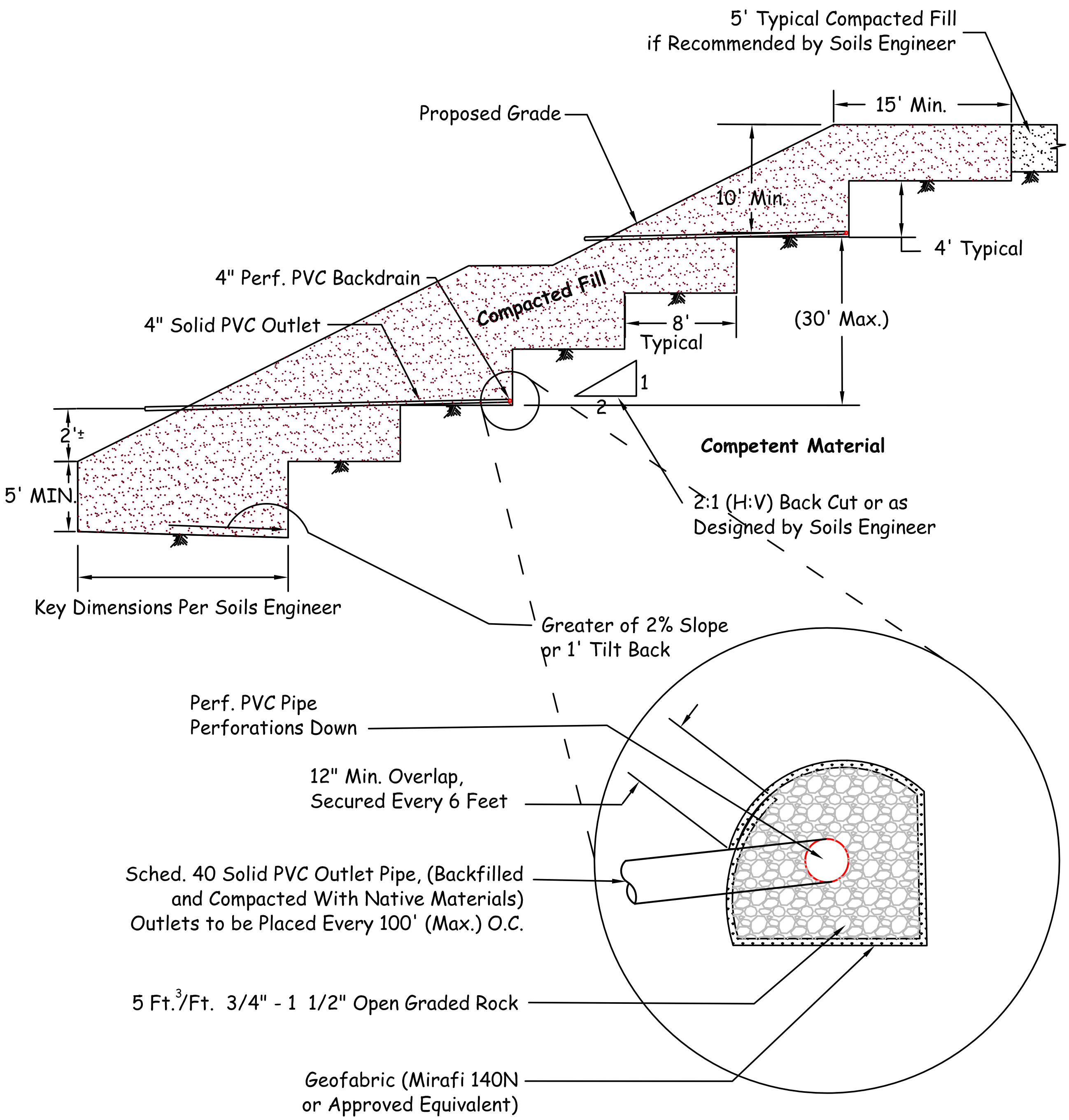


Cut-Over-Fill Slope



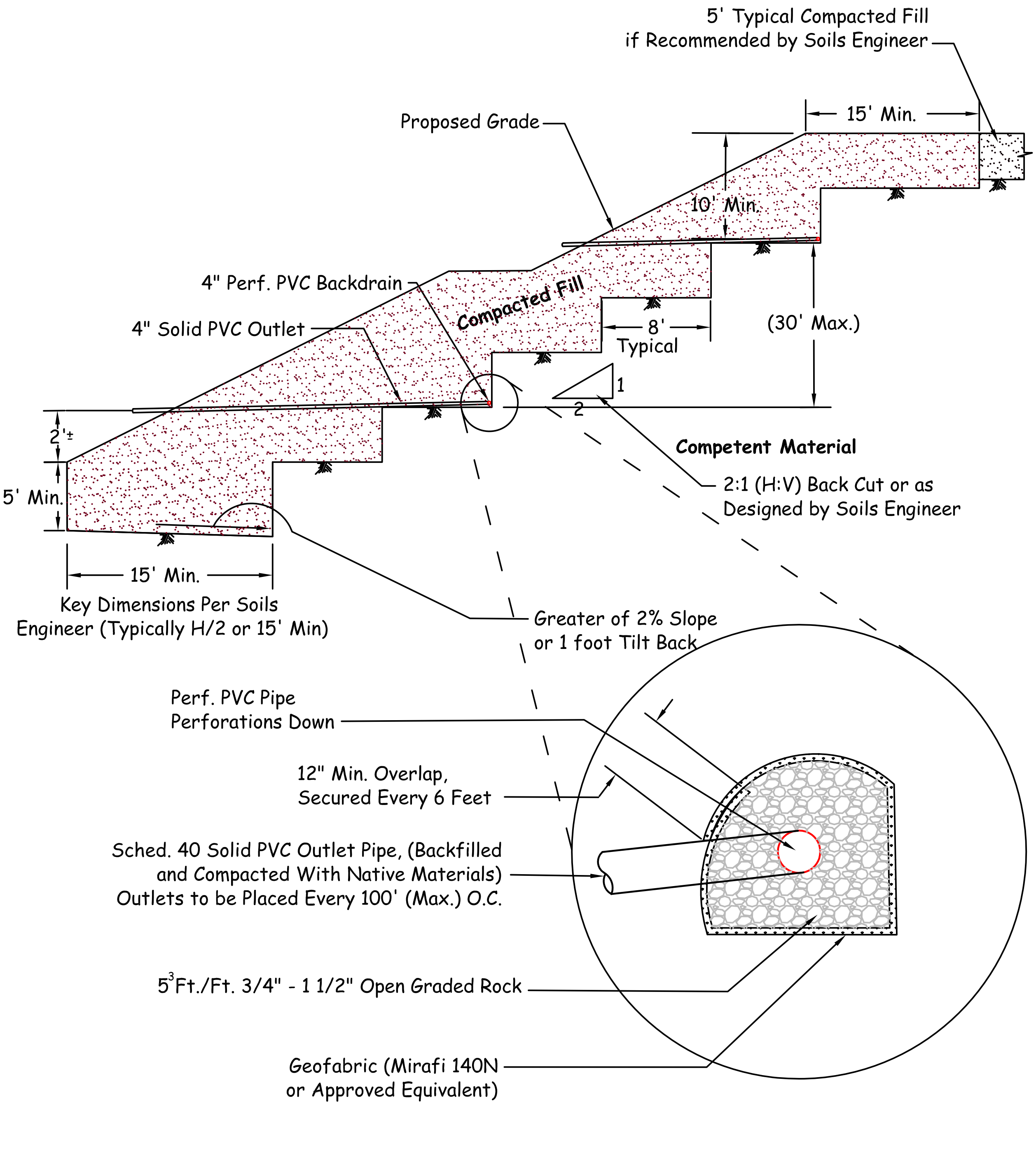
LGC Geotechnical, Inc.

KEYING AND BENCHING



LGC Geotechnical, Inc.

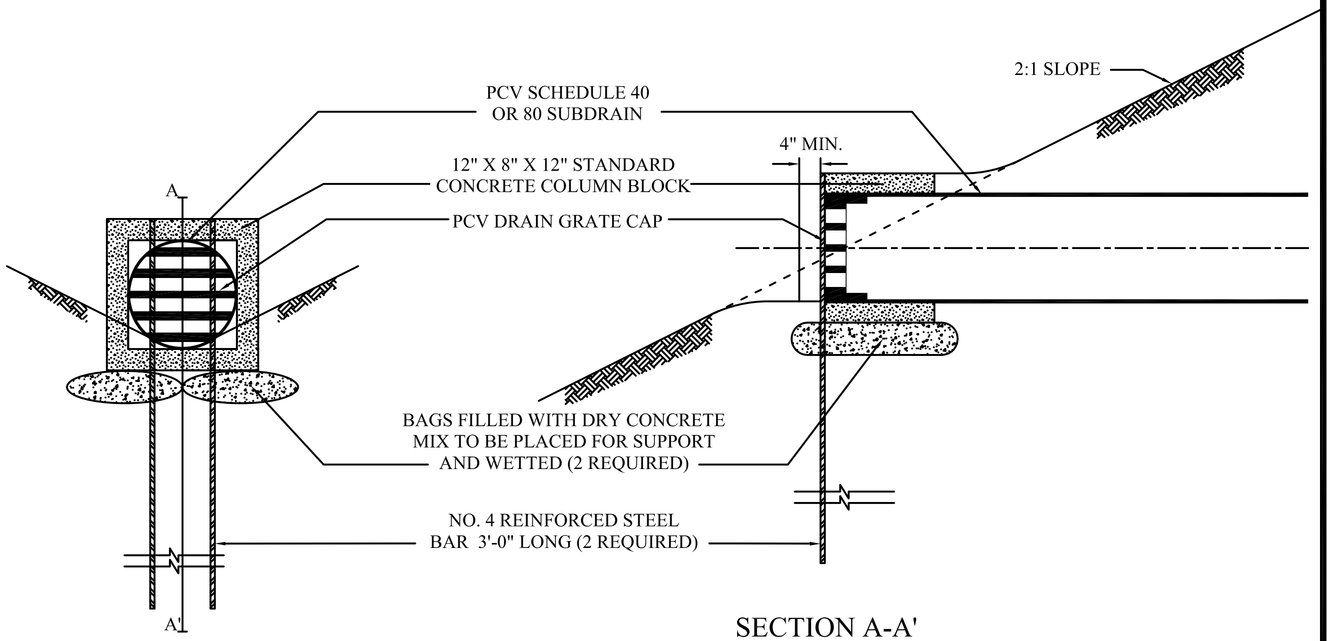
TYPICAL BUTTRESS DETAIL



LGC Geotechnical, Inc.

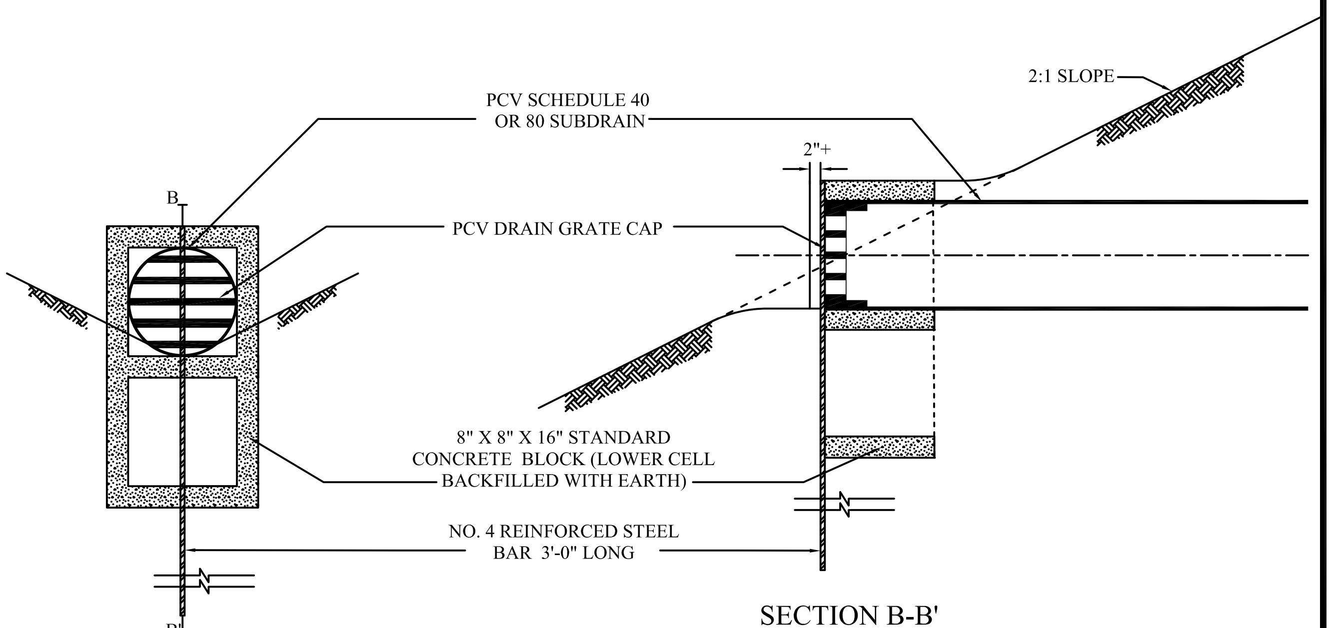
TYPICAL STABILIZATION FILL DETAIL

SUBDRAIN OUTLET MARKER -6" & 8" PIPE



SECTION A-A'

SUBDRAIN OUTLET MARKER -4" PIPE



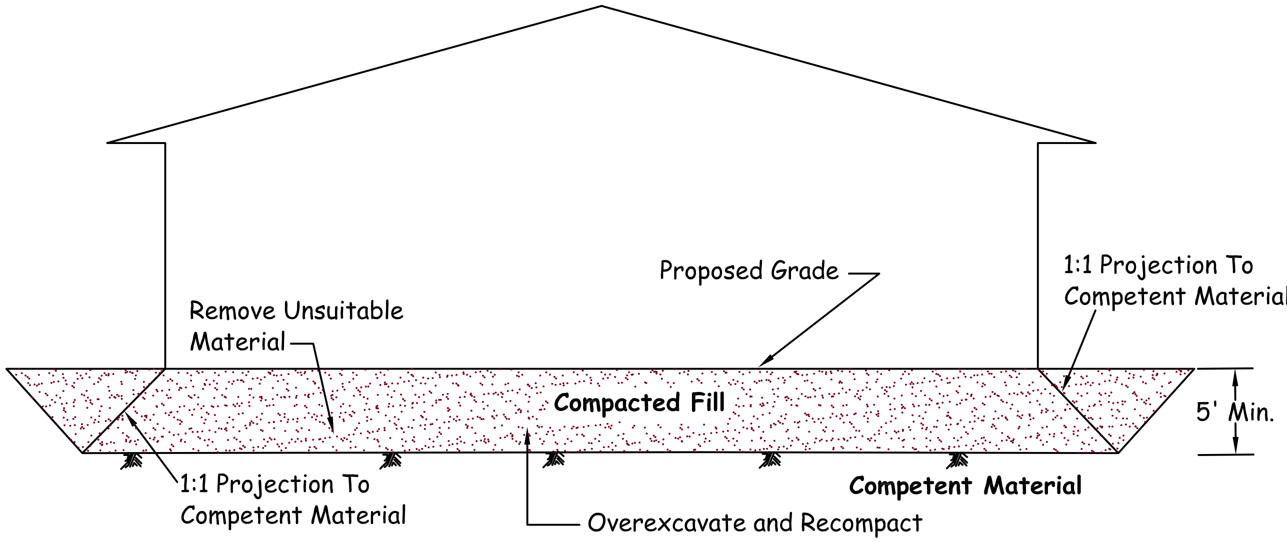
SECTION B-B'

NOT TO SCALE

LGC Geotechnical, Inc.

SUBDRAIN OUTLET MARKER DETAIL

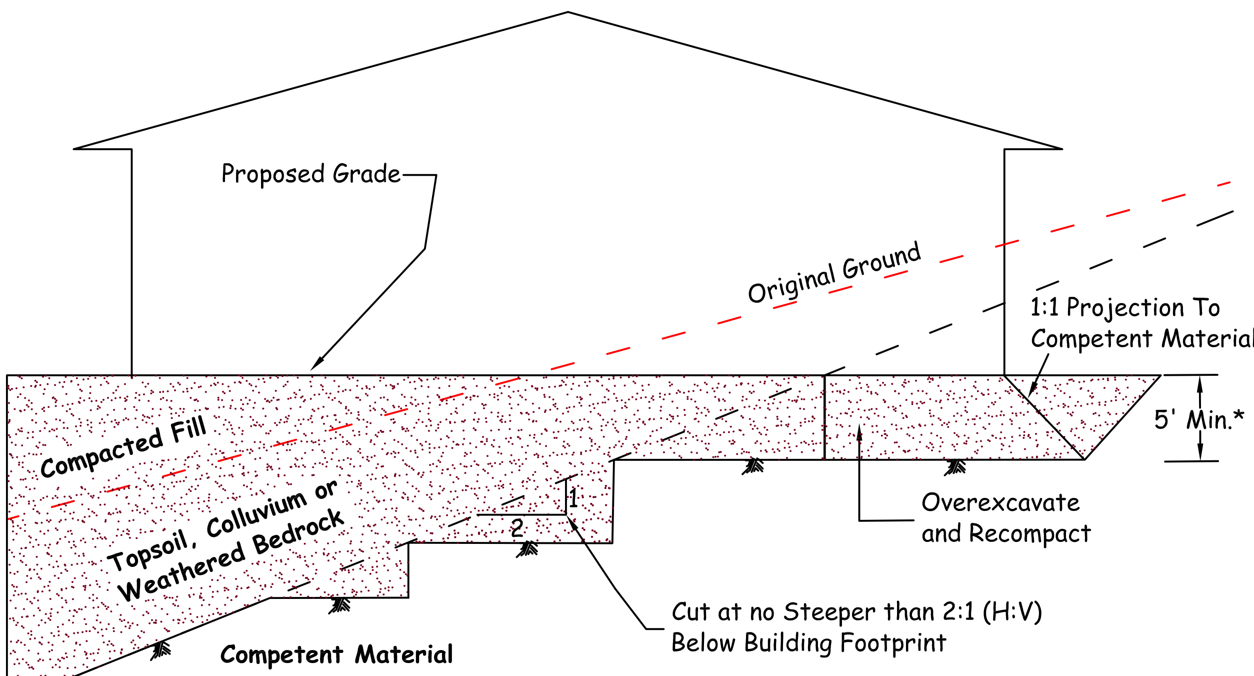
Cut Lot
(Exposing Unsuitable Soils at Design Grade)



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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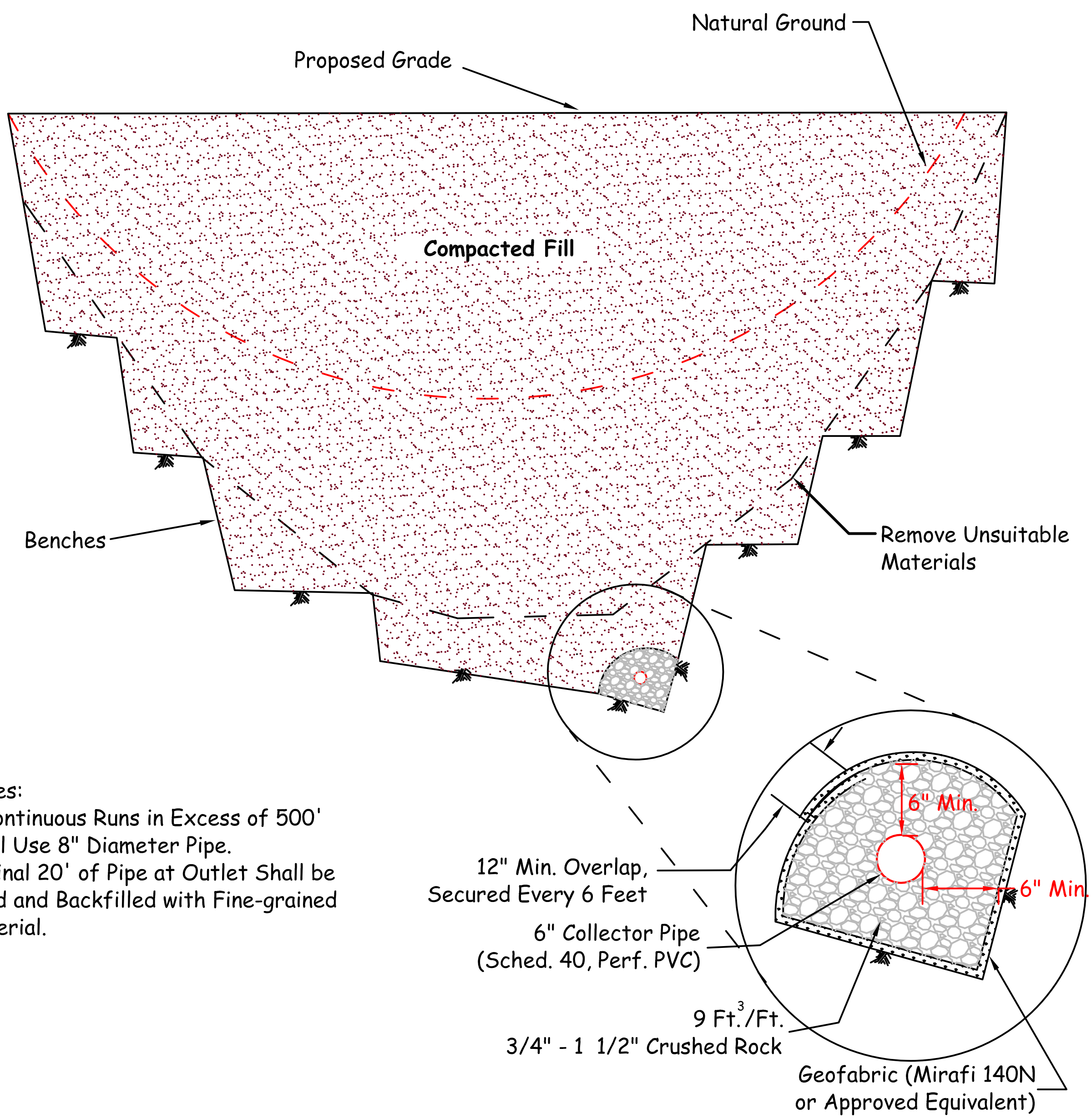
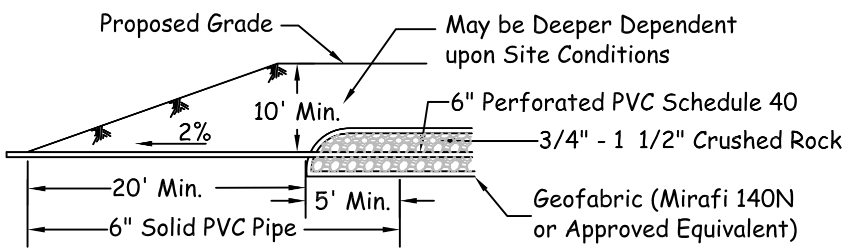
CUT AND TRANSITION LOT OVEREXCAVATION DETAIL



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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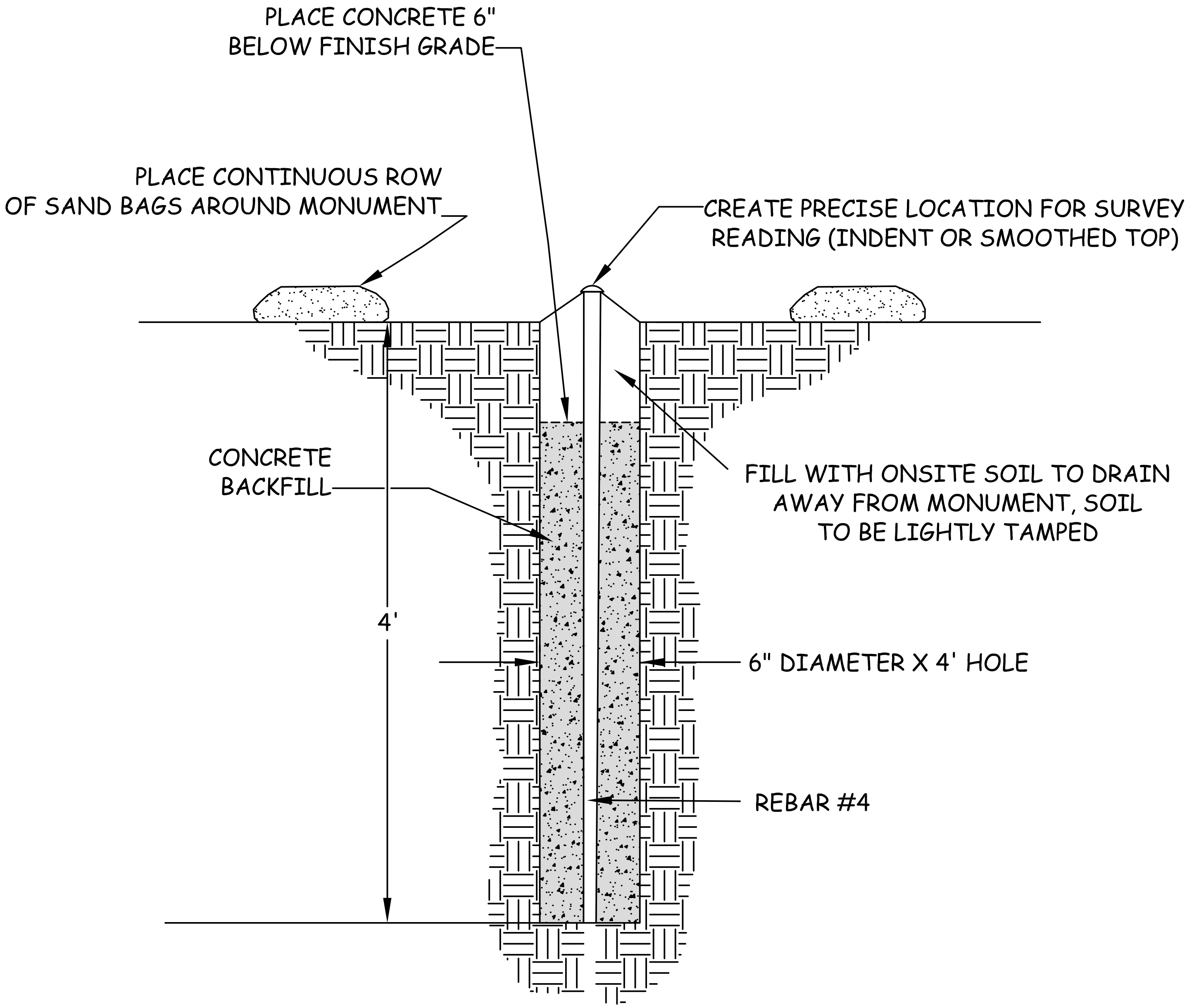
CANYON SUBDRAINS



NO CONSTRUCTION EQUIPMENT WITHIN 25 FEET
OF ANY INSTALLED SETTLEMENT MONUMENTS

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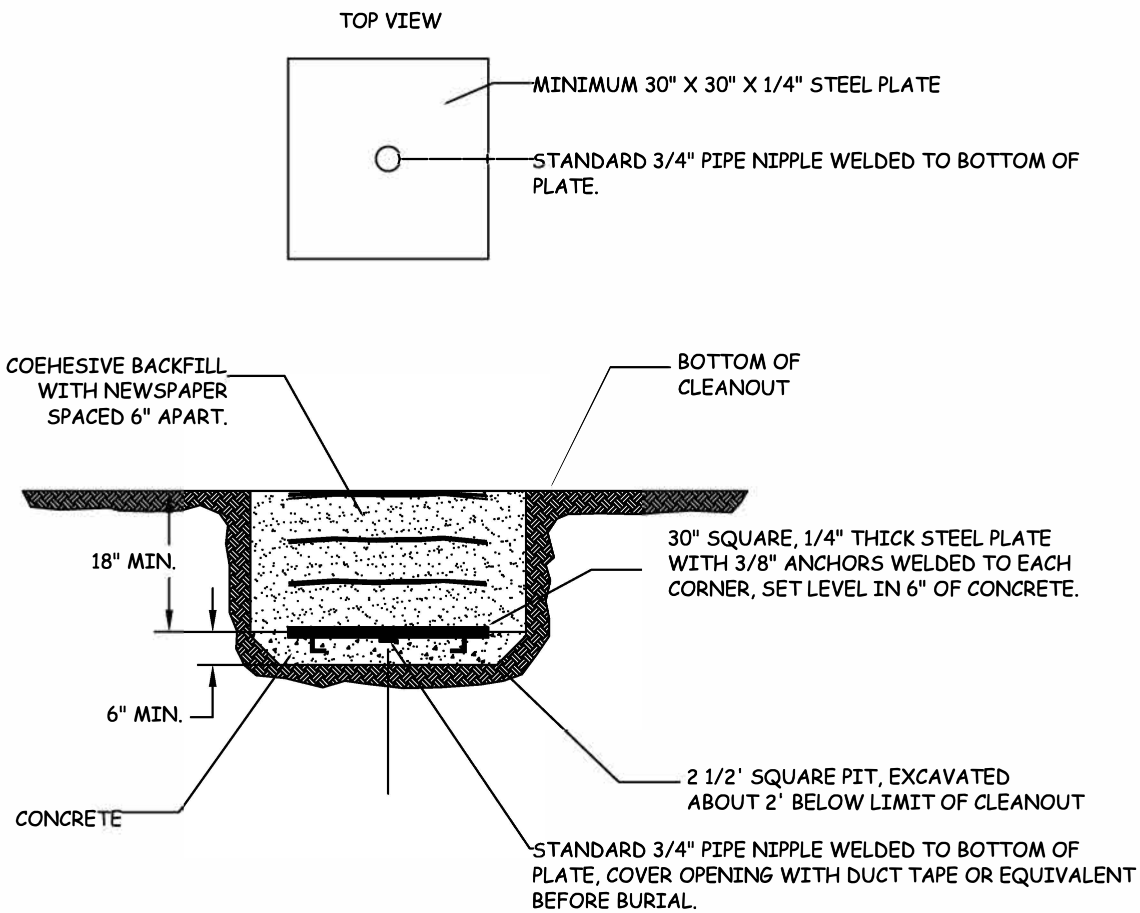
TYPICAL SURFACE SETTLEMENT MONUMENT



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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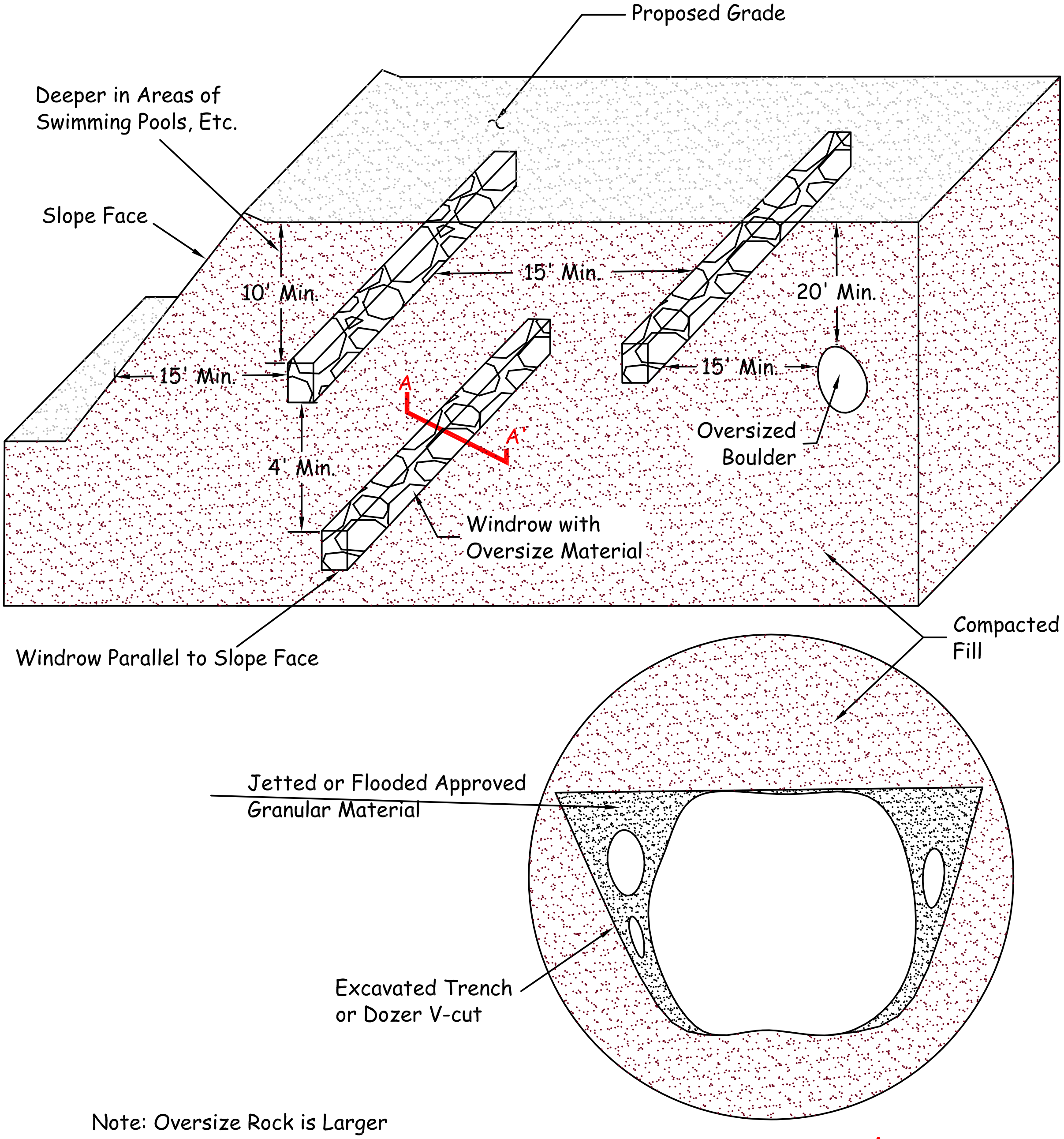
TYPICAL SETTLEMENT PLATE AND RISER



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

Section A-A'

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OVERSIZE ROCK DISPOSAL DETAIL