



**PRELIMINARY GEOTECHNICAL EVALUATION
PROPOSED CHURCH BUILDING
APN 380-112-08-00
SANTEE, CALIFORNIA**

PREPARED FOR

**St. John the Baptizer Ukrainian Catholic Church
P.O. Box 3116
La Mesa, CA 91941**

PREPARED BY

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PROJECT No. 3721-SD

SEPTEMBER 17, 2021



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September 17, 2021
Project No. 3721-SD

St. John the Baptizer Ukrainian Catholic Church
P.O. Box 3116
La Mesa, CA 91941

Attention: Ms. Catherine George

Subject: Preliminary Geotechnical Evaluation
St. John the Baptizer Ukrainian Catholic Church
Carlton Oaks Road and Pike Road (APN 380-112-08-00)
Santee, California 92071

Dear Ms. George:

Presented herein are the results of GeoTek, Inc. (GeoTek) preliminary geotechnical evaluation for the subject project located on the northwest corner of Carlton Oaks Road and Pike Road in City of Santee, California. This report provides geotechnical recommendations for earthwork, foundation design, and construction. Based upon review, planned construction appears feasible from a geotechnical viewpoint provided that the recommendations included in this report are incorporated into the design and construction phases of site development. The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to call GeoTek.

Respectfully submitted,
GeoTek, Inc.



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I. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate the geotechnical conditions on the project site pertinent to the proposed church development. Services provided for this study included the following:

- Research and review of available geologic and geotechnical data, and general information pertinent to the site.
- Excavation of two (2) percolation test borings for infiltration analysis.
- Excavation of five (5) five exploratory test pits and collection of bulk soil samples for subsequent laboratory testing.
- Laboratory testing of soil samples collected in the field during investigation.
- Review and evaluation of site seismicity.
- Compilation of this report presenting GeoTek's findings of pertinent site geotechnical conditions and geotechnical recommendations for site development.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 Site Description

The subject site is located adjacent to the northwest corner of Carlton Oaks Road and Pike Road in the City of Santee, California. The site consists of a single parcel of land identified as County of San Diego Assessor's Parcel Number 380-112-08-00. The site is located south and adjacent to 9308 Pike Road. The general location of the property is presented on Figure 1, Site Location Map.

Existing improvements include a sheet graded building pad at a mean elevation of 335 feet with ascending and descending slopes. Total relief across the building pad is approximately six feet, sloping to the south. The western margin of the property descends 10' at an inclination of 2:1 (horizontal:vertical) into a north-south trending earthen drainage ditch. A six foot tall ascending slope is located in the northern portion of the site. Site grades generally match Carlton Oaks Drive to the south and Pike Road to the east.

2.2 Proposed Development

Based on a draft copy of the project Vesting Tentative Tract Map, provided to GeoTek dated August 30, 2021, the proposed project consists of grading to raise the existing pad to a new pad grade of 339 feet. The tallest fills will be concentrated in the existing drainage swale, where a boxed culvert will be constructed to allow fills to raise grades within the swale. The new grades are designed to support a single-story, approximate 3,620 square foot church facility, a parking lot, pedestrian walkways, landscaping, and utilities.

As site planning progresses and additional or revised plans become available, the plans should be provided to GeoTek for review and comment. Additional geotechnical field exploration, laboratory testing and engineering analyses may be necessary to provide specific earthwork recommendations and geotechnical design parameters for site development.

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 Field Exploration

The field exploration was conducted on August 9, 2021, consisting of a site reconnaissance, excavation of five (5) exploratory test pits advanced with a rubber tracked mini-excavator with a 24-inch bucket, and collection of bulk soil samples for subsequent laboratory testing. A geologist from GeoTek visually logged (based upon the Unified Soil Classification System) the explorations and collected soil samples for laboratory analysis. Additionally, two (2) percolation tests were conducted on August 10, 2021, for infiltration analysis. The approximate locations of the test pits and borings are presented on the Geotechnical Map, Figure 2. Descriptions of materials encountered in the explorations are presented in the Boring Logs in Appendix A.

3.2 Percolation Testing and Infiltration Analysis

Percolation testing was prepared with a flight auger attached to the mini-excavator. The auger was 12-inches in diameter. Construction and percolation testing was conducted in borings P-1 and P-2 by GeoTek in general conformance with the City of Santee's Best Management Practices (BMP) Design Manual. The boreholes were allowed to presoak overnight, and testing was performed on the following day. Percolation testing was performed by adding potable water to the borings, recording the initial depth to water, and allowing the water to percolate for 30 minutes, and the depth to water was then measured. In general, the percolation testing was performed for approximately 6 hours to allow rates to stabilize.

For design of shallow infiltration basins, converting percolation rates to infiltration rates via the Porchet method is generally acceptable and appropriate, as this method factors out the sidewall component of the percolation results and represents the bottom conditions of a shallow basin (infiltration). Therefore, the percolation data were converted to infiltration rates via the Porchet method which is consistent with the guidelines referenced in the City of Santee BMP Design Manual.

A summary of the soil classifications, infiltration rate, and boring location comments are provided in the following table:

Table I			
Log of Percolation Test Borings			
Boring	Soil Description	Infiltration Rate (Inches/Hour)	Comments
P-1	Red brown clayey sand	0.10	Factor of Safety has not been applied
P-2	Red brown clayey sand	0.04	Factor of Safety has not been applied

Copies of the percolation data sheets, and infiltration conversion sheets (Porchet Method) are included in Appendix A. No factors of safety were applied to the rates provided. Over the lifetime of the infiltration areas, the infiltration rates may be affected by sediment build up and biological activities, as well as local variations in near surface soil conditions. A suitable factor of safety should be applied to the field rate in designing the infiltration system.

It should be noted that the infiltration rates provided above were performed in relatively undisturbed on-site soils. Infiltration rates will vary and are mostly dependent on the underlying consistency of the site soils and relative density. Infiltration rates may be impacted by weight of equipment travelling over the soils, placement of engineered fill and other various factors. GeoTek assumes no responsibility or liability for the ultimate design or performance of the storm water facility.

3.3 Laboratory Testing

Laboratory testing was performed on soil samples collected during the field exploration. The purpose of the laboratory testing was to evaluate the physical and chemical soil properties for use in engineering design and analysis. Results of the laboratory testing program, along with a brief description and relevant information regarding testing procedures, are included in Appendix B.

4. GEOLOGIC AND SOILS CONDITIONS

4.1 Regional Setting

The subject property is located in the Peninsular Ranges geomorphic province. The Peninsular Ranges province is one of the largest geomorphic units in western North America. It extends from the north and northeast adjacent the Transverse Ranges geomorphic province to the top of Baja California. This province varies in width from about 30 to 100 miles. It is bounded on the west by the Pacific Ocean, on the south by the Gulf of California and on the east by the Colorado Desert Province.

The Peninsular Ranges are essentially a series of northwest-southeast oriented fault blocks. Several major fault zones are found in this province. The Elsinore Fault zone and the San Jacinto Fault zones trend northwest-southeast and are found in the near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province. The Newport-Inglewood-Rose Canyon Fault zone borders the southwest margin of the province. No active faults are indicated in the immediate site vicinity on the map reviewed for the area.

4.2 Earth Materials

A brief description of the earth materials encountered during the subsurface exploration is presented in the following sections. Based on the site specific subsurface evaluation, the subject site is locally underlain by artificial fill (Af) over old alluvial deposits (map symbol Qoa).

4.2.1 Artificial Fill (Af)

Artificial fill was found in four of the five exploratory test pits, T-2, T-3, T-4, and T-5. Other areas of fills (unmapped) are also likely present on the site. Documentation of existing fill soils were not available for review. Fill soils were generally found to range in thickness from a ½ foot to 2 ½ feet with textures ranging from silty coarse sand with gravel to coarse gravelly sand with some cobbles. Colors were generally noted as reddish brown. Artificial fill soils were found to be dry and loose to medium dense. Numerous trash, debris and asphalt concrete fragments were found in the fill soils within exploration T-2 and an undisturbed asphalt layer, 1.5" thick over 2" base, at a depth of 1.5 feet.

4.2.2 Quaternary Old Alluvium (Qoa)

Quaternary Old Alluvium is the near surface geologic formation underlying the site. Old alluvium was encountered in all 5 test pits at depth ranging from at the surface to 2 ½ feet,

extending to the maximum depth of exploration within the scope of this report. Old alluvial soils were found to range from silty coarse sands to gravelly sands with varying degrees of clay content, reddish brown color, with densities from medium dense to dense, and was generally moist to very moist. Two test pits, TP-2 & TP-4 encountered a sandy clay layer at depths of 7.5 & 5 feet, respectively. In TP-5, old alluvial soils transition to a red-brown sandy silt at a depth of 4 feet. The silt was found to be very moist and hard resulting in practical refusal of the mini excavator at 7 feet.

4.3 Surface Water and Groundwater

4.3.1 Surface Water

Surface water was not observed during the field exploration. If encountered, surface water on this site is likely the result of precipitation along the surface and earthen drainage culvert. Provisions for surface drainage should be addressed by the project designer.

4.3.2 Groundwater

Groundwater was not encountered in the field exploration and is not anticipated to be a factor in the proposed construction. Localized perched groundwater could be present but is also not anticipated to be a factor in site development.

4.4 Earthquake Hazards

4.4.1 Surface Fault Rupture

The geologic structure of the entire southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is not located in a seismically active region. No active or potentially active fault is known to exist at this site nor is the site situated within an "Alquist-Priolo" Earthquake Fault Zone or a Special Studies Zone (Bryant and Hart, 2007). No faults are identified on the geologic maps reviewed for the immediate proximity of the study area.

4.4.2 Liquefaction/Seismic Settlement

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in relatively cohesionless soils. These soils may thereby acquire a high degree of mobility, which can lead to lateral movement, sliding, consolidation and settlement of loose sediments, sand boils and other damaging deformations. This phenomenon occurs only below the water table, but, after liquefaction has developed, the effects can propagate upward into overlying non-saturated soil as excess pore water dissipates.

The factors known to influence liquefaction potential include soil type and grain size, relative density, groundwater level, confining pressures, and both intensity and duration of ground shaking. In general, materials that are susceptible to liquefaction are loose, saturated granular soils having low fines content under low confining pressures.

The liquefaction potential and seismic settlement potential on this site are considered negligible, due to the presence of near surface dense old alluvium and lack of near surface groundwater.

4.4.3 Other Seismic Hazards

Evidence of ancient landslides or slope instabilities at this site was not observed during this study or indicated on regional geologic maps that underly the site. Thus, the potential for landslides is considered negligible.

The potential for secondary seismic hazards such as seiche and tsunami is considered to be remote due to site elevation and distance from an open body of water, as confirmed by the ASCE Tsunami Hazard Tool.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 General Conclusions

Planned construction appears feasible from a geotechnical viewpoint provided that the following recommendations are incorporated in the design and construction phases of the development. The following sections present general recommendations for currently anticipated site development plans. Recommendations contained herein are based on the currently applicable 2019 California Building Code (CBC) and the City of Santee.

5.2 Earthwork Considerations

5.2.1 General

Earthwork should be performed in accordance with the applicable grading ordinances of the City of Santee, the 2019 CBC, and recommendations contained in this report. The Grading Guidelines included in Appendix C outline general procedures and do not anticipate all site-specific situations. In the event of conflict, the recommendations presented in the text of this report should supersede those contained in Appendix C.

5.2.2 Site Clearing and Preparation

Site preparation should start with removal of deleterious materials (e.g., vegetation). These materials should be properly disposed of offsite. If encountered, any existing underground improvements, e.g., footings, utilities and trench backfill, should also be removed, rerouted as appropriate, or be further evaluated as part of site development operations.

The explorations performed for this report were backfilled and compacted by walking the equipment over the surface. Test pit backfill should be removed and replaced with compacted fill.

5.2.3 Remedial Grading

Remedial grading recommendations have been estimated based on the approximate exploration locations. Depending on actual field conditions encountered during grading, locally deeper areas of removal may be necessary. Based on the test pits, artificial fill was present in the upper approximate 2.5 feet of existing ground surface. No documentation of the fills are available and as a result are considered to be compressible and unsuitable to support structural improvements in their current condition. Prior to placement of fill materials potentially compressible materials should be removed. Removals should include all undocumented artificial fill and weathered old alluvial soils below existing grade. It is anticipated that this will include removals to a depth of about three feet below existing grades. The lateral extent of removals should extend to the property limits. Removal bottoms should be relatively uniform in soil type which is not visibly porous and having an in-place density of at least 85 percent of the soil's maximum dry density as determined by ASTM D 1557 test procedures. The bottom of the removals should be observed by a GeoTek representative prior to processing the bottom for receiving placement of compacted fills.

Grading may result in the pad to expose areas of old alluvium at pad grade or with less than three feet of compacted fills under pad grade. This may potentially result in a cut-fill transition to span under a foundation. Therefore, to provide a more uniform bearing surface for foundations, areas with less than 3 feet of compacted fills should be overexcavated a minimum of three feet below designed pad grades or one foot below the base of foundations. Overexcavated cuts should be replaced with engineered fills.

5.2.4 Engineered Fill

Onsite materials are generally considered suitable for reuse as engineered fill, provided they are free from vegetation, roots, debris, and rock/concrete or hard lumps greater than six (6) inches in maximum dimension.

Asphalt debris generated during remedial grading may be placed and compacted within the parking lot area, provided the size does not exceed six inches in maximum dimension, the material is not nested and can be verified to meet project compaction recommendations. Asphalt debris shall not be placed within the building pad or landscape areas of the site.

Engineered fill materials should be moisture conditioned to or slightly above optimum moisture content and compacted in horizontal lifts not exceeding 8 inch in loose thickness to a minimum relative compaction of 90% as determined by ASTM D 1557 test procedures.

5.2.5 Slope Construction

A portion of the existing slope along the north is proposed. Where new fills along the slope will occur, the slope should be reconstructed in accordance with the grading guidelines presented in Appendix C.

5.2.6 Excavation Characteristics

Excavations of onsite materials should generally be accomplished with medium to heavy-duty earthmoving or excavating equipment in good operating condition at least to the depths explored. Excavations in topsoil, weathered old alluvium, and engineered fills constructed from site soils are considered to be Cal OSHA Type C soil. Excavations should conform to current Cal OSHA guidelines. Localized friable material may be encountered and excavation practices may need to be adjusted based on actual conditions exposed.

5.2.7 Shrinkage and Bulking

Several factors will impact earthwork balancing on the site, including bulking of old alluvium, and possible shrinkage of undocumented fill, trench spoil from utilities and footing excavations, as well as the accuracy of topography. Due to the extent of currently proposed work, effects of shrinkage and bulking are anticipated to be minimal.

5.2.8 Trench Excavations and Backfill

Temporary excavations within the onsite materials should be stable at 1:1 inclination for short durations during construction, and where cuts do not exceed 10 feet in height. Temporary cuts to a maximum height of 4 feet can be excavated vertically.

Trench excavations should conform to Cal-OSHA regulations. The contractor should have a competent person, per OSHA requirements, on site during construction to observe conditions and to make the appropriate recommendations.

Utility trench backfill should be compacted to at least 90% relative compaction of the maximum dry density as determined by ASTM D 1557 test procedures. Under-slab trenches should also be compacted to project specifications.

Compaction should be achieved with a mechanical compaction device. Ponding or jetting of trench backfill is not recommended. If backfill soils have dried out, they should be thoroughly moisture conditioned prior to placement in trenches.

5.3 Design Recommendations

5.3.1 Stormwater Infiltration

Many factors control infiltration of surface waters into the subsurface, such as consistency of native soils and bedrock, geologic structure, fill consistency, material density differences, and existing groundwater conditions. Current site plans indicate locations and elevations of the proposed stormwater management systems. Based on the site specific infiltration analysis, the soils may be designed for partial infiltration.

Based on a review of the site soils and proposed design, retaining wall foundations and impacts to offsite utilities are considered to be the primary geotechnical concern. Provided that the sides of the basin are constructed with an impermeable liner, reduction of lateral migration of groundwater is considered to be minimized and reduce the potential to adversely affect proposed retaining walls and offsite utilizes. Settlement and volume changes; lack of slopes, lack of shallow groundwater; a review of properties without environmental impacts adjacent to the site via GeoTracker.com; and other considerations noted in the City of Santee BMP Design Manual were analyzed and found to not be a geotechnical constrained to infiltrating surface waters based on partial infiltration. Percolation and infiltration worksheets relevant to geotechnical design criteria are proved in Appendix A.

5.3.2 Foundation Design Criteria

Foundation design criteria presented herein are for foundations that will bear entirely upon compacted engineered fill prepared in accordance with Section 5.2.3 “Remedial Grading” and are in general conformance with the 2019 CRC. These are typical design criteria based on soil support characteristics only and are not intended to supersede the design by the structural engineer.

Based on the materials encountered on site and as verified by laboratory testing, soils near subgrade can be classified as having a “Very Low” ($0 \leq EI < 20$) expansive potential per ASTM D4829.

The following criteria are for the design of the project’s building foundations.

MINIMUM DESIGN REQUIREMENTS FOR CONVENTIONALLY REINFORCED FOUNDATIONS SUPPORTED ON ENGINEERED FILL	
DESIGN PARAMETER	“Very Low” Expansion Potential ($0 \leq EI \leq 20$)
Foundation Minimum Perimeter Beam Depth (below lowest adjacent finished grade)	12 inches
Minimum Foundation Width*	12 inches
Minimum Slab Thickness (actual)	4 inches
Minimum Slab Reinforcing	No. 3 rebar 18” on-center, placed in the middle 1/3 of the slab
Minimum Footing Reinforcement	Two (4) No. 4 Reinforcing Bars- One (1) top and one (1) bottom
Pre-saturation of Subgrade Soil (percent of optimum moisture content)	Minimum 100% to a depth of 12 inches

*Code minimums per Table 1809.7 of the 2019 CBC should be complied with.

- An allowable bearing capacity of 2,000 pounds per square foot (psf) may be used for design of continuous and perimeter footings that meet the depth and width requirements in the table above. This value may be increased by 400 psf for each additional 12 inches in depth and 200 psf for each additional 12 inches in width to a maximum value of 3,500 psf. Additionally, an increase of one-third may be applied when considering short-term live loads (e.g., seismic and wind loads).
- Based on experience in the area, structural foundations may be designed in accordance with the 2019 CRC, and to withstand a total settlement of 1 inch and maximum differential settlement of one-half of the total settlement over a horizontal distance of 40 feet. These values assume that seismic settlement potential is not a significant constraint.
- The passive earth pressure may be computed as an equivalent fluid having a density of 250 psf per foot of depth, to a maximum earth pressure of 2,000 psf for footings founded on engineered fill. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.
- A grade beam, a minimum of 12 inches wide and 24 inches deep, should be utilized across large entrances, however, the base of the grade beam should be at the same elevation as the bottom of the adjoining footings.

5.3.3 Miscellaneous Foundation Recommendations

- To reduce moisture penetration beneath the slab on grade areas, utility trenches should be backfilled with engineered fill, lean concrete, or concrete slurry where they intercept the perimeter footing or thickened slab edge.
- Spoils from the footing excavations should not be placed in the slab-on-grade areas unless properly compacted and tested. The excavations should be free of loose/sloughed materials and be neatly trimmed at the time of concrete placement.

5.3.4 Under-slab Moisture Membrane

A moisture and vapor retarding system should be placed below slabs-on-grade where moisture migration through the slab is undesirable. Guidelines for these are provided in the 2019 California Green Building Standards Code (CALGreen) Section 4.505.2 and the 2019 California Building Code (CBC) Section 1907.1

It should be realized that the effectiveness of the vapor retarding membrane can be adversely impacted as a result of construction related punctures (e.g., stake penetrations, tears, punctures from walking on the vapor retarder placed atop the underlying aggregate layer, etc.). These occurrences should be limited as much as possible during construction. Thicker membranes are generally more resistant to accidental puncture than thinner ones. Products specifically designed for use as moisture/vapor retarders may also be more puncture resistant.

Moisture and vapor retarding systems are intended to provide a certain level of resistance to vapor and moisture transmission through the concrete, but do not eliminate it. The acceptable level of moisture transmission through the slab is to a large extent based on the type of flooring used and environmental conditions. Ultimately, the vapor retarding system should be comprised of suitable elements to limit migration of water and reduce transmission of water vapor through the slab to acceptable levels. The selected elements should have suitable properties (i.e., thickness, composition, strength, and permeability) to achieve the desired performance level. Moisture retarders can reduce, but not eliminate, moisture vapor rise from the underlying soils up through the slab. Moisture retarder systems should be designed and constructed in accordance with applicable American Concrete Institute, Portland Cement Association, Post-Tensioning Concrete Institute, ASTM and California Building Code requirements and guidelines.

GeoTek does not practice in the field of moisture vapor transmission evaluation/migration since that practice is not a geotechnical discipline. Therefore, GeoTek recommends that a qualified person, such as the flooring contractor, structural engineer, architect, and/or other experts specializing in moisture control within the building be consulted to evaluate the general and

specific moisture and vapor transmission paths and associated potential impact on the proposed construction. That person (or persons) should provide recommendations relative to the slab moisture and vapor retarder systems and for migration of potential adverse impact of moisture vapor transmission on various components of the structures, as deemed appropriate. In addition, the recommendations in this report and GeoTek's services in general are not intended to address mold prevention; since GeoTek, along with geotechnical consultants in general, do not practice in the area of mold prevention. If specific recommendations addressing potential mold issues are desired, then a professional mold prevention consultant should be contacted.

5.3.5 Foundation Set Backs

Where applicable, the following setbacks should apply to all foundations. Any improvements not conforming to these setbacks may be subject to lateral movements and/or differential settlements:

- The outside bottom edge of all footings should be set back a minimum of 7 feet from the face of any descending slope.
- The bottom of all footings for structures near retaining walls should be deepened to extend below a 1:1 projection upward from the bottom inside edge of the wall footing. This applies to the existing retaining walls along the perimeter if they are to remain.
- The bottom of any foundations for structures should be deepened to extend below a 1:1 projection upward from the bottom of the nearest excavation (e.g., utility trenches).

5.3.6 Seismic Design Parameters

The site is located at approximately 32.8461 West Latitude and -116.9975 North Longitude. Site spectral accelerations (S_s and S_1), for 0.2 and 1.0 second periods for a risk targeted two (2) percent probability of exceedance in 50 years (MCER) were determined using the web interface provided by SEAOC/OSHPD (<https://seismicmaps.org>) to access the USGS Seismic Design Parameters. A Site Class "C" has been utilized based on the apparent density of the site soils (old alluvium).

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_s	0.774g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.285g
Site Coefficient for Site Class "C", F_a	1.2
Site Coefficient for Site Class "C", F_v	1.5
Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration for 0.2 Second, S_{MS}	0.929g
Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration for 1.0 Second, S_{M1}	0.427g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	0.619g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.285g

5.3.7 Soil Sulfate Content

The sulfate content was determined in the laboratory for a soil sample collected during the field investigation. The results indicate that the water-soluble sulfate is greater than 0.2 percent and less than 2.00 percent by weight (0.2216 percent), which is considered exposure class "S2" as per Table 19.3.1.1 of ACI 318-19. Recommendations, as contained in ACI Table 19.3.2.1, for concrete for this exposure class include: a maximum water/cement ratio of 0.45, minimum strength of concrete (f'_c) of 4500 pounds per square inch (psi) and Type V cement.

The soil resistivity at this site was tested by others on a sample collected during the field investigation. The results of the testing indicate that the on-site soils are considered "extremely corrosive" (308 ohm-cm) (Roberge, 2000) to buried ferrous metal in accordance with current standards used by corrosion engineers. It is recommended that a corrosion engineer be consulted to provide recommendations for the protection of buried ferrous metal at this site.

Exposure classification "S2" is considered to be unique in this area. After rough grading has been performed, at least four sulfate tests of the near surface pad grades should be obtained to determine if this condition remains.

5.4 RETAINING WALL DESIGN AND CONSTRUCTION

5.4.1 General Design Criteria

Recommendations presented herein may apply to typical masonry or concrete vertical retaining walls to a maximum height of 6 feet. Additional review and recommendations should be requested for higher walls.

Retaining wall foundations embedded a minimum of 18 inches into engineered fill or dense formational materials should be designed using an allowable bearing capacity of 2,000 psf. This value may be increased by 400 psf for each additional 12 inches in depth and 200 psf for each additional 12 inches in width to a maximum value of 3,500 psf. An increase of one-third may be applied when considering short-term live loads (e.g., seismic or wind loads). The passive earth pressure may be computed as an equivalent fluid having a density of 250 psf per foot of depth, to a maximum earth pressure of 2,000 psf. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

Where retaining walls are designed as part of the stormwater management basin, the bottom of the retaining wall foundations should be extended to one foot below the infiltrating bottom of the basin.

An equivalent fluid pressure approach may be used to compute the horizontal active pressure against the wall. The appropriate fluid unit weights are given in the table below for specific slope gradients of retained materials.

Surface Slope of Retained Materials (H:V)	Equivalent Fluid Pressure (PCF) Select Backfill*
Level	45
2:1	60

*Select backfill should consist of imported sand other approved materials with an $SE > 30$ and an $EI \leq 20$.

The above equivalent fluid weights do not include other superimposed loading conditions such as expansive soil, vehicular traffic, structures, seismic conditions, or adverse geologic conditions.

5.4.2 Wall Backfill and Drainage

Wall backfill should include a minimum one (1) foot wide section of $\frac{3}{4}$ to 1-inch clean crushed rock (or approved equivalent). The rock should be wrapped in Mirafi 140N or an approved equivalent and placed immediately along the back of wall and extend up from the backdrain to within approximately 12 inches of finish grade. The upper 12 inches should consist of compacted onsite materials. Alternatively, a manufactured wall drainage product (example: Mira Drain 6000) may be used for wall drainage. Any such product should be installed in conformance with the manufacturer's recommendations. If the walls are designed using the "select" backfill design parameters, then the "select" materials shall be placed within the active zone as defined by a 1:1 (H:V) projection from the back of the retaining wall footing up to the retained surface behind the

wall. Presence of other materials might necessitate revision to the parameters provided and modification of wall designs.

The backfill materials should be placed in lifts no greater than eight (8) inches in thickness and compacted to at least 90% relative compaction as determined by ASTM Test Method D 1557 test procedures. Proper surface drainage needs to be provided and maintained. Water should not be allowed to pond behind retaining walls. Waterproofing of site walls should be performed where moisture migration through the wall is undesirable.

Retaining walls should be provided with an adequate pipe and gravel back drain system to reduce the potential for hydrostatic pressures to develop. A 4-inch diameter perforated collector pipe (Schedule 40 PVC, or approved equivalent) in a minimum of one cubic foot per lineal foot of 3/8 to one-inch clean crushed rock or equivalent, wrapped in filter fabric should be placed near the bottom of the backfill and be directed (via a solid outlet pipe) to an appropriate disposal area. Maximum horizontal spacing between drain outlets should be 100 feet.

Walls from two (2) to four (4) feet in height may be drained using localized gravel packs behind weep holes at 10 feet maximum spacing (e.g., approximately 1.5 cubic feet of gravel in a woven plastic bag). Weep holes should be provided, or the head joints omitted in the first course of block extended above the ground surface. However, nuisance water may still collect in front of the wall.

Drain outlets should be maintained over the life of the project and should not be obstructed or plugged by adjacent improvements.

5.0 Preliminary Pavement Design

Traffic indices have not been provided during this stage of site planning. In addition, site conditions have not been graded to a final design to evaluate specific pavement subgrade conditions. Therefore, the minimum structural sections based on the City of Santee's Engineers Design and Processing Manual's Streets-Design Criteria (Santee, 2017) are presented below.

PRELIMINARY ASPHALT PAVEMENT STRUCTURAL SECTION		
Design Criteria [†]	Asphaltic Concrete (AC) Thickness (inches)	Aggregate Base (AB) Thickness (inches)
Parking Area	4.0	6.0

As noted in the Urban Street Design document, actual structural pavement design is to be determined by the geotechnical engineer's testing (R-Value) of the subgrade. Thus, the actual R-Value of the subgrade soils can only be determined at the completion of grading for street subgrades and the above values are subject to change based laboratory testing of the as-graded soils near subgrade elevations.

Asphalt concrete and aggregate base should conform to current Caltrans Standard Specifications Section 39 and 26-1.02, respectively. As an alternative, asphalt concrete can conform to Section 203-6 of the current Standard Specifications for Public Work (Green Book). Crushed aggregate base or crushed miscellaneous base can conform to Section 200-2.2 and 200-2.4 of the Green Book, respectively. Pavement base should be compacted to at least 95 percent of the laboratory maximum dry density as determined by ASTM D 1557 test procedures.

All pavement installation, including preparation and compaction of subgrade, compaction of base material, placement and rolling of asphaltic concrete, should be done in accordance with the City of Santee specifications, and under the observation and testing of GeoTek and a City Inspector where required. Jurisdictional minimum compaction requirements in excess of the aforementioned minimums may govern.

5.0 Post Construction Considerations

6.1 Landscape Maintenance and Planting

Water has been shown to weaken the inherent strength of soil, and slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from graded slopes should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Controlling surface drainage and runoff and maintaining a suitable vegetation cover can limit erosion. Plants selected for landscaping should be lightweight, deep-rooted types that require little water and can survive the prevailing climate.

Overwatering should be avoided. The soils should be maintained in a solid to semi-solid state as defined by the materials Atterberg Limits. Care should be taken when adding soil amendments to avoid excessive watering. Leaching as a method of soil preparation prior to planting is not recommended. An abatement program to control ground-burrowing rodents should be implemented and maintained. This is critical as burrowing rodents can decreased the long-term performance of slopes.

It is common for planting to be placed adjacent to structures in planter or lawn areas. This will result in the introduction of water into the ground adjacent to the foundation. This type of

landscaping should be avoided. If used, then extreme care should be exercised with regard to the irrigation and drainage in these areas. Waterproofing of the foundation and/or subdrains may be warranted and advisable. GeoTek could discuss these issues, if desired, when plans are made available.

5.2 Drainage

The need to maintain proper surface drainage and subsurface systems cannot be overly emphasized. Positive site drainage should be maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and not allowed to pond or seep into the ground adjacent to the footings. Site drainage should conform to Section 1804.4 of the 2019 CRC. Roof gutters and downspouts should discharge onto paved surfaces sloping away from the structure or into a closed pipe system which outfalls to the street gutter pan or directly to the storm drain system. Pad drainage should be directed toward approved areas and not be blocked by other improvements. It shall be noted that the upper pad currently includes a swale running through the middle of the pad and drainage changes must be made by the Civil Engineer of Record.

It is the owner's responsibility to maintain and clean drainage devices on or contiguous to their lot. In order to be effective, maintenance should be conducted on a regular and routine schedule and necessary corrections made prior to each rainy season.

5.3 Construction Observations

It is recommended that changes to site grading, specifications, and any retaining wall/shoring plans and foundation plans be reviewed by this office prior to construction to check for conformance with the recommendations of this report. Additional recommendations may be necessary based on these reviews. It is also recommended that GeoTek representatives be present during site grading and foundation construction to check for proper implementation of the geotechnical recommendations. The owner/developer should have GeoTek's representative perform at least the following duties:

- Observe site clearing and grubbing operations for proper removal of unsuitable materials.
- Observe and test bottom of removals prior to fill placement.
- Evaluate the suitability of on-site and import materials for fill placement and collect soil samples for laboratory testing when necessary.
- Observe the fill for uniformity during placement including utility trenches.
- Observe and test the fill for field density and relative compaction.
- Observe and probe foundation excavations to confirm suitability of bearing materials.

If requested, a construction observation and compaction report can be provided by GeoTek, which can comply with the requirements of the governmental agencies having jurisdiction over the project. It is recommended that these agencies be notified prior to commencement of construction so that necessary grading permits can be obtained.

6. LIMITATIONS

The scope of this evaluation is limited to the area explored that is shown on the Geotechnical Map (Figure 2). This evaluation does not and should in no way be construed to encompass any areas beyond the specific area of proposed construction as indicated to us by the client. Further, no evaluation of any existing site improvements is included. The scope is based on GeoTek's understanding of the project and the client's needs, GeoTek's proposal (Proposal No. P-0600521-SD) dated August 2, 2021, and geotechnical engineering standards normally used on similar projects in this region.

The materials observed on the project site appear to be representative of the area; however, soil and bedrock materials vary in character between excavations and natural outcrops, or conditions exposed during site construction. Site conditions may vary due to seasonal changes or other factors. GeoTek, Inc. assumes no responsibility or liability for work, testing or recommendations performed or provided by others.

Since the recommendations contained in this report are based on the site conditions observed and encountered, and laboratory testing, GeoTek's conclusions and recommendations are professional opinions that are limited to the extent of the available data. Observations during construction are important to allow for any change in recommendations found to be warranted. These opinions have been derived in accordance with current standards of practice and no warranty is expressed or implied. Standards of practice are subject to change with time.

7. SELECTED REFERENCES

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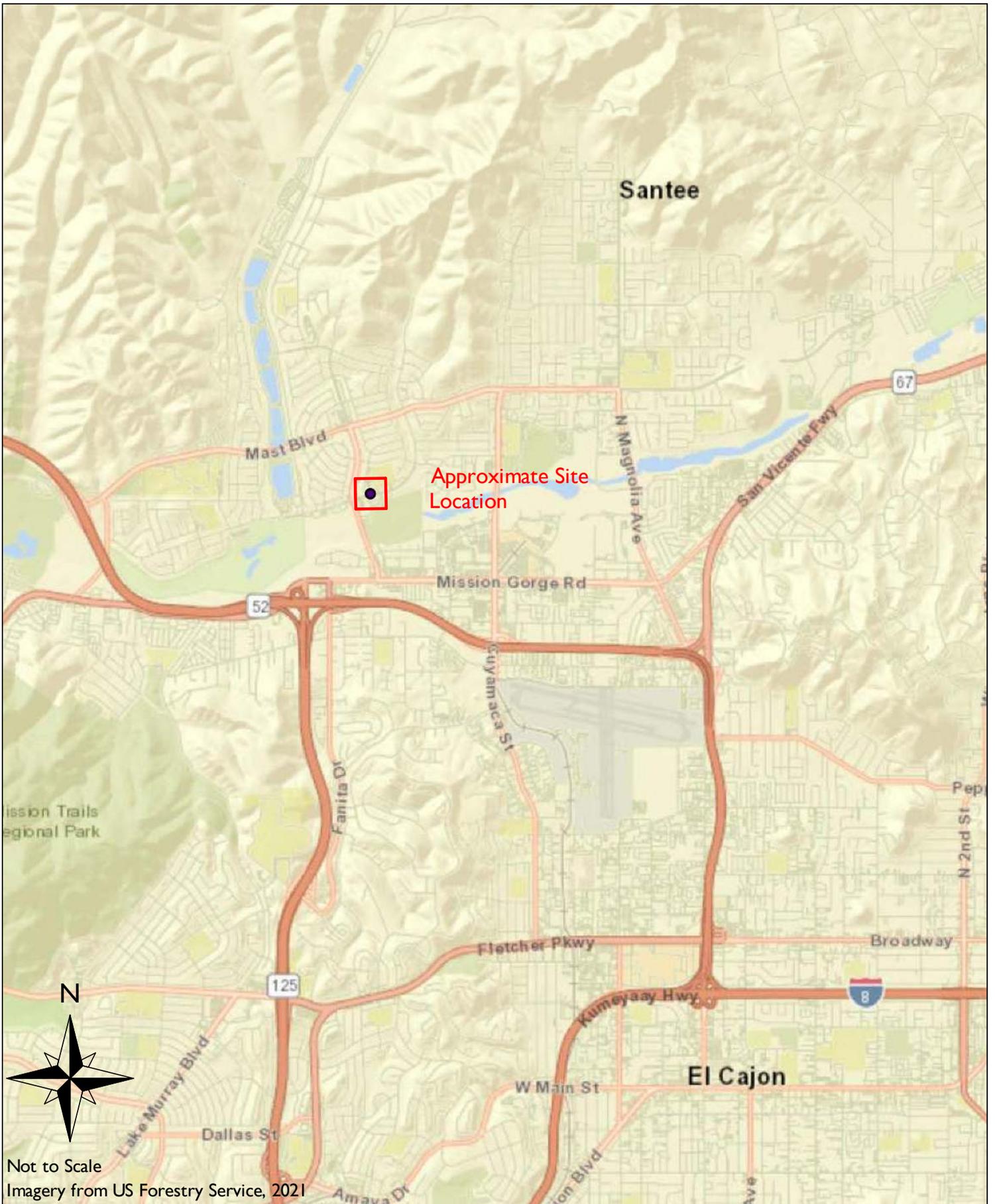
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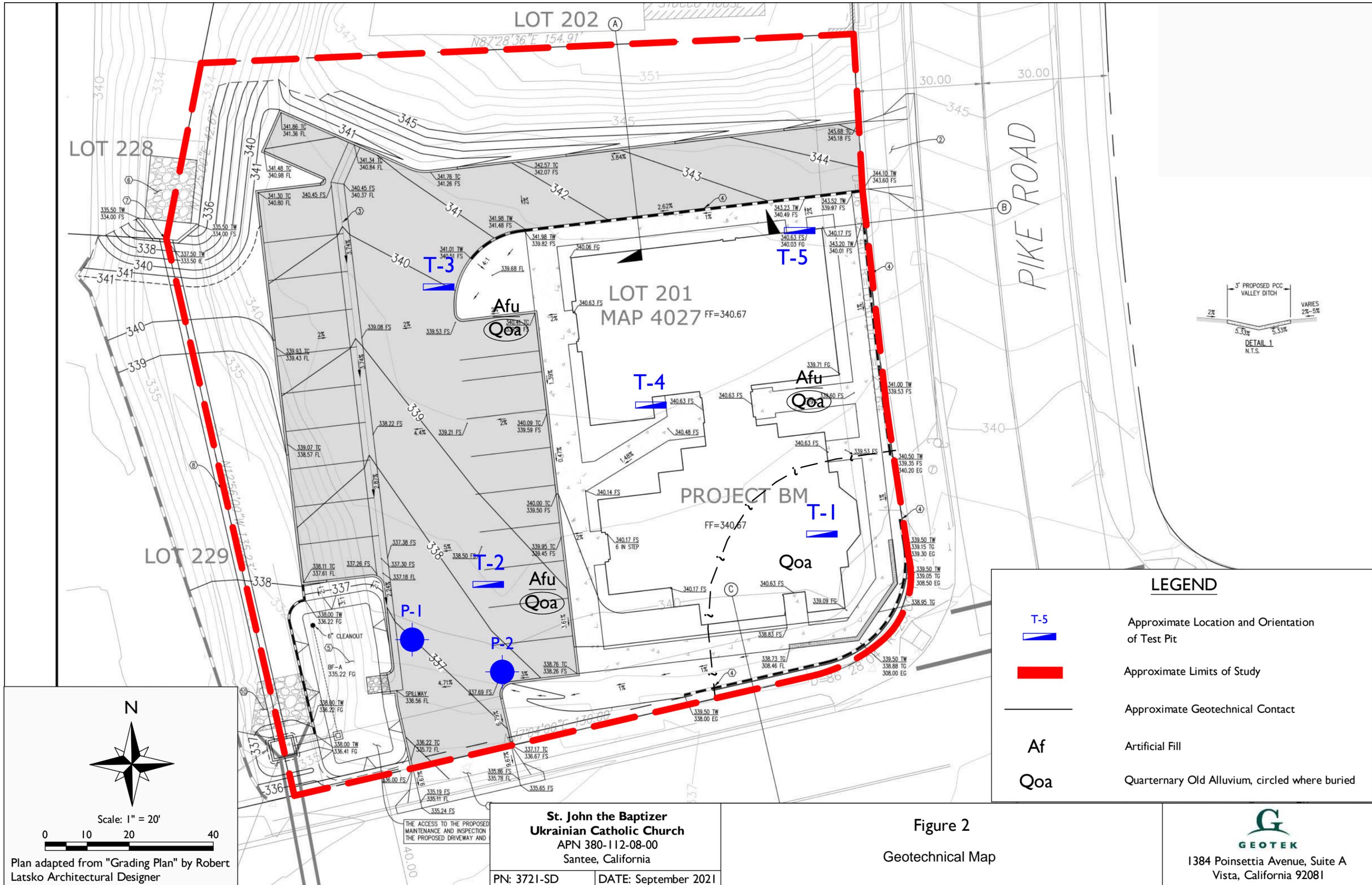


**St. John the Baptizer
Ukrainian Catholic Church**
 APN 380-112-08-00
 Santee, California

PN: 3721-SD DATE: September 2021

Figure I
 Site Location Map

GEOTEK
 1384 Poinsettia Avenue, Suite A
 Vista, California 92081



THE ACCESS TO THE PROPOSED MAINTENANCE AND INSPECTION THE PROPOSED DRIVEWAY AND

APPENDIX A

BORING LOGS

PERCOLATION TEST DATA

A - FIELD TESTING AND SAMPLING PROCEDURES

Bulk Samples (Large)

These samples are normally large bags of earth materials over 20 pounds in weight collected from the field by means of hand digging or exploratory cuttings.

Bulk Samples (Small)

These are plastic bag samples which are normally airtight and contain less than 5 pounds in weight of earth materials collected from the field by means of hand digging or exploratory cuttings. These samples are primarily used for determining natural moisture content and classification indices.

B – EXPLORATORY LOG LEGEND

The following abbreviations and symbols often appear in the classification and description of soil and rock on the logs of borings:

SOILS

USCS Unified Soil Classification System

f-c Fine to coarse

f-m Fine to medium

GEOLOGIC

B: Attitudes Bedding: strike/dip

J: Attitudes Joint: strike/dip

C: Contact line

..... Dashed line denotes USCS material change

———— Solid Line denotes unit / formational change

———— Thick solid line denotes end of the boring

(Additional denotations and symbols are provided on the log of Explorations)

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

CLIENT: St. John Ukranian Catholic Church	DRILLER: Luna Construction	LOGGED BY: MSB
PROJECT NAME: St. John Ukranian Catholic Church	DRILL METHOD: 2' Bucket	OPERATOR: Sal
PROJECT NO.: 3721-SD	HAMMER:	RIG TYPE: Mini Excavator
LOCATION: Santee, Ca	ELEVATION: 334'	DATE: 8/9/2021

Depth (ft)	SAMPLES			USCS Symbol	TRENCH NO.: TP-1 MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing			
	Sample Type	Blows/6 in	Sample Number			Water Content (%)	Dry Density (pcf)		Others
2.5	S-1		SC	Quaternary Old Alluvium (Qoa) Sandy CLAY, dark brown, dry, medium stiff					
3.5	BB-1		SM	Silty coarse SAND, red brown, very moist, medium dense to dense				SR	
4.5	R-1		SM	Silty coarse SAND, few clay, red brown, very moist, medium dense to dense					
7.5			SP	Course SAND, some silt, slight cementation, olive gray, moist, dense to very dense, some cobbles					
8.5			SP	Silty coarse SAND, red brown, very moist, dense to medium dense					
10	HOLE TERMINATED AT 10 FEET								
13	No groundwater encountered Backfilled with soil cuttings								
15									

LEGEND	Sample type:	---Ring	---SPT	---Small Bulk	---Large Bulk	---Water Table
	Lab testing:	AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Anal	RV = R-Value Test	
	SR = Sulfate/Resisitvity Test	SH = Shear Test	CO = Consolida	MD = Maximum Density		

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

CLIENT:	St. John Ukrainian Catholic Church	DRILLER:	Luna Construction	LOGGED BY:	MSB
PROJECT NAME:	St. John Ukrainian Catholic Church	DRILL METHOD:	2' Bucket	OPERATOR:	Sal
PROJECT NO.:	3721-SD	HAMMER:		RIG TYPE:	Mini Excavator
LOCATION:	Santee, Ca	ELEVATION:	334'	DATE:	8/9/2021

Depth (ft)	SAMPLES			USCS Symbol	TRENCH NO.: TP-2 MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows / 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
2.5				SP	Artificial Fill (Af) Gravelly SAND, medium to coarse gravels & cobbles, red brown, dry, loose to medium dense, glass shards 1.5" thick Asphalt / 2" Base			
2.5	X		BB-1	SC	Quaternary Old Alluvium (Qoa) Clayey coarse SAND, red brown, moist to very moist, medium dense, some gravels and cobbles			AL
5				SW	Gravelly SAND with clay, very moist, medium dense			
7.5				SC	Sandy CLAY, dark brown, moist, medium stiff to stiff			
10				SM	Silty SAND, few clay, yellow brown, moist to very moist, medium dense to dense			
10					HOLE TERMINATED AT 10 FEET No groundwater encountered Backfilled with soil cuttings			
13								
15								

LEGEND	Sample type:	---Ring	---SPT	---Small Bulk	---Large Bulk	---Water Table
	Lab testing:	AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Anal	RV = R-Value Test	
	SR = Sulfate/Resistivity Test	SH = Shear Test	CO = Consolida	MD = Maximum Density		

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

CLIENT: St. John Ukranian Catholic Church	DRILLER: Luna Construction	LOGGED BY: MSB
PROJECT NAME: St. John Ukranian Catholic Church	DRILL METHOD: 2' Bucket	OPERATOR: Sal
PROJECT NO.: 3721-SD	HAMMER:	RIG TYPE: Mini Excavator
LOCATION: Santee, Ca	ELEVATION: 334'	DATE: 8/9/2021

Depth (ft)	SAMPLES			USCS Symbol	TRENCH NO.: TP-3 MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows/6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
2.5				SM	Artificial Fill (Af) Silty coarse SAND with gravel, red brown, dry, loose			
				SM	Silty fine SAND, gray, loose, dry			
5			S-1 BB-1	SM	Quaternary Old Alluvium (Qoa) Silty coarse SAND with clay, red brown, moist, medium dense			EI SH, MD
				SM	Silty coarse SAND with gravel, red brown, moist, medium dense, some clay			
				SM	Silty fine to medium SAND, red brown, very moist, dense, few fine gravels			
10	HOLE TERMINATED AT 10 FEET							
	No groundwater encountered Backfilled with soil cuttings							
13								
15								

LEGEND	Sample type:	---Ring	---SPT	---Small Bulk	---Large Bulk	---Water Table
	Lab testing:	AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Anal	RV = R-Value Test	
	SR = Sulfate/Resisitivity Test	SH = Shear Test	CO = Consolida	MD = Maximum Density		

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

CLIENT:	St. John Ukranian Catholic Church	DRILLER:	Luna Construction	LOGGED BY:	MSB
PROJECT NAME:	St. John Ukranian Catholic Church	DRILL METHOD:	2' Bucket	OPERATOR:	Sal
PROJECT NO.:	3721-SD	HAMMER:		RIG TYPE:	Mini Excavator
LOCATION:	Santee, Ca	ELEVATION:	334'	DATE:	8/9/2021

Depth (ft)	SAMPLES			USCS Symbol	TRENCH NO.: TP-4	Laboratory Testing			
	Sample Type	Blows/6 in	Sample Number				Water Content (%)	Dry Density (pcf)	Others
					MATERIAL DESCRIPTION AND COMMENTS				
2.5	X		BB-1	SP	Artificial Fill (Af) Gravelly SAND, red brown, dry, medium dense, many cobbles, asphalt & glass bottle fragments				SA, RV
5	X		BB-2	SM	Quaternary Old Alluvium (Qoa) Silty coarse SAND, red brown, some cobbles, moist, medium dense				
7.5				SC	Sandy CLAY, dark red brown, very moist, soft to slightly stiff				
10				SM	Silty fine SAND, yellow brown, slightly moist, medium dense to dense				
13				SM	Silty fine to medium SAND, yellow brown, moist, dense				
15					HOLE TERMINATED AT 10 FEET				
					No groundwater encountered Backfilled with soil cuttings				

LEGEND	Sample type:	---Ring	---SPT	---Small Bulk	---Large Bulk	---Water Table
	Lab testing:	AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Anal	RV = R-Value Test	
	SR = Sulfate/Resisitivity Test	SH = Shear Test	CO = Consolida	MD = Maximum Density		

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

CLIENT: St. John Ukranian Catholic Church	DRILLER: Luna Construction	LOGGED BY: MSB
PROJECT NAME: St. John Ukranian Catholic Church	DRILL METHOD: 2' Bucket	OPERATOR: Sal
PROJECT NO.: 3721-SD	HAMMER:	RIG TYPE: Mini Excavator
LOCATION: Santee, Ca	ELEVATION: 334'	DATE: 8/9/2021

Depth (ft)	SAMPLES			USCS Symbol	TRENCH NO.: TP-5 MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows/6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
2.5			BB-1	SP	Artificial Fill (Af) Gravelly fine SAND, white to dark brown, dry, loose			
			R-1	SM SM	Quaternary Older Alluvium (Qoa) Silty very fine SAND, yellow brown, slightly moist, medium dense Silty fine to medium SAND, yellow-gray mottled brown, very moist, medium dense to dense			
5			BB-1	ML	Sandy SILT, some very fine to fine sand, red brown, moist to very moist, very dense			
				ML	Sandy Silt, red brown, moist, very dense, refusal			
7.5					HOLE TERMINATED AT 7 FEET Refusal at 7 feet No groundwater encountered Backfilled with soil cuttings			
10								
13								
15								

LEGEND	Sample type:	---Ring	---SPT	---Small Bulk	---Large Bulk	---Water Table
	Lab testing:	AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Anal	RV = R-Value Test	
	SR = Sulfate/Resisitivity Test	SH = Shear Test	CO = Consolida	MD = Maximum Density		

Client: St John the Baptizer Ukranian Catholic Church
Project: St John the Baptizer Ukranian Catholic Church
Project No: 3721-SD
Date: 8/10/2021

Boring No. P-1

Infiltration Rate (Porchet Method)

Time Interval, $\Delta t =$ 30
 Final Depth to Water, $D_F =$ 20.50
 Test Hole Radius, $r =$ 6.00
 Initial Depth to Water, $D_O =$ 20
 Total Test Hole Depth, $D_T =$ 48

Equation -
$$I_t = \frac{\Delta H (60r)}{\Delta t (r+2H_{avg})}$$

$H_O = D_T - D_O =$ 28.00
 $H_F = D_T - D_F =$ 27.50
 $\Delta H = \Delta D = H_O - H_F =$ 0.50
 $H_{avg} = (H_O + H_F) / 2 =$ 27.75

$I_t =$ 0.10 Inches per Hour



Client: St John the Baptizer Ukranian Catholic Church
Project: St John the Baptizer Ukranian Catholic Church
Project No: 3721-SD
Date: 8/10/2021

Boring No. P-2

Infiltration Rate (Porchet Method)

Time Interval, $\Delta t =$ 30
 Final Depth to Water, $D_F =$ 20.25
 Test Hole Radius, $r =$ 6.00
 Initial Depth to Water, $D_O =$ 20
 Total Test Hole Depth, $D_T =$ 53

Equation -
$$I_t = \frac{\Delta H (60r)}{\Delta t (r+2H_{avg})}$$

$H_O = D_T - D_O =$ 33.00
 $H_F = D_T - D_F =$ 32.75
 $\Delta H = \Delta D = H_O - H_F =$ 0.25
 $H_{avg} = (H_O + H_F) / 2 =$ 32.88

$I_t =$ 0.04 Inches per Hour



Categorization of Infiltration Feasibility Condition

Form I-8

Part 1 - Full Infiltration Feasibility Screening Criteria

Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
1	Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X

Provide basis:

Based on a site specific infiltration analysis "Preliminary Geotechnical Evaluation, Proposed Church Building, APN 380-112-08-00, Santee California" project number 3721-SD, dated September 13, 2021 prepared by GeoTek, Inc., the infiltration rate for the proposed basin is 0.10 and 0.04 inches per hour. This does not include the final designed factor of safety.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

2	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		
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Provide basis:

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

Form I-8 Page 3 of 4

Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
5	Do soil and geologic conditions allow for infiltration in any appreciable rate or volume? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	

Provide basis:

Based on a site specific infiltration analysis "Preliminary Geotechnical Evaluation, Proposed Church Building, APN 380-112-08-00, Santee California" project number 3721-SD, dated September 13, 2021 prepared by GeoTek, Inc., the infiltration rate for the proposed basin in 0.10 and 0.04 inches per hour. This does not include the final designed factor of safety.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

6	Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	
---	---	---	--

Provide basis:

Provided the approved geotechnical recommendations are implemented into the design and construction of the stormwater management system, increased risk of geotechnical hazards are null.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

Form I-8 Page 4 of 4

Criteria	Screening Question	Yes	No
7	<p>Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</p>	X	
<p>Provide basis:</p> <p align="center">Yes, see section 5.3.1 Stormwater Infiltration of the project's preliminary soils report</p> <p>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</p>			
8	<p>Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</p>	X	
<p>Provide basis:</p> <p align="center">Yes, see section 5.3.1 Stormwater Infiltration of the project's preliminary soils report</p> <p>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</p>			
Part 2 Result*	<p>If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration.</p> <p>If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.</p>		Partial Infiltration

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by Agency/Jurisdictions to substantiate findings

Factor of Safety and Design Infiltration Rate Worksheet				Form I-9	
Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) $p = w \times v$
A	Suitability Assessment	Soil assessment methods	0.25	2	0.5
		Predominant soil texture	0.25	2	0.5
		Site soil variability	0.25	2	0.5
		Depth to groundwater / impervious layer	0.25	1	0.25
		Suitability Assessment Safety Factor, $S_A = \Sigma p$			
B	Design	Level of pretreatment/ expected sediment loads	0.5	These Sections are To Be Completed by Stormwater Design Engineer	
		Redundancy/resiliency	0.25		
		Compaction during construction	0.25		
		Design Safety Factor, $S_B = \Sigma p$			
Combined Safety Factor, $S_{total} = S_A \times S_B$					
Observed Infiltration Rate, inch/hr, $K_{observed}$ (corrected for test-specific bias)					
Design Infiltration Rate, in/hr, $K_{design} = K_{observed} / S_{total}$					
Supporting Data					
<p>Briefly describe infiltration test and provide reference to test forms:</p> <p>Based on a site specific infiltration analysis by shallow borehole method as discussed in the "Preliminary Geotechnical Evaluation, Proposed Church Building, APN 380-112-08-00, Santee California" project number 3721-SD, dated September 13, 2021 prepared by GeoTek, Inc.</p>					

APPENDIX B

RESULTS OF LABORATORY TESTING

SUMMARY OF LABORATORY TESTING

Identification and Classification

Soils were identified visually in general accordance with the procedures of the Standard Practice for Description and Identification of Soils (ASTM D2488). The soil identifications and classifications are shown on the exploration logs in Appendix A.

Moisture-Density Relationship

Laboratory testing was performed on a soil sample collected during the subsurface exploration. The laboratory maximum dry density and optimum moisture content were determined in general accordance with ASTM D 1557 test procedures. The results of the testing are presented in Appendix B.

Direct Shear

Shear testing was performed on a remolded sample in a direct shear machine of the strain-control type in general accordance with ASTM Test Method D 3080. The rate of deformation is approximately 0.035 inch per minute. The samples were sheared under varying confining loads in order to determine the coulomb shear strength parameters, angle of internal friction and cohesion. The results of the testing are presented in Appendix B.

Expansion Index

Expansion Index testing was performed on a representative site soil sample. Testing was performed in general accordance with ASTM D 4829 test procedures. The results of the testing are presented in Appendix B.

Sulfate Content

The soluble sulfate content of a representative site soil sample was determined by GeoTek's subconsultant, Project X, in general accordance with ASTM D 4327 test procedures. The results of the testing are provided in Appendix B.

Atterberg Limits

Atterberg limits testing were performed on two (2) clayey samples collected from the site. The tests were performed in general accordance with ASTM D 4318. The test results are presented on the exploration logs in Appendix A.

Percent of Soil Passing No 200 Sieve

The amount of soil finer than No. 200 sieve was determined for two clayey samples collected from the site. The tests were performed in general accordance with ASTM D 1140. The test results are presented on the exploration logs in Appendix A.

R-Value

A sample of the subgrade soil was tested for its R-value in general accordance with CAL Test 301. The test result is presented in Appendix B.

Sulfate Content, Resistivity, and Chloride Content

Testing to determine the water-soluble sulfate content was performed by others in general accordance with ASTM D 4327. Resistivity testing was completed by others in general accordance with ASTM G51. Testing to determine the chloride content was performed by other in general accordance with ASTM D 4327. The results are included in Appendix B.



-200 WASH

Date: 8/17/2021
W.O.: 3721-SD sample ID T4- BB1
Client: St. John Ukranian Catholic Church depth 2-3'
Project: St. John Ukranian Catholic Church

Sieve Size	Particle Diameter		Wt. Retained	Wt. Passing	% Passing	Specs
	in.	mm.				
#200	0.0029	0.074	157.41	67.52	30.0%	
Dry Weight	<u>224.93</u>					
Soak Time	<u>144</u> Minutes					



EXPANSION INDEX TEST

(ASTM D4829)

Project Name:	<u>St. John Ukranian Catholic Church</u>	Tested/ Checked By:	BRM Lab No <u>3709</u>
Project Number:	<u>3721-SD</u>	Date Tested:	<u>8/12/2021</u>
Project Location:	<u>Santee, CA</u>	Sample Source:	<u>TP-3</u>
		Sample Description:	<u>Brown Silty Sand</u>

Ring Id: 12 Ring Dia. " : 4" Ring H 1"
 Loading weight: 5516. grams

DENSITY DETERMINATION

A	Weight of compacted sample & ring	785.3
B	Weight of ring	371.2
C	Net weight of sample	414.1
D	Wet Density, lb / ft3 (C*0.3016)	124.9
E	Dry Density, lb / ft3 (D/1.F)	115.5

READINGS			
DATE	TIME	READING	
8/12/2021	11:38	25	Initial
	11:48	25	10 min/Dry
	11:50	25	1 min/Wet
	12:00	26	5 min/Wet
	[-	-	Random
	16:20	26	Final

SATURATION DETERMINATION

	Wet Weight of sample & tare	171.6
	Dry Weight of sample & tare	159
	Tare	4.78
F	Initial Moisture Content, %	8.2
G	(E*F)	943.3
H	(E/167.232)	0.69
I	(1.-H)	0.31
J	(62.4*I)	19.3
K	(G/J)= L % Saturation	48.8

FINAL MOISTURE			
Weight of wet sample & tare	Wt. of dry sample & tare	Tare	% Moisture
356.3	269	12.7	34.1%

EXPANSION INDEX = 1

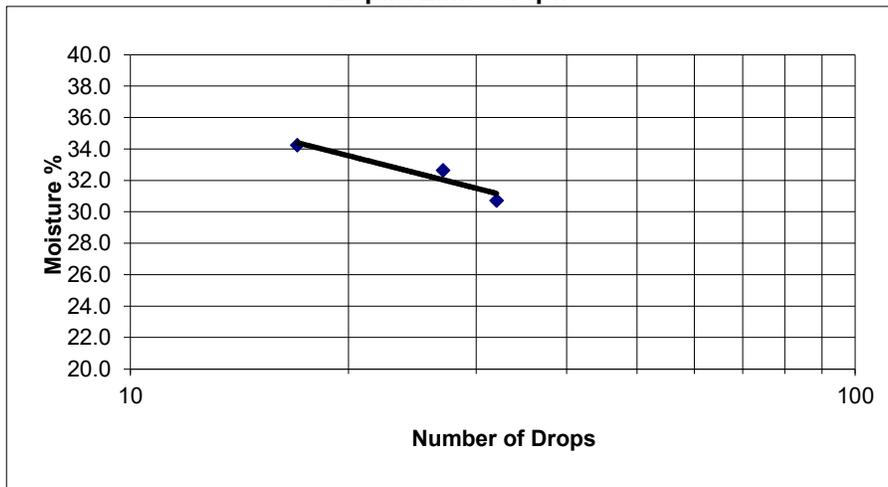


ATTERBERG LIMITS DATA

Field Classification	_____	Job No.	3721-SD
Sample Number	TP-2 BB-1	Client	St. John Ukranian Catholic Church
Sample Type	_____	Project	St. John Ukranian Catholic Church
Location	TP-2 BB-1		
Tested by:	CH		

	Plastic Limit		Liquid Limit		
	1	2	1	2	3
Number of Blows			32	27	17
Determination	1	2	1	2	3
Dish					
Wt. of Dish + Wet Soil	11.40	12.30	46.40	36.90	39.70
Wt. of Dish + Dry Soil	10.10	10.90	36.60	29.00	30.80
Wt. of Moisture	1.30	1.40	9.80	7.90	8.90
Wt. of Dish	4.80	4.80	4.70	4.80	4.80
Wt. of Dry Soil	5.30	6.10	31.90	24.20	26.00
Moisture Content %	24.5	23.0	30.7	32.6	34.2

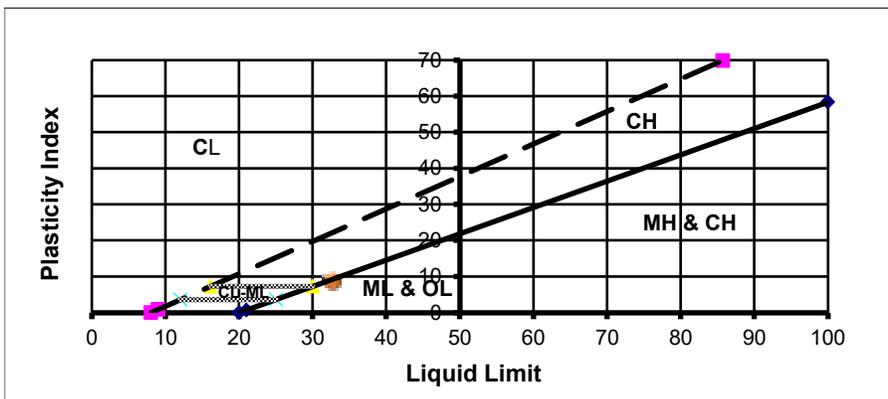
Liquid Limit Graph



Liquid Limit
33

Plastic Limit
24

Plasticity Index
9





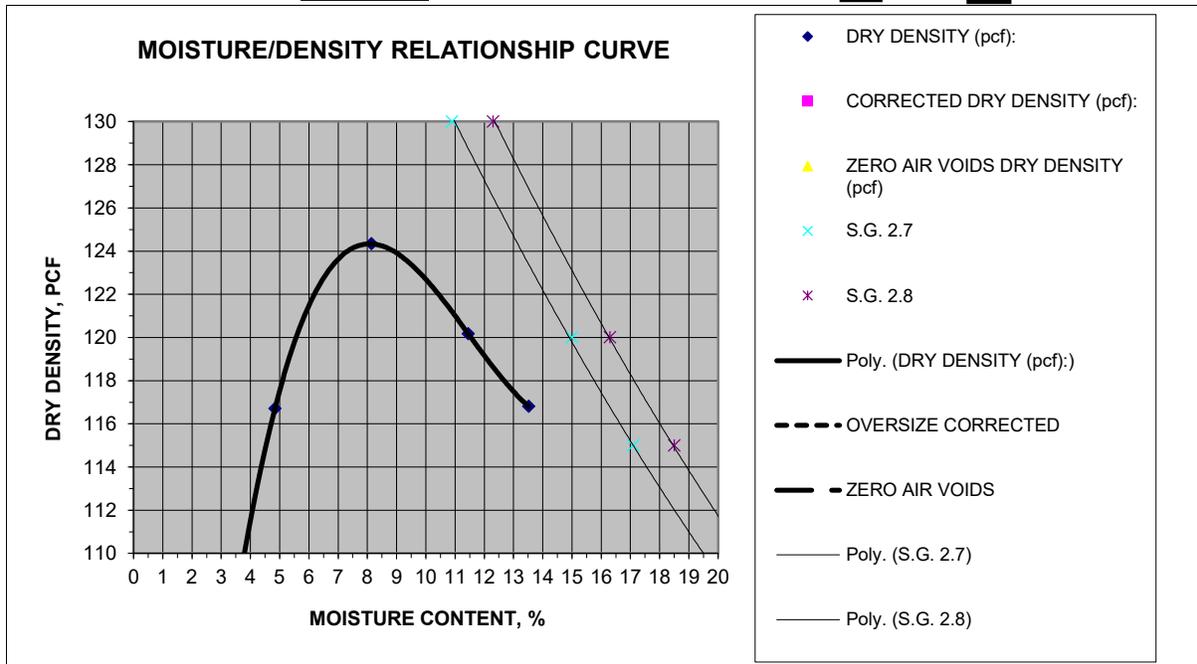
MOISTURE/DENSITY RELATIONSHIP

Client: St. John Ukranian Catholic Church
Project: St. John Ukranian Catholic Church
Location: Santee, CA
Material Type: Brown Silty Sand w/ Clay
Material Supplier: -
Material Source: Test Pit 3 (BB-1)
Sample Location: Northwest portion of site
 -
Sampled By: MSB
Received By: BRM
Tested By: BRM
Reviewed By: -

Job No.: 3721-SD
Lab No.: 3709

Date Sampled: 8/9/2021
Date Received: 8/9/2021
Date Tested: 8/12/2021
Date Reviewed: -

Test Procedure: ASTM D1557 **Method:** A
Oversized Material (%): 0.0 **Correction Required:** yes no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf **@ Optimum Moisture, %**
Corrected Maximum Dry Density, pcf **@ Optimum Moisture, %**

MATERIAL DESCRIPTION

Grain Size Distribution:

	% Gravel (retained on No. 4)
	% Sand (Passing No. 4, Retained on No. 200)
	% Silt and Clay (Passing No. 200)

Atterberg Limits:

	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %

Classification:

Unified Soils Classification: _____
 AASHTO Soils Classification: _____

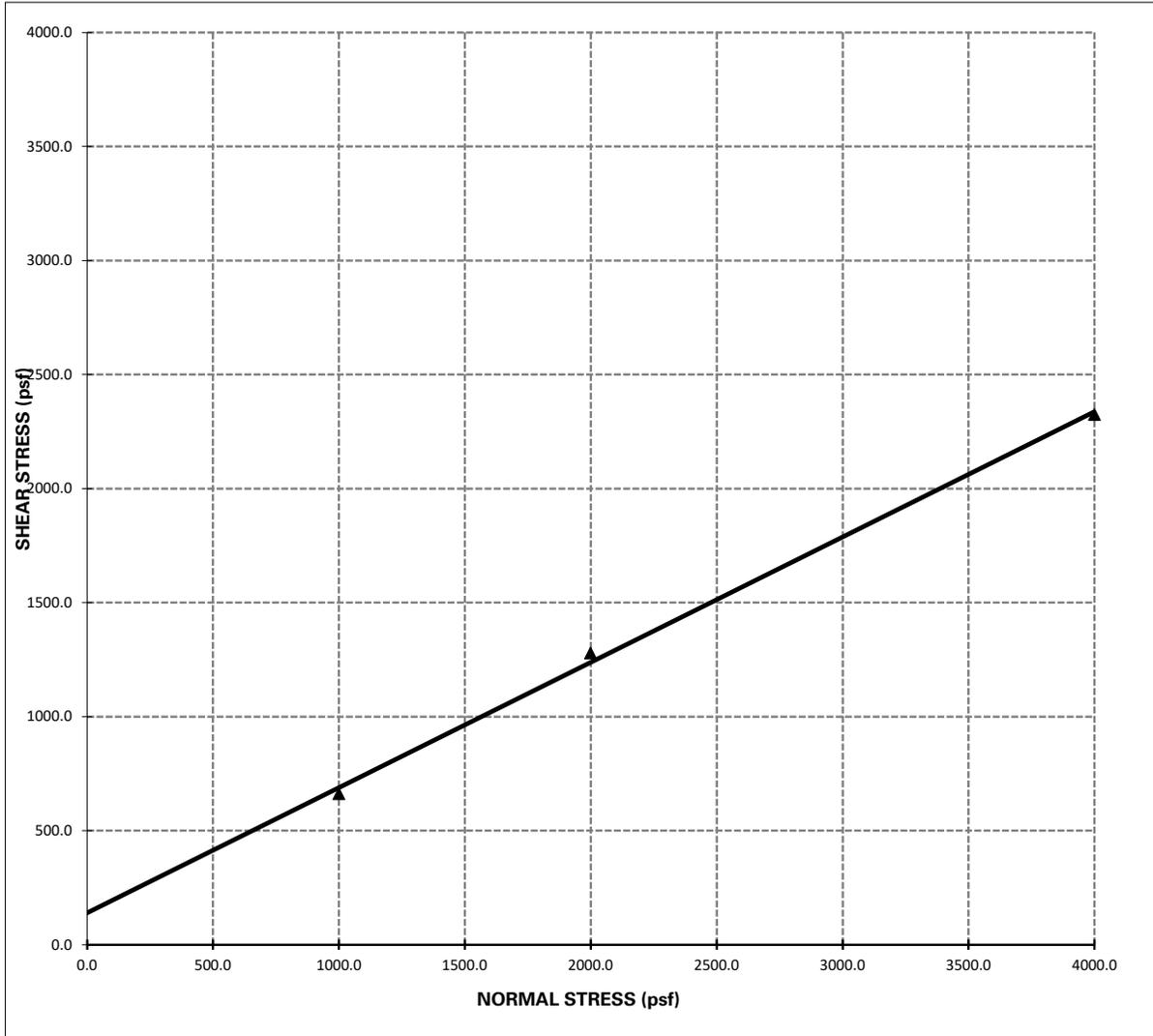


DIRECT SHEAR TEST

Project Name: St. John Ukranian Catholic Church
Project Number: 3721-SD

Sample Location: TP-3 BB-1 @ 3-5'
Date Tested: 8/25/2021

PEAK VALUE



Shear Strength: $\Phi = 29^\circ$; **C = 139 psf**

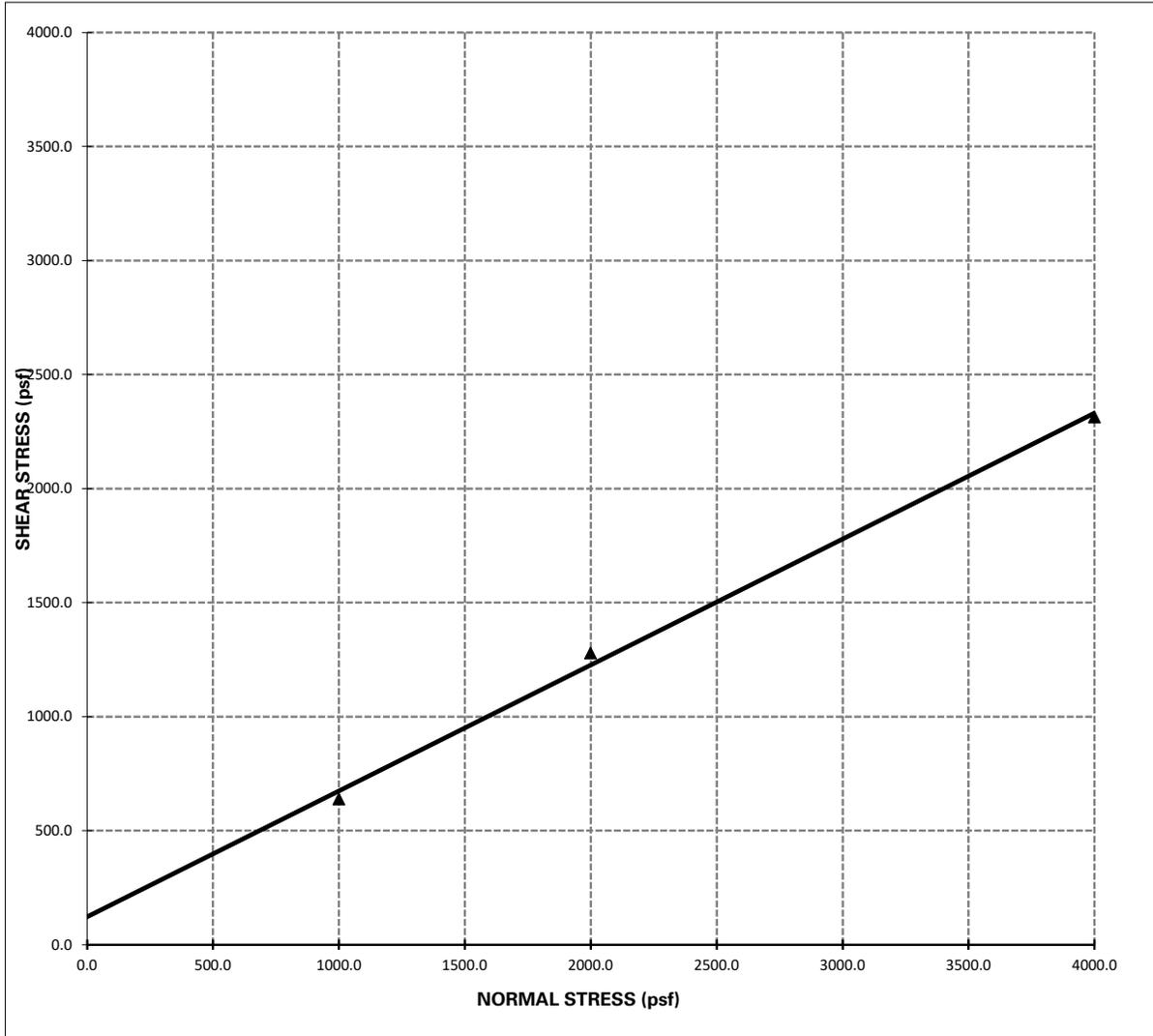
- Notes:**
- 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.
 - 2 - The above reflect direct shear strength at saturated conditions.
 - 3 - The tests were run at a shear rate of 0.035 in/min.



DIRECT SHEAR TEST

Project Name: St. John Ukrainian Catholic Church
Project Number: 3721-SD

Sample Location: TP-3 BB-1 @ 3-5'
Date Tested: 8/25/2021



Shear Strength: $\Phi = 29^\circ$; **C = 122 psf**

- Notes:**
- 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.
 - 2 - The above reflect direct shear strength at saturated conditions.
 - 3 - The tests were run at a shear rate of 0.035 in/min.

August 30, 2021

Mr. Chris Livesey

GeoTek Inc.

1384 Poinsettia Avenue Suite A

Vista, CA 92081-8505

Project No. 47556

Dear Mr. Livesey:

Laboratory testing of the bulk soil sample delivered to our laboratory on 8/26/2021 has been completed.

Reference: W.O. # 3721-SD
Project: St. John Ukaranian Catholic Church
Sample: TP-4 BB-1 @ 2'-3'

Data sheets and graphical presentations are transmitted herewith for your use and information. Any untested portion of the samples will be retained for a period of sixty (60) days prior to disposal. The opportunity to be of service is appreciated, and should you have any questions, kindly call.

Very truly yours,



Steven R. Marvin
RCE 30659

SRM:tw
Enclosures



R - VALUE DATA SHEET

PROJECT No. 47556

DATE: 8/30/2021

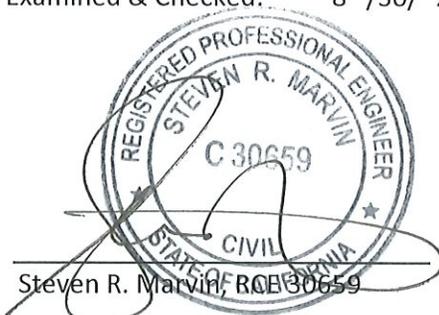
BORING NO. TP-4 BB-1 @ 2'-3'
St. John Ukaranian Catholic Church
W.O.# 3721-SD

SAMPLE DESCRIPTION: Brown Gravelly Silty Sand

R-VALUE TESTING DATA CA TEST 301			
	SPECIMEN ID		
	a	b	c
Mold ID Number	1	2	3
Water added, grams	84	52	64
Initial Test Water, %	14.2	11.1	12.2
Compact Gage Pressure, psi	40	150	80
Exudation Pressure, psi	207	695	354
Height Sample, Inches	2.67	2.49	2.55
Gross Weight Mold, grams	3120	3081	3101
Tare Weight Mold, grams	1954	1946	1958
Sample Wet Weight, grams	1166	1135	1143
Expansion, Inches x 10exp-4	9	54	33
Stability 2,000 lbs (160psi)	53 / 130	28 / 61	33 / 80
Turns Displacement	4.13	3.41	3.64
R-Value Uncorrected	12	54	41
R-Value Corrected	13	54	41
Dry Density, pcf	115.9	124.3	121.0

DESIGN CALCULATION DATA

Traffic Index	Assumed:	4.0	4.0	4.0
G.E. by Stability		0.89	0.47	0.60
G. E. by Expansion		0.30	1.80	1.10

Equilibrium R-Value	32 by EXPANSION	Examined & Checked: <u>8 /30/ 21</u>
REMARKS:	Gf = <u>1.25</u>	 Steven R. Marvin, RCB30659
	<u>14.2% Retained on the</u> <u>3/4" Sieve.</u>	

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.



R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 47556

DATE: 8 /30/ 2021

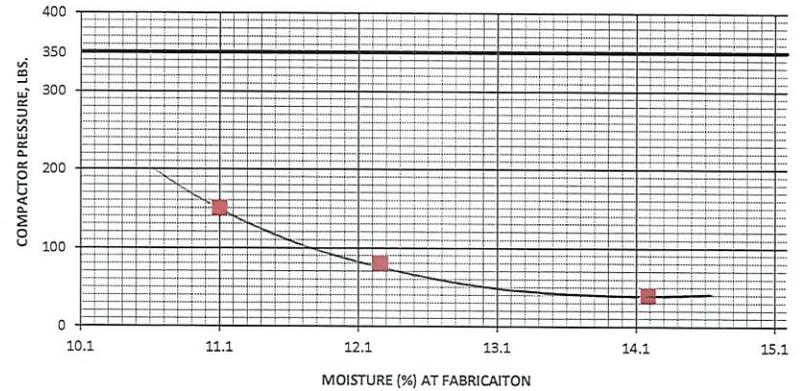
REMARKS: _____

BORING NO. TP-4 BB-1 @ 2'-3'

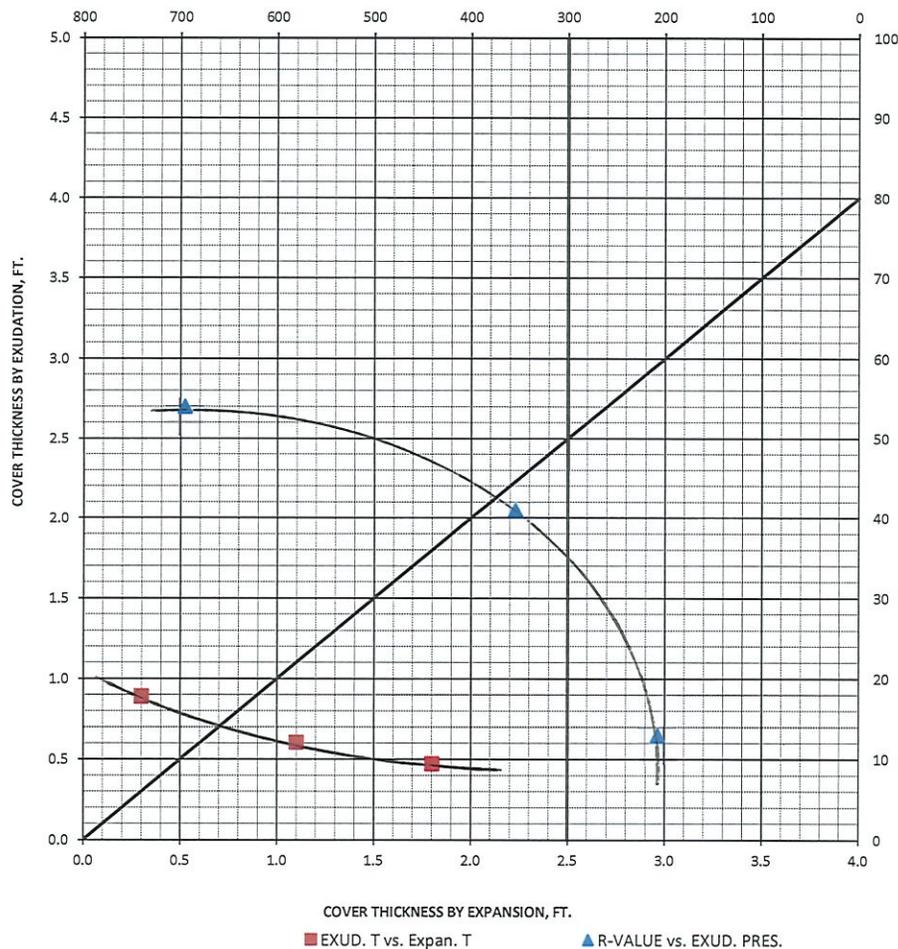
St. John Ukaranian Catholic Church

W.O.# 3721-SD

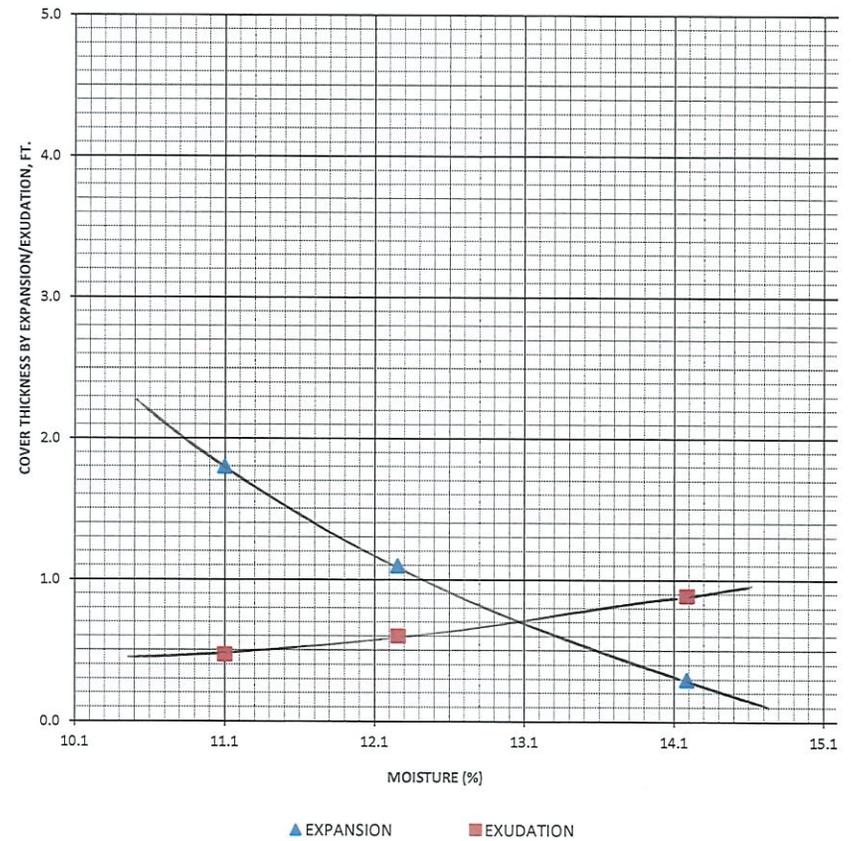
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %





Results Only Soil Testing for St John the Baptizer Ukranian Catholic Church

August 17, 2021

Prepared for:

Chris Livesey

GeoTek, Inc.

1384 Poinsettia Ave, Suite A

Vista, CA, 92081

clivesey@geotekusa.com

Project X Job#: S210813F

Client Job or PO#: 3721-SD

Respectfully Submitted,

Eduardo Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
Professional Engineer
California No. M37102
ehernandez@projectxcorrosion.com





Soil Analysis Lab Results

Client: GeoTek, Inc.
 Job Name: St John the Baptizer Ukranian Catholic Church
 Client Job Number: 3721-SD
 Project X Job Number: S210813F
 August 17, 2021

Bore# / Description	Method Depth	ASTM D4327 Sulfates		ASTM D4327 Chlorides		ASTM G187 Resistivity		ASTM D4972 pH	ASTM G200 Redox	ASTM D4658 Sulfide	ASTM D4327 Nitrate	ASTM D6919 Ammonium	ASTM D6919 Lithium	ASTM D6919 Sodium	ASTM D6919 Potassium	ASTM D6919 Magnesium	ASTM D6919 Calcium	ASTM D4327 Fluoride	ASTM D4327 Phosphate
		SO ₄ ²⁻ (mg/kg)	(wt%)	Cl ⁻ (mg/kg)	(wt%)	As Rec'd	Minimum												
T-1, BB-1 Red-Brown Silty Coarse Sand	2-4	2,215.8	0.2216	881.5	0.0882	1,943	308	7.4	195	<0.01	1.1	19.6	0.06	730.5	1.4	207.8	612.0	4.7	0.1

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
 mg/kg = milligrams per kilogram (parts per million) of dry soil weight
 ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
 Chemical Analysis performed on 1:3 Soil-To-Water extract
 PPM = mg/kg (soil) = mg/L (Liquid)

APPENDIX C

GENERAL EARTHWORK GRADING GUIDELINES

GENERAL GRADING GUIDELINES

Guidelines presented herein are intended to address general construction procedures for earthwork construction. Specific situations and conditions often arise which cannot reasonably be discussed in general guidelines, when anticipated these are discussed in the text of the report. Often unanticipated conditions are encountered which may necessitate modification or changes to these guidelines. It is our hope that these will assist the contractor to more efficiently complete the project by providing a reasonable understanding of the procedures that would be expected during earthwork and the testing and observation used to evaluate those procedures.

General

Grading should be performed to at least the minimum requirements of governing agencies, the California Building Code, CBC (2019) and the guidelines presented below.

Preconstruction Meeting

A preconstruction meeting should be held prior to site earthwork. Any questions the contractor has regarding our recommendations, general site conditions, apparent discrepancies between reported and actual conditions and/or differences in procedures the contractor intends to use should be brought up at that meeting. The contractor (including the main onsite representative) should review our report and these guidelines in advance of the meeting. Any comments the contractor may have regarding these guidelines should be brought up at that meeting.

Grading Observation and Testing

1. Observation of the fill placement should be provided by our representative during grading. Verbal communication during the course of each day will be used to inform the contractor of test results. The contractor should receive a copy of the "Daily Field Report" indicating results of field density tests that day. If our representative does not provide the contractor with these reports, our office should be notified.
2. Testing and observation procedures are, by their nature, specific to the work or area observed and location of the tests taken, variability may occur in other locations. The contractor is responsible for the uniformity of the grading operations; our observations and test results are intended to evaluate the contractor's overall level of efforts during grading. The contractor's personnel are the only individuals participating in all aspect of site work. Compaction testing and observation should not be considered as relieving the contractor's responsibility to properly compact the fill.
3. Cleanouts, processed ground to receive fill, key excavations, and subdrains should be observed by our representative prior to placing any fill. It will be the contractor's responsibility to notify our representative or office when such areas are ready for observation.
4. Density tests may be made on the surface material to receive fill, as considered warranted by this firm.
5. In general, density tests would be made at maximum intervals of two feet of fill height or every 1,000 cubic yards of fill placed. Criteria will vary depending on soil conditions and size of the fill. More frequent testing may be performed. In any case, an adequate number of field density tests should be made to evaluate the required compaction and moisture content is generally being obtained.

6. Laboratory testing to support field test procedures will be performed, as considered warranted, based on conditions encountered (e.g. change of material sources, types, etc.) Every effort will be made to process samples in the laboratory as quickly as possible and in progress construction projects are our first priority. However, laboratory workloads may cause in delays and some soils may require a **minimum of 48 to 72 hours to complete test procedures**. Whenever possible, our representative(s) should be informed in advance of operational changes that might result in different source areas for materials.
7. Procedures for testing of fill slopes are as follows:
 - a) Density tests should be taken periodically during grading on the flat surface of the fill, three to five feet horizontally from the face of the slope.
 - b) If a method other than over building and cutting back to the compacted core is to be employed, slope compaction testing during construction should include testing the outer six inches to three feet in the slope face to determine if the required compaction is being achieved.
8. Finish grade testing of slopes and pad surfaces should be performed after construction is complete.

Site Clearing

1. All vegetation, and other deleterious materials, should be removed from the site. If material is not immediately removed from the site it should be stockpiled in a designated area(s) well outside of all current work areas and delineated with flagging or other means. Site clearing should be performed in advance of any grading in a specific area.
2. Efforts should be made by the contractor to remove all organic or other deleterious material from the fill, as even the most diligent efforts may result in the incorporation of some materials. This is especially important when grading is occurring near the natural grade. All equipment operators should be aware of these efforts. Laborers may be required as root pickers.
3. Nonorganic debris or concrete may be placed in deeper fill areas provided the procedures used are observed and found acceptable by our representative. Typical procedures are similar to those indicated on Plate G-4.

Treatment of Existing Ground

1. Following site clearing, all surficial deposits of alluvium and colluvium as well as weathered or creep effected bedrock, should be removed (see Plates G-1, G-2 and G-3) unless otherwise specifically indicated in the text of this report.
2. In some cases, removal may be recommended to a specified depth (e.g. flat sites where partial alluvial removals may be sufficient). The contractor should not exceed these depths unless directed otherwise by our representative.
3. Groundwater existing in alluvial areas may make excavation difficult. Deeper removals than indicated in the text of the report may be necessary due to saturation during winter months.
4. Subsequent to removals, the natural ground should be processed to a depth of six inches, moistened to near optimum moisture conditions and compacted to fill standards.
5. Exploratory back hoe or dozer trenches still remaining after site removal should be excavated and filled with compacted fill if they can be located.

Subdrainage

1. Subdrainage systems should be provided in canyon bottoms prior to placing fill, and behind buttress and stabilization fills and in other areas indicated in the report. Subdrains should conform to schematic diagrams G-1 and G-5, and be acceptable to our representative.
2. For canyon subdrains, runs less than 500 feet may use six-inch pipe. Typically, runs in excess of 500 feet should have the lower end as eight-inch minimum.
3. Filter material should be clean, 1/2 to 1-inch gravel wrapped in a suitable filter fabric. Class 2 permeable filter material per California Department of Transportation Standards tested by this office to verify its suitability, may be used without filter fabric. A sample of the material should be provided to the Soils Engineer by the contractor at least two working days before it is delivered to the site. The filter should be clean with a wide range of sizes.
4. Approximate delineation of anticipated subdrain locations may be offered at 40-scale plan review stage. During grading, this office would evaluate the necessity of placing additional drains.
5. All subdrainage systems should be observed by our representative during construction and prior to covering with compacted fill.
6. Subdrains should outlet into storm drains where possible. Outlets should be located and protected. The need for backflow preventers should be assessed during construction.
7. Consideration should be given to having subdrains located by the project surveyors.

Fill Placement

1. Unless otherwise indicated, all site soil and bedrock may be reused for compacted fill; however, some special processing or handling may be required (see text of report).
2. Material used in the compacting process should be evenly spread, moisture conditioned, processed, and compacted in thin lifts six (6) to eight (8) inches in compacted thickness to obtain a uniformly dense layer. The fill should be placed and compacted on a nearly horizontal plane, unless otherwise found acceptable by our representative.
3. If the moisture content or relative density varies from that recommended by this firm, the contractor should rework the fill until it is in accordance with the following:
 - a) Moisture content of the fill should be at or above optimum moisture. Moisture should be evenly distributed without wet and dry pockets. Pre-watering of cut or removal areas should be considered in addition to watering during fill placement, particularly in clay or dry surficial soils. The ability of the contractor to obtain the proper moisture content will control production rates.
 - b) Each six-inch layer should be compacted to at least 90 percent of the maximum dry density in compliance with the testing method specified by the controlling governmental agency. In most cases, the testing method is ASTM Test Designation D 1557.
4. Rock fragments less than eight inches in diameter may be utilized in the fill, provided:
 - a) They are not placed in concentrated pockets;
 - b) There is a sufficient percentage of fine-grained material to surround the rocks;
 - c) The distribution of the rocks is observed by, and acceptable to, our representative.
5. Rocks exceeding eight (8) inches in diameter should be taken off site, broken into smaller fragments, or placed in accordance with recommendations of this firm in areas designated suitable for rock disposal (see Plate G-4). On projects where significant large quantities of oversized materials are anticipated, alternate guidelines for placement may be included. If

significant oversized materials are encountered during construction, these guidelines should be requested.

6. In clay soil, dry or large chunks or blocks are common. If in excess of eight (8) inches minimum dimension, then they are considered as oversized. Sheepsfoot compactors or other suitable methods should be used to break up blocks. When dry, they should be moisture conditioned to provide a uniform condition with the surrounding fill.

Slope Construction

1. The contractor should obtain a minimum relative compaction of 90 percent out to the finished slope face of fill slopes. This may be achieved by either overbuilding the slope and cutting back to the compacted core, or by direct compaction of the slope face with suitable equipment.
2. Slopes trimmed to the compacted core should be overbuilt by at least three (3) feet with compaction efforts out to the edge of the false slope. Failure to properly compact the outer edge results in trimming not exposing the compacted core and additional compaction after trimming may be necessary.
3. If fill slopes are built "at grade" using direct compaction methods, then the slope construction should be performed so that a constant gradient is maintained throughout construction. Soil should not be "spilled" over the slope face nor should slopes be "pushed out" to obtain grades. Compaction equipment should compact each lift along the immediate top of slope. Slopes should be back rolled or otherwise compacted at approximately every 4 feet vertically as the slope is built.
4. Corners and bends in slopes should have special attention during construction as these are the most difficult areas to obtain proper compaction.
5. Cut slopes should be cut to the finished surface. Excessive undercutting and smoothing of the face with fill may necessitate stabilization.

Keyways, Buttress and Stabilization Fills

Keyways are needed to provide support for fill slope and various corrective procedures.

1. Side-hill fills should have an equipment-width key at their toe excavated through all surficial soil and into competent material and tilted back into the hill (Plates G-2, G-3). As the fill is elevated, it should be benched through surficial soil and slopewash, and into competent bedrock or other material deemed suitable by our representatives (See Plates G-1, G-2, and G-3).
2. Fill over cut slopes should be constructed in the following manner:
 - a) All surficial soils and weathered rock materials should be removed at the cut-fill interface.
 - b) A key at least one and one-half (1.5) equipment width wide (or as needed for compaction), and tipped at least one (1) foot into slope, should be excavated into competent materials and observed by our representative.
 - c) The cut portion of the slope should be excavated prior to fill placement to evaluate if stabilization is necessary. The contractor should be responsible for any additional earthwork created by placing fill prior to cut excavation. (see Plate G-3 for schematic details.)
3. Daylight cut lots above descending natural slopes may require removal and replacement of the outer portion of the lot. A schematic diagram for this condition is presented on Plate G-2.

4. A basal key is needed for fill slopes extending over natural slopes. A schematic diagram for this condition is presented on Plate G-2.
5. All fill slopes should be provided with a key unless within the body of a larger overall fill mass. Please refer to Plate G-3 for specific guidelines.

Anticipated buttress and stabilization fills are discussed in the text of the report. The need to stabilize other proposed cut slopes will be evaluated during construction. Plate G-5 shows a schematic of buttress construction.

1. All backcuts should be excavated at gradients of 1:1 or flatter. The backcut configuration should be determined based on the design, exposed conditions, and need to maintain a minimum fill width and provide working room for the equipment.
2. On longer slopes, backcuts and keyways should be excavated in maximum 250 feet long segments. The specific configurations will be determined during construction.
3. All keys should be a minimum of two (2) feet deep at the toe and slope toward the heel at least one foot or two (2%) percent, whichever is greater.
4. Subdrains are to be placed for all stabilization slopes exceeding 10 feet in height. Lower slopes are subject to review. Drains may be required. Guidelines for subdrains are presented on Plate G-5.
5. Benching of backcuts during fill placement is required.

Lot Capping

1. When practical, the upper three (3) feet of material placed below finish grade should be comprised of the least expansive material available. Preferably, highly and very highly expansive materials should not be used. We will attempt to offer advice based on visual evaluations of the materials during grading, but it must be realized that laboratory testing is needed to evaluate the expansive potential of soil. Minimally, this testing takes two (2) to four (4) days to complete.
2. Transition lots (cut and fill) both per plan and those created by remedial grading (e.g. lots above stabilization fills, along daylight lines, above natural slopes, etc.) should be capped with a minimum three foot thick compacted fill blanket.
3. Cut pads should be observed by our representative(s) to evaluate the need for overexcavation and replacement with fill. This may be necessary to reduce water infiltration into highly fractured bedrock or other permeable zones, and/or due to differing expansive potential of materials beneath a structure. The overexcavation should be at least three feet. Deeper overexcavation may be recommended in some cases.

ROCK PLACEMENT AND ROCK FILL GUIDELINES

If large quantities of oversize material would be generated during grading, it's likely that such materials may require special handling for burial. Although alternatives may be developed in the field, the following methods of rock disposal are recommended on a preliminary basis.

Limited Larger Rock

When materials encountered are principally soil with limited quantities of larger rock fragments or boulders, placement in windrows is recommended. The following procedures should be applied:

1. Oversize rock (greater than 8 inches) should be placed in windrows.
 - a) Windrows are rows of single file rocks placed to avoid nesting or clusters of rock.

- b) Each adjacent rock should be approximately the same size (within ~one foot in diameter).
- c) The maximum rock size allowed in windrows is four feet
2. A minimum vertical distance of three feet between lifts should be maintained. Also, the windrows should be offset from lift to lift. Rock windrows should not be closer than 15 feet to the face of fill slopes and sufficient space must be maintained for proper slope construction (see Plate G-4).
3. Rocks greater than eight inches in diameter should not be placed within seven feet of the finished subgrade for a roadway or pads and should be held below the depth of the lowest utility. This will allow easier trenching for utility lines.
4. Rocks greater than four feet in diameter should be broken down, if possible, or they may be placed in a dozer trench. Each trench should be excavated into the compacted fill a minimum of one foot deeper than the largest diameter of rock.
 - a) The rock should be placed in the trench and granular fill materials (SE>30) should be flooded into the trench to fill voids around the rock.
 - b) The over size rock trenches should be no closer together than 15 feet from any slope face.
 - c) Trenches at higher elevation should be staggered and there should be a minimum of four feet of compacted fill between the top of the one trench and the bottom of the next higher trench.
 - d) It would be necessary to verify 90 percent relative compaction in these pits. A 24 to 72 hour delay to allow for water dissipation should be anticipated prior to additional fill placement.

Structural Rock Fills

If the materials generated for placement in structural fills contains a significant percentage of material more than six (6) inches in one dimension, then placement using conventional soil fill methods with isolated windrows would not be feasible. In such cases the following could be considered:

1. Mixes of large rock or boulders may be placed as rock fill. They should be below the depth of all utilities both on pads and in roadways and below any proposed swimming pools or other excavations. If these fills are placed within seven (7) feet of finished grade, they may affect foundation design.
2. Rock fills are required to be placed in horizontal layers that should **not exceed two feet in thickness, or the maximum rock size present, which ever is less**. All rocks exceeding two feet should be broken down to a smaller size, windrowed (see above), or disposed of in non-structural fill areas. Localized larger rock up to 3 feet in largest dimension may be placed in rock fill as follows:
 - a) individual rocks are placed in a given lift so as to be roughly 50% exposed above the typical surface of the fill ,
 - b) loaded rock trucks or alternate compactors are worked around the rock on all sides to the satisfaction of the soil engineer,
 - c) the portion of the rock above grade is covered with a second lift.
3. Material placed in each lift should be well graded. No unfilled spaces (voids) should be permitted in the rock fill.

Compaction Procedures

Compaction of rock fills is largely procedural. The following procedures have been found to generally produce satisfactory compaction.

1. Provisions for routing of construction traffic over the fill should be implemented.
 - a) Placement should be by rock trucks crossing the lift being placed and dumping at its edge.
 - b) The trucks should be routed so that each pass across the fill is via a different path and that all areas are uniformly traversed.
 - c) The dumped piles should be knocked down and spread by a large dozer (D-8 or larger suggested). (Water should be applied before and during spreading.)
2. Rock fill should be generously watered (sluiced)
 - a) Water should be applied by water trucks to the:
 - i) dump piles,
 - ii) front face of the lift being placed and,
 - iii) surface of the fill prior to compaction.
 - b) No material should be placed without adequate water.
 - c) The number of water trucks and water supply should be sufficient to provide constant water.
 - d) Rock fill placement should be suspended when water trucks are unavailable:
 - i) for more than 5 minutes straight, or,
 - ii) for more than 10 minutes/hour.
3. In addition to the truck pattern and at the discretion of the soil engineer, large, rubber tired compactors may be required.
 - a) The need for this equipment will depend largely on the ability of the operators to provide complete and uniform coverage by wheel rolling with the trucks.
 - b) Other large compactors will also be considered by the soil engineer provided that required compaction is achieved.
4. Placement and compaction of the rock fill is largely procedural. Observation by trenching should be made to check:
 - a) the general segregation of rock size,
 - b) for any unfilled spaces between the large blocks, and
 - c) the matrix compaction and moisture content.
5. Test fills may be required to evaluate relative compaction of finer grained zones or as deemed appropriate by the soil engineer.
 - a) A lift should be constructed by the methods proposed, as proposed
6. Frequency of the test trenching is to be at the discretion of the soil engineer. Control areas may be used to evaluate the contractor's procedures.
7. A minimum horizontal distance of 15 feet should be maintained from the face of the rock fill and any finish slope face. At least the outer 15 feet should be built of conventional fill materials.

Piping Potential and Filter Blankets

Where conventional fill is placed over rock fill, the potential for piping (migration) of the fine grained material from the conventional fill into rock fills will need to be addressed.

The potential for particle migration is related to the grain size comparisons of the materials present and in contact with each other. Provided that 15 percent of the finer soil is larger than the effective

pore size of the coarse soil, then particle migration is substantially mitigated. This can be accomplished with a well-graded matrix material for the rock fill and a zone of fill similar to the matrix above it. The specific gradation of the fill materials placed during grading must be known to evaluate the need for any type of filter that may be necessary to cap the rock fills. This, unfortunately, can only be accurately determined during construction.

In the event that poorly graded matrix is used in the rock fills, properly graded filter blankets 2 to 3 feet thick separating rock fills and conventional fill may be needed. As an alternative, use of two layers of filter fabric (Mirafi 700 x or equivalent) could be employed on top of the rock fill. In order to mitigate excess puncturing, the surface of the rock fill should be well broken down and smoothed prior to placing the filter fabric. The first layer of the fabric may then be placed and covered with relatively permeable fill material (with respect to overlying material) 1 to 2 feet thick. The relative permeable material should be compacted to fill standards. The second layer of fabric should be placed and conventional fill placement continued.

Subdrainage

Rock fill areas should be tied to a subdrainage system. If conventional fill is placed that separates the rock from the main canyon subdrain, then a secondary system should be installed. A system consisting of an adequately graded base (3 to 4 percent to the lower side) with a collector system and outlets may suffice.

Additionally, at approximately every 25 foot vertical interval, a collector system with outlets should be placed at the interface of the rock fill and the conventional fill blanketing a fill slope.

Monitoring

Depending upon the depth of the rock fill and other factors, monitoring for settlement of the fill areas may be needed following completion of grading. Typically, if rock fill depths exceed 40 feet, monitoring would be recommended prior to construction of any settlement sensitive improvements. Delays of 3 to 6 months or longer can be expected prior to the start of construction.

UTILITY TRENCH CONSTRUCTION AND BACKFILL

Utility trench excavation and backfill is the contractor's responsibility. The geotechnical consultant typically provides periodic observation and testing of these operations. While efforts are made to make sufficient observations and tests to verify that the contractors' methods and procedures are adequate to achieve proper compaction, it is typically impractical to observe all backfill procedures. As such, it is critical that the contractor use consistent backfill procedures.

Compaction methods vary for trench compaction and experience indicates many methods can be successful. However, procedures that "worked" on previous projects may or may not prove effective on a given site. The contractor(s) should outline the procedures proposed, so that we may discuss them **prior** to construction. We will offer comments based on our knowledge of site conditions and experience.

1. Utility trench backfill in slopes, structural areas, in streets and beneath flat work or hardscape should be brought to at least optimum moisture and compacted to at least 90 percent of the laboratory standard. Soil should be moisture conditioned prior to placing in the trench.

2. Flooding and jetting are not typically recommended or acceptable for native soils. Flooding or jetting may be used with select sand having a Sand Equivalent (SE) of 30 or higher. This is typically limited to the following uses:
 - a) shallow (12 + inches) under slab interior trenches and,
 - b) as bedding in pipe zone.The water should be allowed to dissipate prior to pouring slabs or completing trench compaction.
3. Care should be taken not to place soils at high moisture content within the upper three feet of the trench backfill in street areas, as overly wet soils may impact subgrade preparation. Moisture may be reduced to 2% below optimum moisture in areas to be paved within the upper three feet below sub grade.
4. Sand backfill should not be allowed in exterior trenches adjacent to and within an area extending below a 1:1 projection from the outside bottom edge of a footing, unless it is similar to the surrounding soil.
5. Trench compaction testing is generally at the discretion of the geotechnical consultant. Testing frequency will be based on trench depth and the contractor's procedures. A probing rod would be used to assess the consistency of compaction between tested areas and untested areas. If zones are found that are considered less compact than other areas, this would be brought to the contractor's attention.

JOB SAFETY

General

Personnel safety is a primary concern on all job sites. The following summaries are safety considerations for use by all our employees on multi-employer construction sites. On ground personnel are at highest risk of injury and possible fatality on grading construction projects. The company recognizes that construction activities will vary on each site and that job site safety is the contractor's responsibility. However, it is, imperative that all personnel be safety conscious to avoid accidents and potential injury.

In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of our field personnel on grading and construction projects.

1. Safety Meetings: Our field personnel are directed to attend the contractor's regularly scheduled safety meetings.
2. Safety Vests: Safety vests are provided for and are to be worn by our personnel while on the job site.
3. Safety Flags: Safety flags are provided to our field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.

In the event that the contractor's representative observes any of our personnel not following the above, we request that it be brought to the attention of our office.

Test Pits Location, Orientation and Clearance

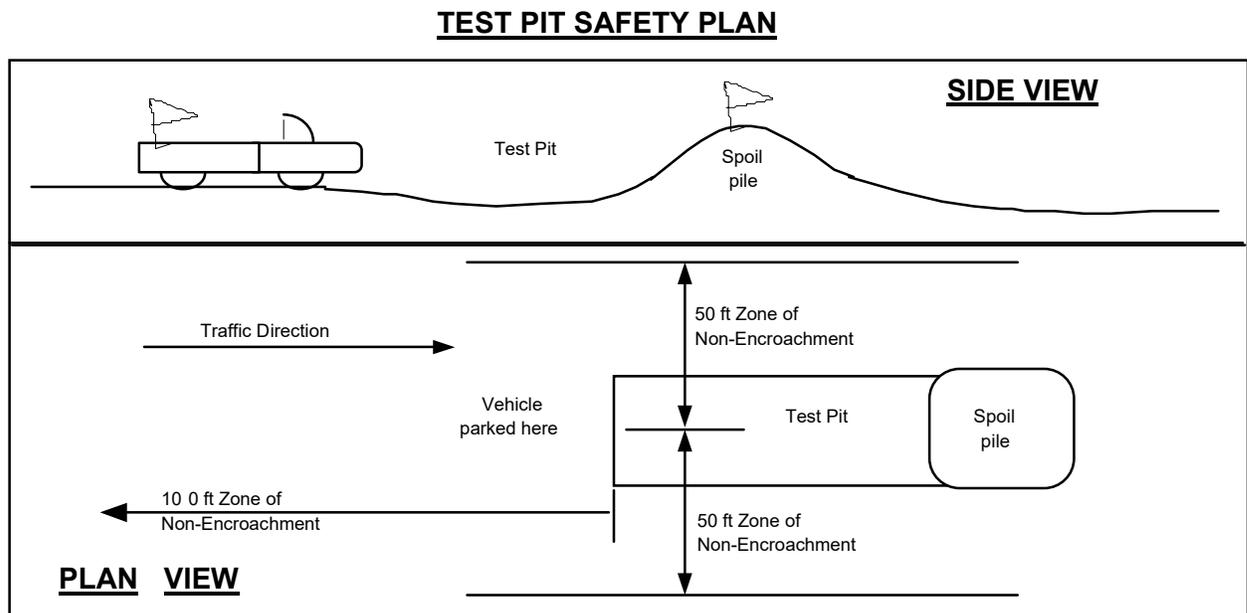
The technician is responsible for selecting test pit locations. The primary concern is the technician's safety. However, it is necessary to take sufficient tests at various locations to obtain a representative sampling of the fill. As such, efforts will be made to coordinate locations with the grading contractors authorized representatives (e.g. dump man, operator, supervisor, grade checker, etc.),



and to select locations following or behind the established traffic pattern, preferably outside of current traffic. The contractors authorized representative should direct excavation of the pit and safety during the test period. Again, safety is the paramount concern.

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic. The technician's vehicle is to be placed next to the test pit, opposite the spoil pile. This necessitates that the fill be maintained in a drivable condition. Alternatively, the contractor may opt to park a piece of equipment in front of test pits, particularly in small fill areas or those with limited access.

A zone of non-encroachment should be established for all test pits (see diagram below). No grading equipment should enter this zone during the test procedure. The zone should extend outward to the sides approximately 50 feet from the center of the test pit and 100 feet in the direction of traffic flow. This zone is established both for safety and to avoid excessive ground vibration, which typically decreases test results.



Slope Tests

When taking slope tests, the technician should park their vehicle directly above or below the test location on the slope. The contractor's representative should effectively keep all equipment at a safe operation distance (e.g. 50 feet) away from the slope during testing.

The technician is directed to withdraw from the active portion of the fill as soon as possible following testing. The technician's vehicle should be parked at the perimeter of the fill in a highly visible location.

Trench Safety

It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Trenches for all utilities should be excavated in accordance with CAL-OSHA and any other applicable safety standards. Safe conditions will be required to enable compaction testing of the trench backfill.

All utility trench excavations in excess of 5 feet deep, which a person enters, are to be shored or laid back. Trench access should be provided in accordance with OSHA standards. Our personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

Our personnel are directed not to enter any excavation which;

1. is 5 feet or deeper unless shored or laid back,
2. exit points or ladders are not provided,
3. displays any evidence of instability, has any loose rock or other debris which could fall into the trench, or
4. displays any other evidence of any unsafe conditions regardless of depth.

If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraws and notifies their supervisor. The contractor's representative will then be contacted in an effort to affect a solution. All backfill not tested due to safety concerns or other reasons is subject to reprocessing and/or removal.

Procedures

In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is directed to inform both the developer's and contractor's representatives. If the condition is not rectified, the technician is required, by company policy, to immediately withdraw and notify their supervisor. The contractor's representative will then be contacted in an effort to affect a solution. No further testing will be performed until the situation is rectified. Any fill placed in the interim can be considered unacceptable and subject to reprocessing, recompaction or removal.

In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to technician's attention and notify our project manager or office. Effective communication and coordination between the contractors' representative and the field technician(s) is strongly encouraged in order to implement the above safety program and safety in general.

The safety procedures outlined above should be discussed at the contractor's safety meetings. This will serve to inform and remind equipment operators of these safety procedures particularly the zone of non-encroachment.

The safety procedures outlined above should be discussed at the contractor's safety meetings. This will serve to inform and remind equipment operators of these safety procedures particularly the zone of non-encroachment.

ALTERNATES

Finish Grade

Original Ground

Loose Surface Materials

Suitable Material

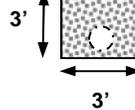
4 feet typical

Construct Benches where slope exceeds 5:1

Slope to Drain

Suitable Material

Bottom of Cleanout to Be At Least 1.5 Times the Width of Compaction Equipment



6" Perforated Pipe in 9 cubic feet per Lineal Foot Clean Gravel Wrapped in Filter Fabric

Finish Grade

Original Ground

Loose Surface Materials

Construct Benches where slope exceeds 5:1

Slope to Drain

Suitable Material

4 feet typical

Bottom of Cleanout to Be At Least 1.5 Times the Width of Compaction Equipment

6" Perforated Pipe in 9 cubic feet per Lineal Foot Clean Gravel Wrapped in Filter Fabric

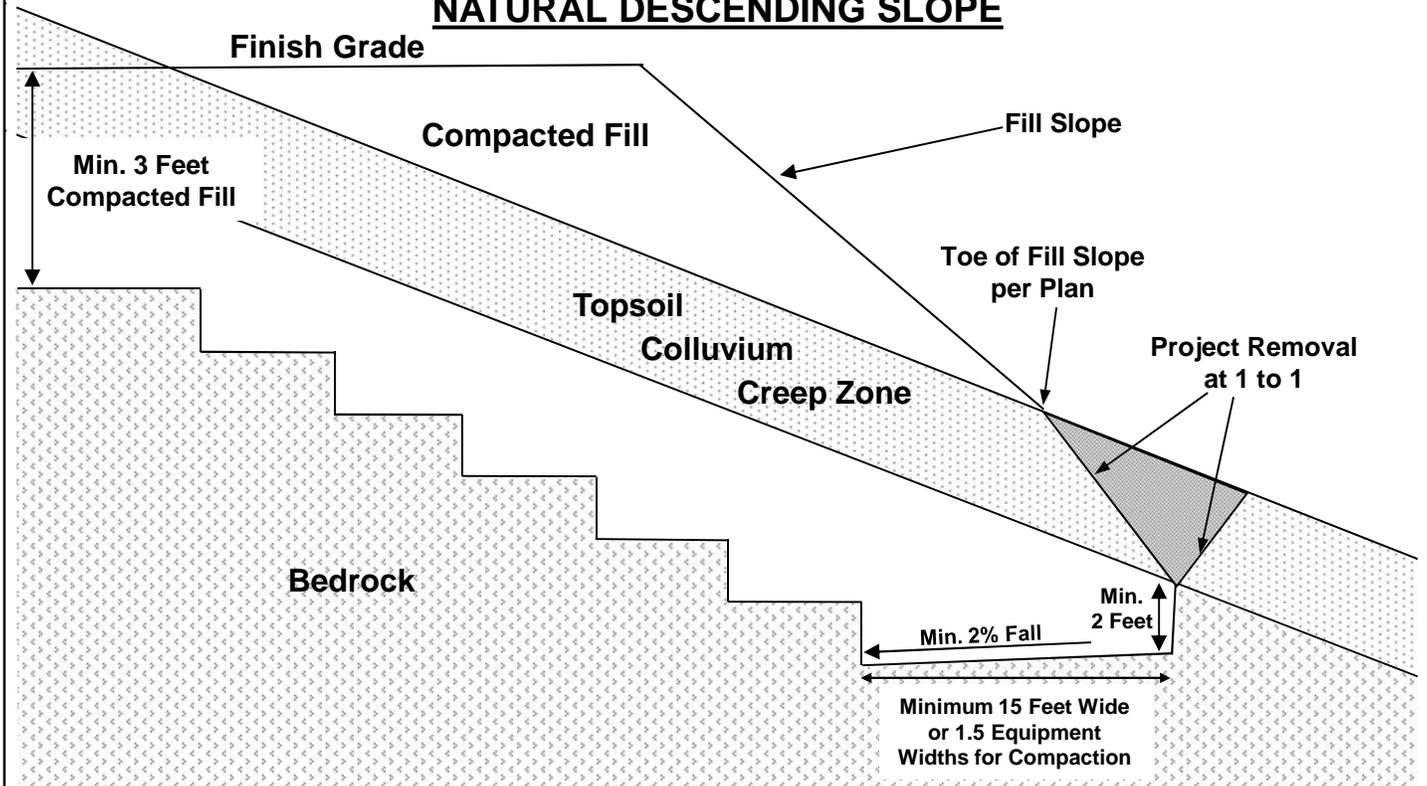


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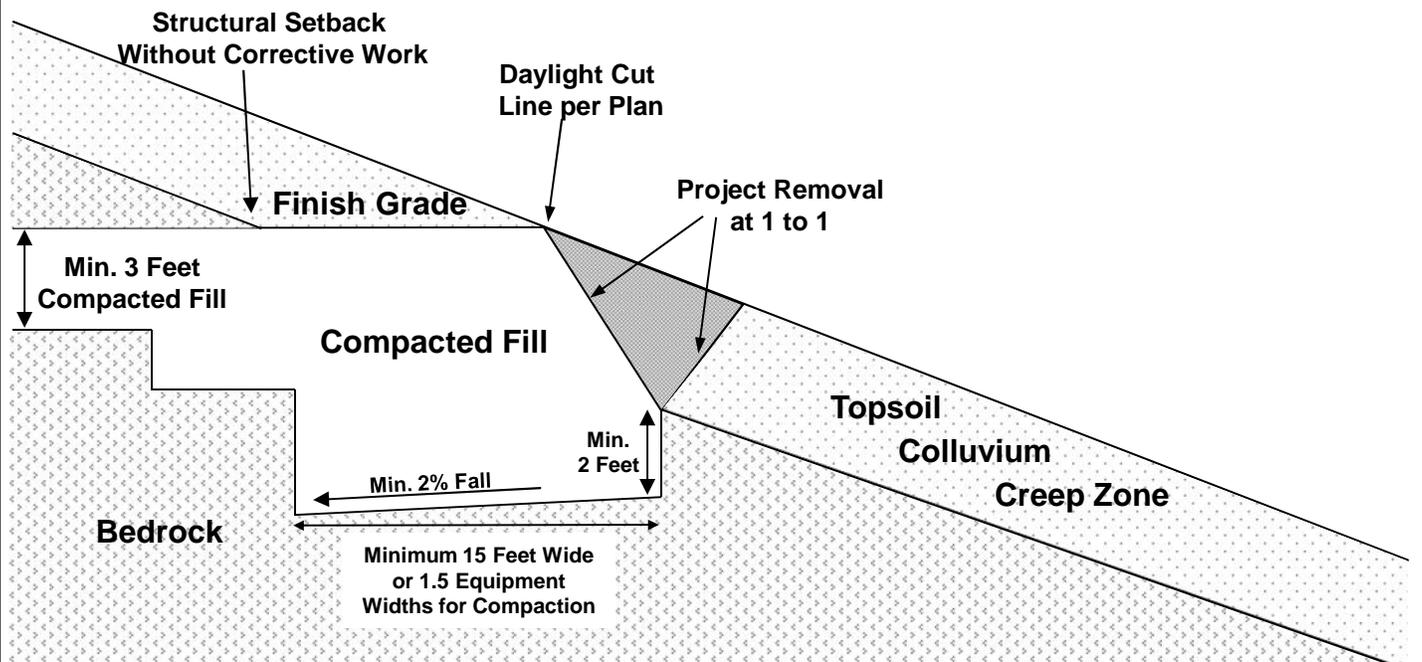
TYPICAL CANYON
CLEANOUT

STANDARD GRADING
GUIDELINES
PLATE G-1

TYPICAL FILL SLOPE OVER NATURAL DESCENDING SLOPE



DAYLIGHT CUT AREA OVER NATURAL DESCENDING SLOPE



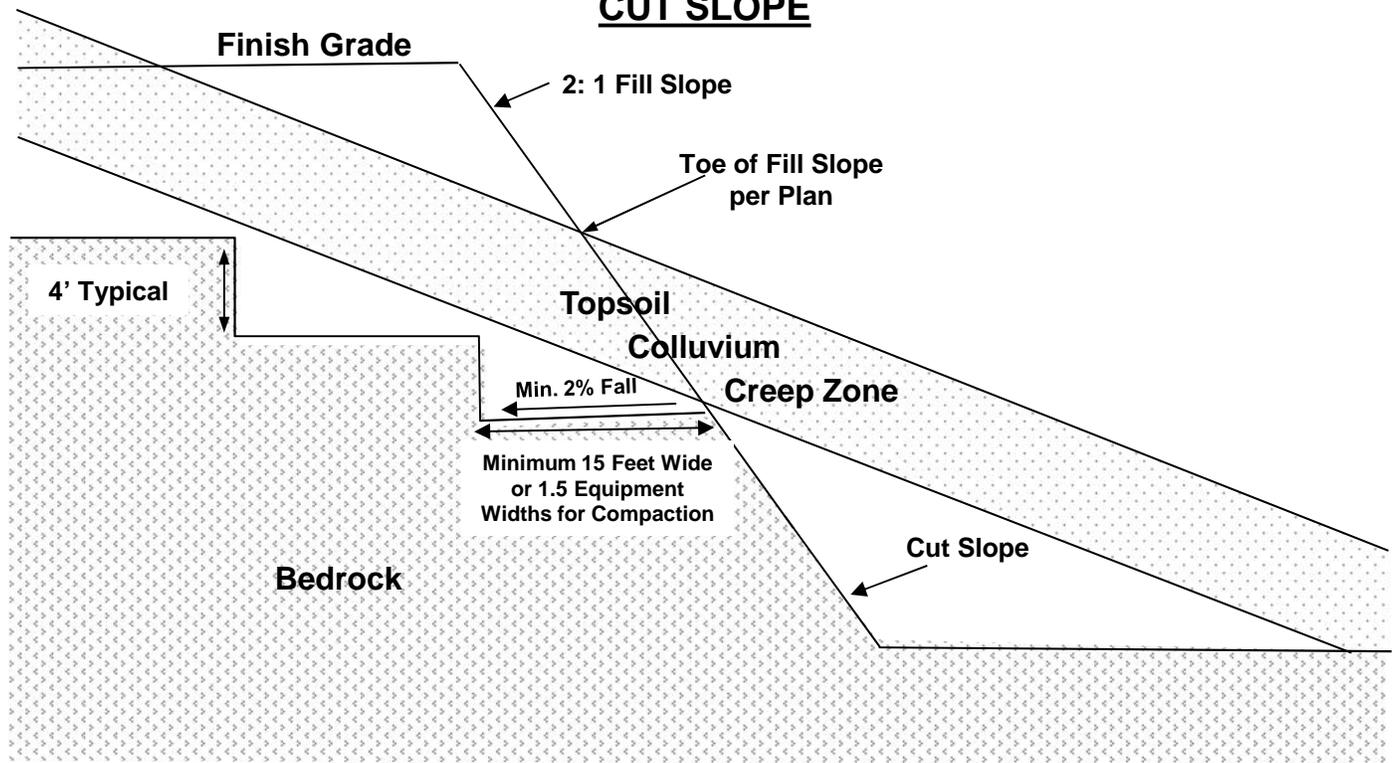
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**TREATMENT ABOVE
NATURAL SLOPES**

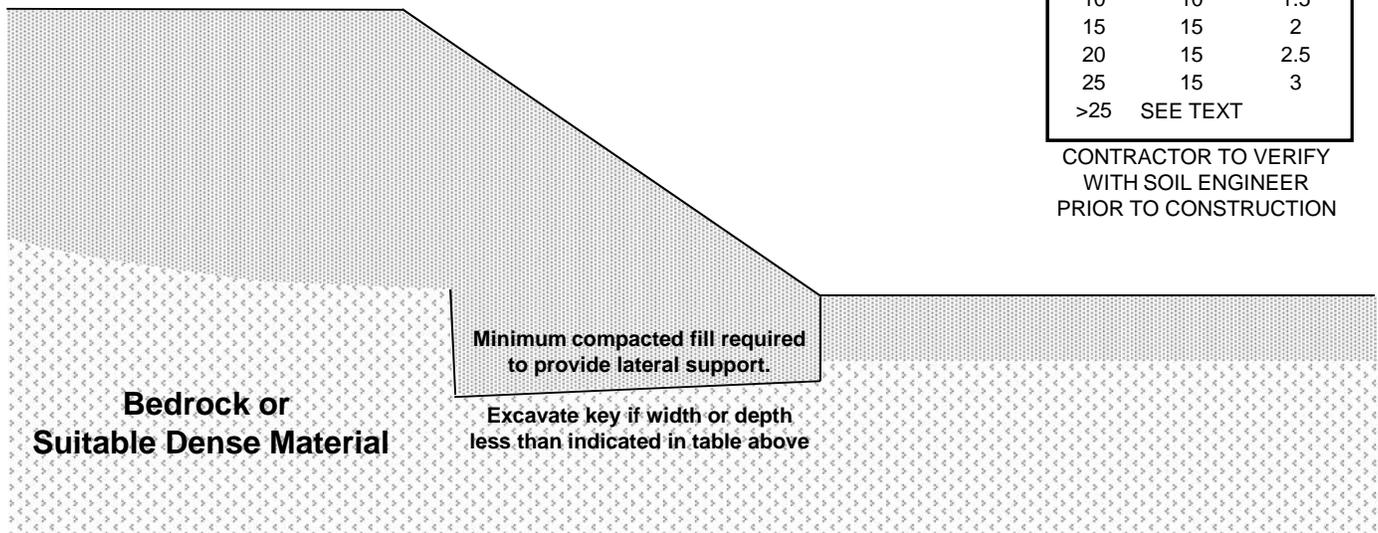
**STANDARD GRADING
GUIDELINES**

PLATE G-2

TYPICAL FILL SLOPE OVER CUT SLOPE



TYPICAL FILL SLOPE



SLOPE HEIGHT	MIN. KEY WIDTH	MIN. KEY DEPTH
5	7	1
10	10	1.5
15	15	2
20	15	2.5
25	15	3
>25	SEE TEXT	

CONTRACTOR TO VERIFY WITH SOIL ENGINEER PRIOR TO CONSTRUCTION

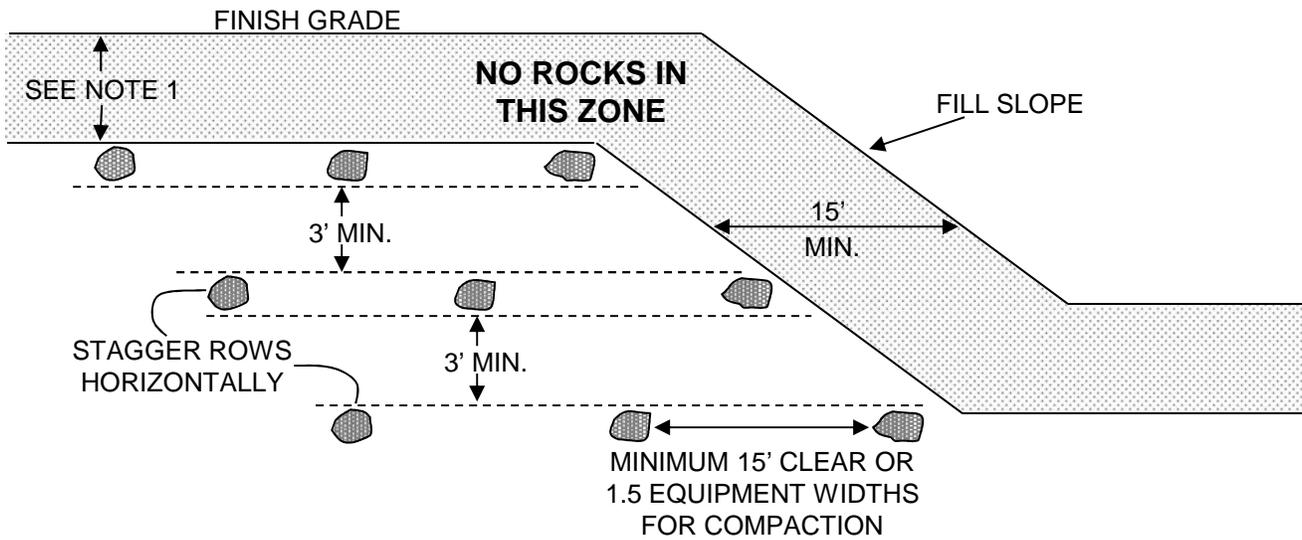


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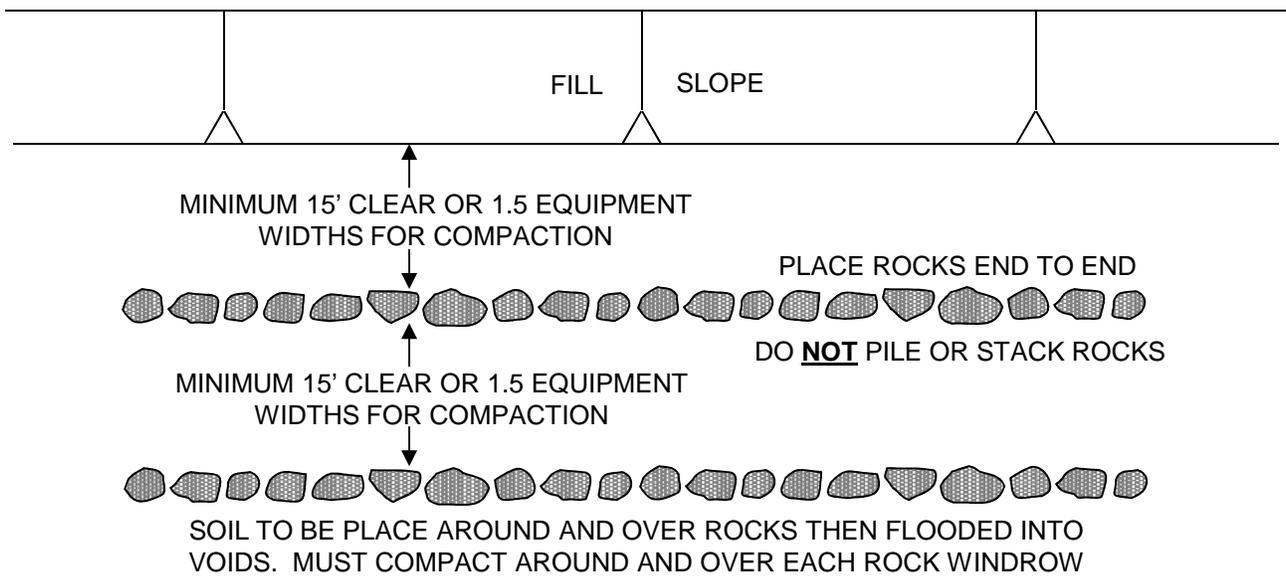
**COMMON FILL
SLOPE KEYS**

**STANDARD GRADING
GUIDELINES
PLATE G-3**

CROSS SECTIONAL VIEW



PLAN VIEW



NOTES:

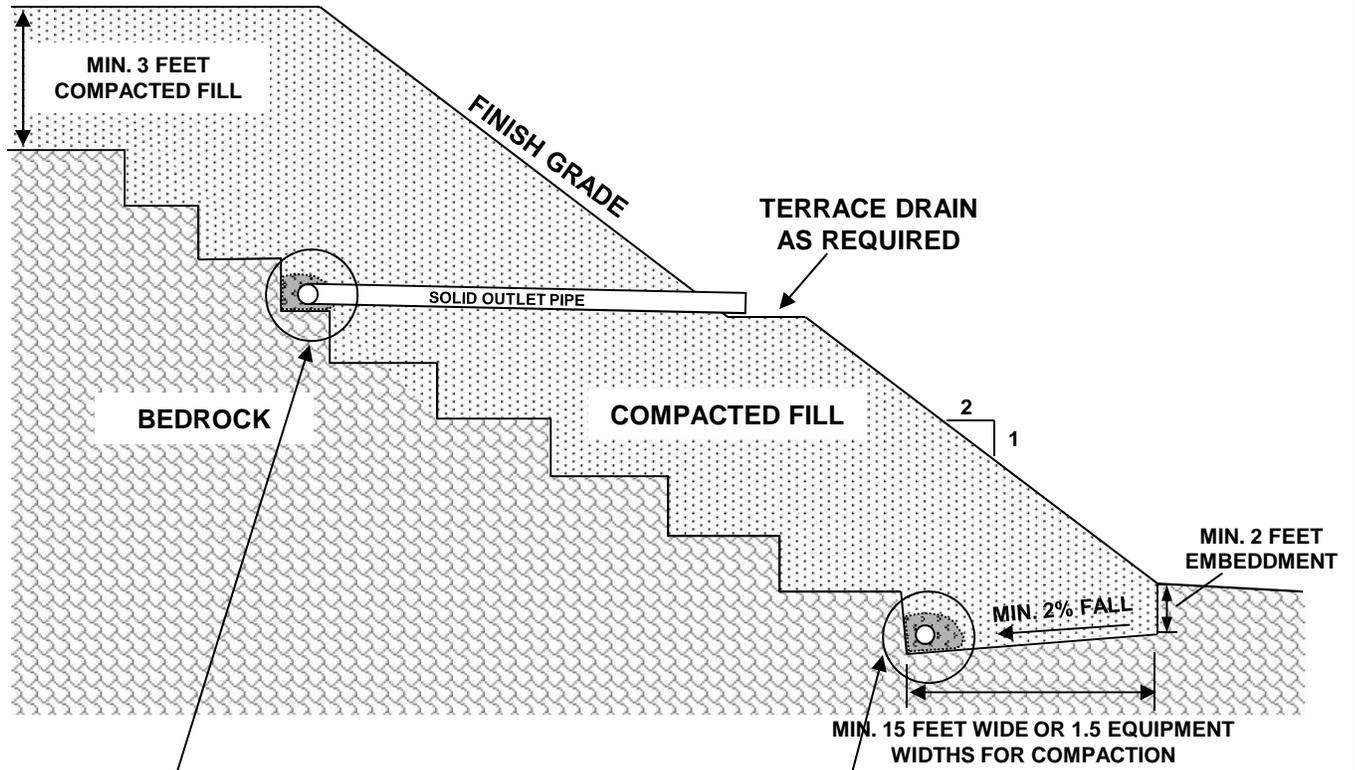
- 1) SOIL FILL OVER WINDROW SHOULD BE 7 FEET OR PER JURISDICTIONAL STANDARDS AND SUFFICIENT FOR FUTURE EXCAVATIONS TO AVOID ROCKS
- 2) MAXIMUM ROCK SIZE IN WINDROWS IS 4 FEET MINIMUM DIAMETER
- 3) SOIL AROUND WINDROWS TO BE SANDY MATERIAL SUBJECT TO SOIL ENGINEER ACCEPTANCE
- 4) SPACING AND CLEARANCES MUST BE SUFFICIENT TO ALLOW FOR PROPER COMPACTION
- 5) INDIVIDUAL LARGE ROCKS MAY BE BURIED IN PITS.



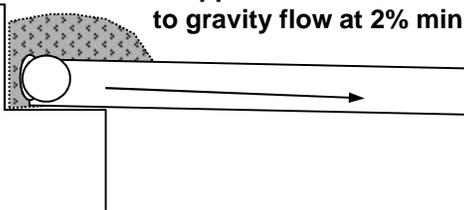
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ROCK BURIAL DETAILS

**STANDARD GRADING
GUIDELINES
PLATE G-4**



4" or 6" Perforated Pipe in 6 cubic feet per lineal foot clean gravel wrapped in filter fabric outlet pipe to gravity flow at 2% min.



6" Perforated Pipe in 6 cubic feet per lineal foot clean gravel wrapped in filter fabric outlet pipe to gravity flow

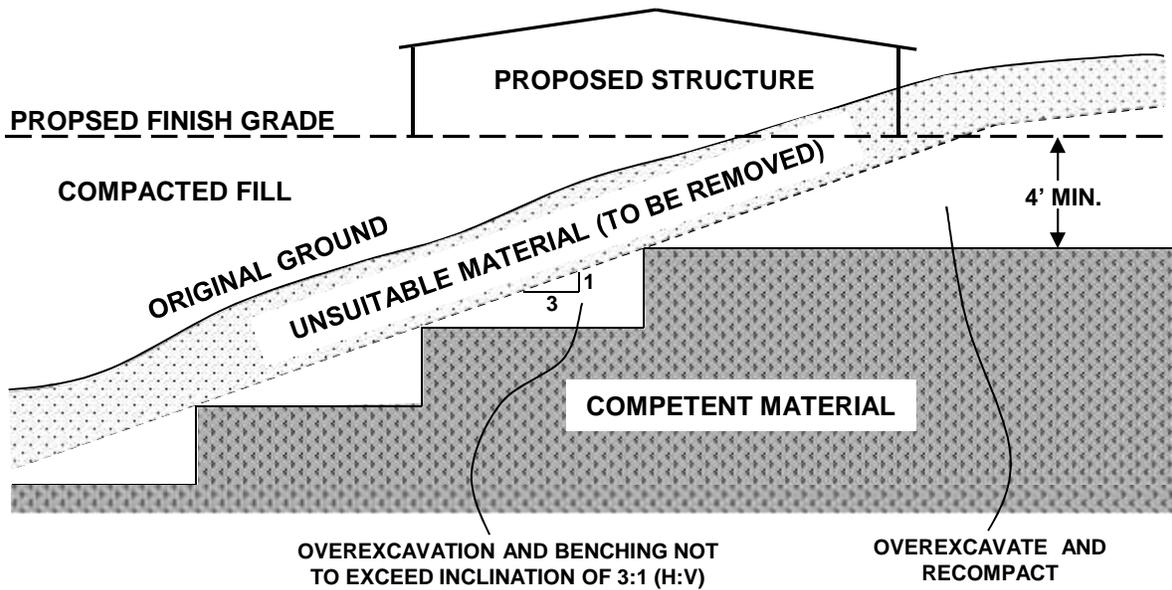


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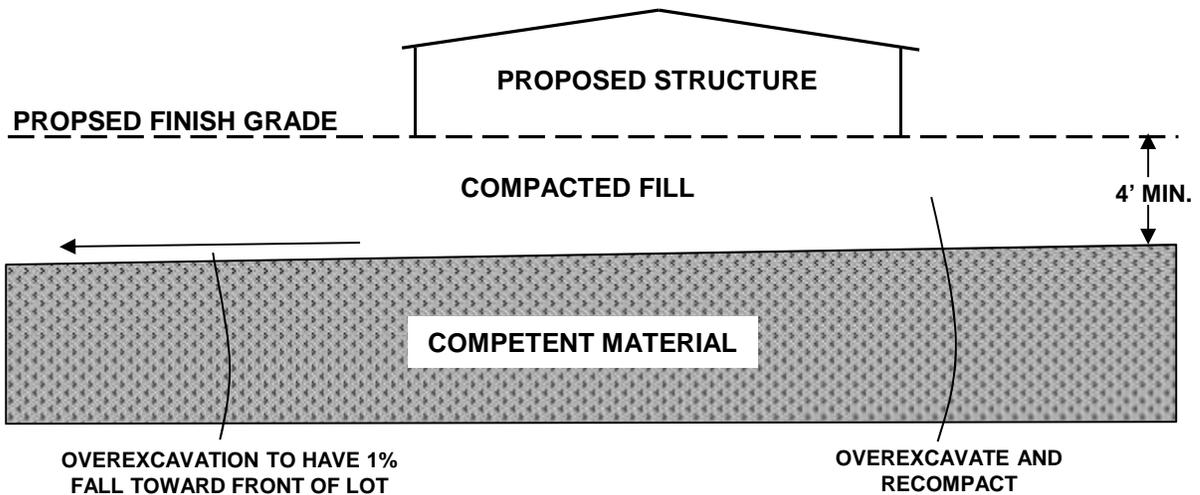
**Typical Buttress and
Stabilization Fill**

PLATE G-5

TRANSITION LOT



UNDERCUT LOT



Notes:

1. Removed/overexcavated soils should be recompactd in accordance with recommendations included in the text of the report.
2. Location of cut/fill transition should be verified in the field during site grading.



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**TRANSITION &
UNDERCUT LOTS**

**STANDARD GRADING
GUIDELINES
PLATE G-6**