GEOTECHNICAL AND INFILTRATION EVALUATION

PROPOSED SINGLE-FAMILY RESIDENTIAL TRACT DEVELOPMENT
DISCOVERY CHURCH (73 LOTS) PROJECT
APN 486-240-010-01
BRODIAEA AND OLIVER STREET
MORENO VALLEY, RIVERSIDE COUNTY, CALIFORNIA

PREPARED FOR

D. R. HORTON LOS ANGELES HOLDING COMPANY, INC.
2280 WARDLOW CIRCLE, SUITE 100
CORONA, CALIFORNIA 92878

PREPARED BY

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PROJECT NO. 2867-CR **SEPTEMBER 30, 2021**

GEOTEK

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September 30, 2021
Project No. 2867-CR

D. R. Horton Los Angeles Holding Company, Inc.
2280 Wardlow Circle, Suite 100
Corona, California 92878

Attention: Ms. Megan Whieldon

Subject: Geotechnical and Infiltration Evaluation
Proposed Single-Family Residential Tract Development
Discovery Church (73 Lots) Project – APN 486-240-010-01
Brodiaea Avenue and Oliver Street
Moreno Valley, Riverside County, California

Dear Ms. Whieldon:

GeoTek, Inc. (GeoTek) is pleased to provide the results of this geotechnical and infiltration evaluation for the proposed project located in Moreno Valley, Riverside County, California. This report presents the results of GeoTek's evaluation, discussion of findings, and provides geotechnical recommendations for foundation design and construction.

Based upon review and evaluation, site development appears feasible from a geotechnical viewpoint provided that the recommendations included in this report are incorporated into the design and construction phases of the project.

The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to contact GeoTek.

Respectfully submitted,
GeoTek, Inc.

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1. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate the geotechnical engineering and geologic conditions at the project site, as outlined in GeoTek's proposal P-00409821-CR, dated April 27, 2021. Services provided for this study included the following:

- Research and review of available geologic data and general information pertinent to the site,
- Site exploration consisting of the excavation, logging, and sampling of five (5) exploratory test borings extending to depths ranging from 15.5 to 51.5 feet below grade,
- Excavation of four (4) additional borings to a depth of about five (5) feet below grade and performing an infiltration test in each boring,
- Laboratory testing of soil samples collected during the field investigation,
- Review and evaluation of site seismicity, and
- Preparation of this geotechnical report which presents GeoTek's findings, conclusions, and recommendations for this site.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The approximate 9-acre rectangular-shaped project site is located adjacent to the northeast corner of Brodiaea Avenue and Oliver Street, in the City of Moreno Valley, Riverside County, California (See Figure 1). Access to the site is available from Brodiaea Avenue and Oliver Street, both paved, improved streets located adjacent to the southern and western boundaries of the site, respectively. An existing church facility is located adjacent to the northern boundary of the site while vacant land is located adjacent to the eastern boundary of the site.

Topographically, the site slopes gently downward to the south/southwest at an approximate two (2) percent gradient. Elevation of the northern portion of the the site is approximately 1,560 feet with approximately 10 feet of elevation differential across the site.

The site was vacant land at the time of the field exploration. A high-pressure gas line trends east-west along the northern border of the site. Remnants of a small structure and associated hardscaping are present in the central portion of the property. Soil stockpiles were observed in the west-central portion of the site. A light covering of grass and small brush was present in the site along with scattered domestic trees adjacent to edges of the property and in the central portion of the property. The site appears to have been disced for vegetation control in the past.

2.2 PROJECT DESCRIPTION

Based upon discussions with representatives of D. R. Horton, the project will consist of a single-family residential development of 73 lots. In addition to the planned 73 single-family residential lots, associated planned improvements include street improvements, underground utilities, hardscaping and landscaping. Stormwater disposal is to be by means of water quality basins. Based upon past experience, grading of the site will likely involve cuts and fills generally less than about five feet in height, not including any recommended remedial grading.

The proposed residential structures are anticipated to be of wood-frame construction, one- to two-stories in height, and incorporate conventional shallow foundations and concrete slab-on-grade floors. Sewage disposal will be by a public sewer. For the purposes of this report, it is assumed maximum column and wall loads will be about 50 kips and 2.5 kips per foot, respectively. Specific site development plans were not provided as of the date of this report. Once actual loads are known that information should be provided to GeoTek to determine if modifications to the recommendations presented in this report are warranted.

If site development differs from the assumptions made herein, the recommendations included in this report should be subject to further review and evaluation. Site development plans should be reviewed by GeoTek when they become available.

3. FIELD EXPLORATION AND LABORATORY TESTING

3.1 FIELD EXPLORATION

The field exploration for this report was conducted on September 10, 2021 and consisted of excavating five (5) geotechnical exploratory borings with a hollow-stem drill rig to depths ranging from about 15.5 to 51.5 feet below grade. The approximate locations of the GeoTek excavations are shown on the Exploration Location Map (Figure 2). A geologist from GeoTek logged the excavations and collected soil samples for use in subsequent laboratory testing. The logs of the exploratory borings are included in Appendix A.

Relatively undisturbed soil samples were recovered at various intervals in the geotechnical borings with a California sampler. The California sampler is a 3-inch outside diameter, 2.5-inch inside diameter, split barrel sampler lined with brass rings. The sampler was 18 inches long. The sampler conformed to the requirements of ASTM D 3550. A 140-pound automatic trip hammer was utilized, dropping 30 inches for each blow. The relatively undisturbed samples, together with bulk samples of representative soil types, were returned to the laboratory for testing and evaluation. The California sampler test data are presented on the boring logs in Appendix A. In Boring B-2 standard penetration tests (SPT) were performed with a 2.0-inch outside diameter, 1.5-inch inside diameter, split-barrel sampler. The sampler was 18 inches long. The inside diameter of the sampler shoe was 1.4 inches. The sampler was unlined. The sampler conformed to the requirements of ASTM D 1586. A 140-pound automatic trip hammer was utilized, dropping 30 inches for each blow. The sampler penetration test data are presented on the Log for Boring for Boring B-2.

Percolation Testing

In addition to the geotechnical exploratory borings, four (4) percolation test borings (I-1 through I-4) were excavated in the areas of the proposed storm water management basins to depths of about 5 feet. Infiltration testing was conducted in these borings in general accordance County of Riverside guidelines. The infiltration tests consisted of drilling eight-inch diameter test holes to the desired depth and installing approximately two inches of gravel in the bottom of the holes. A three-inch diameter perforated PVC pipe, wrapped in a filter sock, was placed in the excavations and the annular space was filled with gravel to prevent caving within the borings. Water was then placed in the borings to presoak the holes and percolation testing was performed following the pre-soak period. Following presoaking, the percolation tests were performed which consisted of adding water to each test hole and measuring the water drop over a 10 to 30-minute period. The water drop was recorded for six test intervals

(10-minute readings) or twelve test intervals (30-minute readings). Water was added to the test holes after each test interval. The field percolation rates were then converted to an infiltration rate using the Porchet Method. The infiltration rates calculated using the Porchet Method are presented in the following table:

SUMMARY OF INFILTRATION RATES		
Boring	Depth of Test (Feet)	Infiltration Rate (Inches per hour)
I-1	5.0	0.76
I-2	5.0	0.67
I-3	5.0	1.73
I-4	5.0	1.21

The results of the conversions indicate infiltration rate range from about 0.76 to 1.73 inch per hour. Copies of the percolation data sheets and the Porchet infiltration rate conversion calculations are presented in Appendix C. 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.

3.2 LABORATORY TESTING

Laboratory testing was performed on selected relatively undisturbed ring and bulk samples collected during the field exploration. The purpose of the laboratory testing was to confirm the field classification of the materials encountered and to evaluate their physical properties for use in the engineering design and analysis. Results of the laboratory testing program along with a brief description and relevant information regarding testing procedures are included on the exploratory borings logs included in Appendix A and in Appendix B.

4. GEOLOGIC AND SOILS CONDITIONS

4.1 REGIONAL SETTING

The subject property is situated in the Peninsular Ranges geomorphic province. The Peninsular Ranges province is one of the largest geomorphic units in western North America. It extends approximately 975 miles south of the Transverse Ranges geomorphic province to the tip 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 zone trend northwest-southeast and are found near the middle of the province. The San Andreas Fault zone borders the northeasterly margin of the province.

More specific to the subject property, the site is located in an area geologically mapped to be underlain by alluvium (Dibblee, T.W., and Minch, J.A., 2003). No active faults are shown in the immediate site vicinity on the maps reviewed for the area.

4.2 GENERAL SOIL CONDITIONS

A brief description of the earth materials encountered is presented in the following section. Based on the site reconnaissance, the exploratory excavations and review of published geologic maps, the area investigated is locally underlain by fill that is over younger alluvium.

4.2.1 Fill

While not encountered in any of the exploratory borings, fill deposits are anticipated in areas adjacent to existing streets and within the high pressure gas line along the northern boundary of the site and in the area of the previous site structure in the central portion of the site. This fill is most likely from locally derived sources. Fill may be present within unexplored areas of the site.

4.2.2 Younger Alluvium

Younger alluvial soils were encountered in all the borings and extended to the maximum depths explored (51.5 feet). As encountered in the borings, the alluvium consisted of interbedded layers of sandy silts, silty sands, clayey sands, and relatively clean sands with variable amounts of gravel (ML, SM, SC and SP soil types based upon the Unified Soil Classification System).

Based on the laboratory test results, the near surface soils have a "low" (21-50) expansion potential (ASTM D 4829). Based on the laboratory test results, the near surface soils have a soluble sulfate content of less than 0.1 percent (ASTM D 4327). The test results are provided in Appendix B.

4.3 SURFACE WATER AND GROUNDWATER

4.3.1 Surface Water

If encountered during earthwork operations, surface water on this site is the result of precipitation or possibly some minor surface run-off from the surrounding areas. Overall site area drainage varies due to the site topography and existing improvements. Provisions for surface drainage will need to be accounted for by the project civil engineer.

4.3.2 Groundwater

Groundwater was not encountered within any of the exploratory borings drilled at the site to the maximum depth drilled of 51.5 below the existing ground surface. Based on a review of groundwater depths noted on the State Department of Water Resources Water Data Library website, it is estimated the historic high groundwater depth is in excess of 100 feet below existing grade at the site. Based on the results of the field exploration, review of site area geomorphology and geology, groundwater is not anticipated to adversely affect the proposed improvements.

4.4 FAULTING AND SEISMICITY

4.4.1 Faulting

The geologic structure of the entire California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. However, the site is not situated within a State of California designated *"Alquist-Priolo"* Earthquake Fault Zone. The County of Riverside indicates that the site is "not in a fault zone," "not in a fault line," having a "low to moderate" liquefaction potential, and is "susceptible" to subsidence. The nearest known active fault is the San Andreas fault located about 3.75 miles to the northeast.

4.4.2 Seismic Design Parameters

The site is located at approximately 33.9147 degrees West Latitude and -117.1816 degrees North Longitude. Site spectral accelerations (S_a and S_1) for 0.2 and 1.0 second periods for a Class "D" site, was determined from the SEAOC/OSHPD web interface that utilizes the USGS web services and retrieves the seismic design data and presents that information in a report format. Using the ASCE 7-16 option on the SEAOC/OSHPD website results in the values for S_{M1} and S_{D1} reported as "null-See Section 11.4.8" (of ASCE 7-16). As noted in ASCE 7-16, Section 11.4.8, a site-specific ground motion procedure is recommended for Site Class D when the value S_1 exceeds 0.2. The value S_1 for the subject site exceeds 0.2.

For a site Class D, an exception to performing a site-specific ground motion analysis is allowed in ASCE 7-16 where S_1 exceeds 0.2 provided the value of the seismic response coefficient, Cs, is conservatively calculated by Eq 12.8-2 of ASCE 7-16 for values of $T \leq 1.5Ts$ and taken as equal to 1.5 times the value computed in accordance with either Eq. 12.8-3 for $T_L \geq T > 1.5Ts$ or Eq. 12.8-4 for $T > T_L$.

The results, based on the 2015 NEHRP and the 2019 CBC, are presented in the following table assuming that the exception as allowed in ASCE 7-16 is applicable. If the exception is deemed not appropriate, a site-specific ground motion analysis will be required.

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_s	1.905g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.75g
Site Coefficient for Site Class "D", F_a	1
Site Coefficient for Site Class "D", F_v	1.7
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	1.905g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	1.276g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	1.27g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.851g
Peak Ground Acceleration (PGA_M)	0.885
Seismic Design Category	E

Final selection of the appropriate seismic design coefficients should be made by the project structural engineer based upon the local practices and ordinances, expected building response and desired level of conservatism.

4.5 LIQUEFACTION

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 site is not designated as having the potential for liquefaction by the State of California; however, the County of Riverside indicates that the site has "low to moderate" liquefaction potential. The depth to groundwater at the site is estimated to be greater than 100 feet below grade. Based on the depth to groundwater, it is GeoTek's opinion that the potential for liquefaction at this site is very low.

4.6 OTHER SEISMIC HAZARDS

The potential for seismic densification (dry seismic settlement) resulting from seismic activity was assessed. For this analysis, the soil profile identified within Boring B-2, a ground acceleration (PGA_M) of 0.885g and a mean earthquake magnitude of 7.0. The ground acceleration and earthquake magnitude values were obtained from the USGS websites. The computer software program LiquefyPro Version 5 was utilized to estimate the dry seismic settlement potential.

The result of this analysis indicates a total seismic settlement of a little more than three-quarters of an inch. It is estimated a seismic differential settlement of ½ the total estimated settlement over a 40-foot span will occur. Based on the magnitudes of estimated seismic settlements, special mitigation or design is not considered necessary. However, the estimated seismic settlements should be considered in structural design. A copy of the computer output file for this analysis is presented in Appendix D.

Due to the general flat terrain, the potential for seismic induced landslides or lateral spreading is considered nil. The potential for secondary seismic hazards such as a seiche and tsunami is considered negligible due to site elevation and distance from an open body of water.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

Development of the site appears feasible from a geotechnical engineering viewpoint. The following recommendations should be incorporated into the design and construction phases of development.

5.2 EARTHWORK CONSIDERATIONS

5.2.1 General

Earthwork and grading should be performed in accordance with the applicable grading ordinances of the County of Riverside, City of Moreno Valley and the 2019 California Building Code (CBC), and recommendations contained in this report. The Grading Guidelines included in Appendix F 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 F.

5.2.2 Site Clearing

Initial site preparation should commence with removal of debris, deleterious materials and vegetation within the limits of the planned improvements. These materials should be properly disposed of off-site. Voids resulting from removing any materials should be replaced with engineered fill materials with expansion characteristics similar to the onsite materials.

5.2.3 Site Preparation

Due to the non-uniform nature and thickness of the near-surface undocumented fill and loose condition of the upper younger alluvium, it is recommended that the soils be removed beneath the planned building footprint of the proposed structure to a depth of at least 4 feet below existing natural (below existing fill) grade, or two (2) feet beneath the base of the proposed foundations, whichever is greater. 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. A representative of this firm should observe and approve the bottom of all remedial excavations. The lateral extent of this recommended over-excavation should extend at least 5 feet beyond the building or foundation limits.

Following site clearing operations, over-excavation and lowering of site grades, where necessary, it is recommended that the exposed subgrade soils beneath all surface improvements be proof rolled with a heavy rubber-tired piece of construction equipment approved by and in the presence of the geotechnical engineering representative. The proof rolling equipment should possess a minimum weight of 15 tons and proof rolling should include at least 4 passes, two in each perpendicular direction. All soil that ruts or excessively deflects during proof rolling should be removed as recommended by the GeoTek representative. Following proof rolling and removal of any unsuitable bearing soil, the exposed subgrade should be scarified to a depth of about 12 inches, be moisture conditioned to slightly above the soil's optimum moisture content and then be compacted to at least 90 percent of the soil's maximum dry density as determined by ASTM D-1557 test procedures.

5.2.4 Engineered Fill

The on-site soils are generally considered suitable for reuse as engineered fill provided they are free from vegetation, debris, oversized materials (6 inch diameter or greater) and other deleterious material. All areas should be brought to final subgrade elevations with fill materials that are placed and compacted in general accordance with minimum project standards. Engineered fill should be placed in 6- to 8-inch loose lifts, moisture conditioned to slightly above the optimum moisture content and compacted to a minimum relative compaction of 90 percent as determined by ASTM D-1557 test procedures.

If wet soils are encountered during remedial grading, methods for drying soils such as stockpiling or mixing with dry soils may be required to bring the soils to the required moisture content for placement as engineered fill. Placement of engineered fill should be observed and tested on a full-time basis by a GeoTek representative during grading activities.

5.2.5 Transition Lot Condition

Building pads graded with a cut/fill transition should be undercut to reduce the potential for differential settlement. The cut portion of the cut/fill transition should be undercut to a depth of at least 3 feet or one (1) foot below the deepest proposed footing, whichever is deeper, and be backfilled with a properly compacted engineered fill. The bottom of the undercut should be sloped at a minimum of 1 percent toward the adjacent lot area.

5.2.6 Oversized Rock Disposal

Although unlikely, oversized cobbles, boulders and rock fragments may be encountered during rough grading and utility trench operations. If encountered, on-site disposal of oversized materials is possible, provided the oversized materials are placed as recommended on Plate 4 within Appendix E. Alternatively, over-sized materials can be exported from the site.

5.2.7 Excavation Characteristics

Excavations in the on-site younger alluvium should be readily accomplished with heavy-duty earthmoving or excavating equipment in good operating condition. All excavations should be formed in accordance with current Cal-OSHA requirements.

5.2.8 Trench Excavations and Backfill

Temporary trench excavations within the on-site materials should be stable at a 1:1 inclination for short durations during construction and where cuts do not exceed 15 feet in height. Deeper temporary excavations should be reviewed by GeoTek prior to their planned excavation to determine if supplemental recommendations or analysis are warranted. It is anticipated that 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 percent relative compaction (as determined by ASTM D-1557 test procedures). Under-slab trenches should also be compacted to project specifications. Where applicable, based on jurisdictional requirements, the top 12 inches of backfill below subgrade for road pavements should be compacted to at least 95 percent relative compaction. On-site materials may not be suitable for use as bedding material but should be suitable as backfill provided particles larger than 6 inches are removed.

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 properly moisture conditioned prior to placement in trenches.

5.2.9 Shrinkage and Bulking

For planning purposes, a shrinkage loss of about 15 to 25 percent is anticipated for excavations within the younger alluvium in the upper 1 foot of the site. Shrinkage ranging from 10 to 20 percent is anticipated for materials necessitating removal and replacement below the upper 1 foot. Several factors will impact earthwork balancing on the site, including shrinkage, trench spoil from utilities and footing excavations, as well as the accuracy of topography. Shrinkage and bulking are primarily dependent upon the degree of compactive effort achieved during construction, depth of fill and underlying site conditions. A subsidence loss ranging from about 0.1 to 0.2 foot is estimated for the site.

Site balance areas should be available in order to adjust project grades, depending on actual field conditions at the conclusion of earthwork construction.

5.2.10 Grading Plan Review

Upon completion of the site grading plans, it is recommended that those plans be provided to GeoTek for review. Based on that review, some modifications to the recommendations provided in this report may be necessary.

5.3 DESIGN RECOMMENDATIONS

5.3.1 Foundation Design Criteria

Foundation design criteria for a conventional foundation system, in general conformance with the 2019 CBC, are presented herein. These are typical design criteria and are not intended to supersede the design by the structural engineer.

Based on the expansion index testing performed for this report and visual examination of the site soils, site soils possess a "low" (21-50) expansion potential (ASTM D4829). Therefore, it is GeoTek's opinion that conventional foundations supported by engineered fill may be used for this site.

A summary of GeoTek's preliminary foundation design recommendations is presented in the table below:

Design Parameter	"Low" Expansion Potential (21≤EI≤50)
Foundation Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent grade)	12" – One and Two-Stories
Minimum Foundation Width (Inches)*	12 – 1-Story 15 – 2-Story
Minimum Slab Thickness (actual)	4 inches
Minimum Slab Reinforcing	6" x 6" – W2.9/W2.9 welded wire fabric placed in middle of slab or No. 3 bars at 18-inch centers
Minimum Footing Reinforcement	Two (2) No. 4 Reinforcing Bars, one (1) top and one (1) bottom
Effective Plasticity Index	PI<35
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum 110% to a depth of 12 inches prior to placement of concrete

*Code minimums per Table 1809.7 of the 2019 CBC.

It should be noted that the criteria provided are based on soil support characteristics only. The structural engineer should design the slab and beam reinforcement based on actual loading conditions.

The following criteria for design of foundations are preliminary and should be re-evaluated based on the results additional laboratory testing of samples obtained at/near finish pad grade.

5.3.1.1 An allowable bearing capacity of 2,000 pounds per square foot (psf) may be used for design of continuous and perimeter footings 12 inches deep and 12 inches wide, and pad footings 24 inches square and 12 inches deep. This allowable soil bearing capacity may be increased by 300 psf for each additional foot of footing depth and 300 psf for each additional foot of footing width to a maximum value of 4,000 psf. An increase of one-third may be applied when considering short-term live loads (e.g., seismic and wind loads).

5.3.1.2 Structural foundations should be designed in accordance with the 2019 CBC, and to withstand a total static settlement of 1 inch and maximum differential static settlement of one-half of the total settlement over a horizontal distance of 40 feet.

5.3.1.3 The passive earth pressure may be computed as an equivalent fluid having a density of 325 psf per foot of depth, to a maximum earth pressure of 3,250 psf for footings founded on engineered fill or competent native soil. A coefficient of friction between soil and concrete of 0.30 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third. The upper one foot of soil should be ignored in the passive pressure calculations unless the surface is covered with pavements.

5.3.1.4 A grade beam, a minimum of 12 inches wide and 12 inches deep, should be utilized across large entrances. The base of the grade beam should be at the same elevation as the bottom of the adjoining footings.

5.3.1.5 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, the 2019 CBC Section 1907.1 and ACI 360R-10. The vapor retarder design and construction should also meet the requirements of ASTM E 1643. A portion of the vapor retarder design should be the implementation of a moisture vapor retardant membrane.

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. Although the CBC specifies a 6-mil vapor retarder membrane, it is GeoTek's opinion that a minimum 10 mil thick membrane with joints properly overlapped and sealed should be considered, unless otherwise specified by the slab design professional. The membrane should consist of Stego wrap or the equivalent.

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 limited 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 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.

5.3.1.6 It is recommended that control joints be placed in two directions spaced approximately 24 to 36 times the thickness of the slab in inches. These joints are a widely accepted means to control cracks and should be reviewed by the project structural engineer.

5.3.2 Miscellaneous Foundation Recommendations

5.3.2.1 To reduce moisture penetration beneath the slab on grade areas, utility trench excavations should be backfilled with engineered fill, lean concrete or concrete slurry where they intercept the perimeter footing or thickened slab edge.

5.3.2.2 Soils 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.

5.3.3 Foundation Setbacks

Minimum setbacks for all foundations should comply with the 2019 CBC or City of Moreno Valley requirements, whichever is more stringent. Improvements not conforming to these setbacks are subject to the increased likelihood of excessive lateral movements and/or differential settlements. If large enough, these movements can compromise the integrity of the improvements. The top outside edge of all footings should be set back a minimum of H/3 (where H is the slope height) from the face of any descending slope. The setback should be at least five feet and need not exceed 40 feet.

5.3.4 Soil Corrosivity

A soil corrosivity report was prepared for the project by GeoTek's sub-consultant HDR, Inc. The corrosivity report is included in Appendix E. In general, the report concluded that the soils are moderately corrosive to ferrous metals, aggressive to copper and provided mitigation recommendations for such conditions.

Soil Sulfate Content

The soil sulfate content was determined as part of the testing conducted for preparation of the soil corrosivity report (see Appendix E). The results indicate that the water-soluble sulfate result is less than 0.1 percent by weight, which is considered "negligible" as per Table 4.2.1 of ACI 318. Based on the test results and Table 4.3.1 of ACI 318, no special recommendations for concrete are required for this project due to soil sulfate exposure.

5.4 RETAINING AND GARDEN WALL DESIGN AND CONSTRUCTION

5.4.1.1 General Design Criteria

Recommendations presented in this report apply to typical masonry or concrete vertical retaining walls to a maximum height of up to six (6) feet. Additional review and recommendations should be requested for higher walls. These are typical design criteria and are not intended to supersede the design by the structural engineer.

Retaining wall foundations should be embedded a minimum of 18 inches into engineered fill. Retaining wall foundations should be designed in accordance with Section 5.3 of this report. Structural needs may govern and should be evaluated by the project structural engineer.

All earth retention structure plans, as applicable, should be reviewed by this office prior to finalization.

Earthwork considerations, site clearing and remedial earthwork for all earth retention structures should meet the requirements of this report, unless specifically provided otherwise, or more stringent requirements or recommendations are made by the designer. The backfill material placement for all earth retention structures should meet the requirement of Section 5.2.4 in this report.

In general, cantilever earth retention structures, which are designed to yield at least 0.001H, where H is equal to the height of the earth retention structure, may be designed using the “active” condition. Rigid earth retention structures (including but not limited to rigid walls, and walls braced at top, such as typical basement walls) should be designed using the “at-rest” condition.

In addition to the design lateral forces due to retained earth, surcharges due to improvements, such as an adjacent building or traffic loading, should be considered in the design of the earth retention structures. Loads applied within a 1:1 (horizontal:vertical) projection from the surcharge on the stem of the earth retention structure should be considered in the design.

Final selection of the appropriate design parameters should be made by the designer of the earth retention structures.

5.4.1.2 Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to six (6) feet high. Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions such as traffic, structures, seismic events, or adverse geologic conditions.

ACTIVE EARTH PRESSURES	
Surface Slope of Retained Materials (horizontal:vertical)	Equivalent Fluid Pressure (pcf) Select Backfill* and Native Soils
Level	40
2:1	62

*The design pressures assume the backfill material has an expansion index less than or equal to 20. Backfill zone includes area between back of the wall to a plane (1:1 horizontal : vertical) up from bottom of the wall foundation (on the backside of the wall) to the ground surface.

For walls with a retained height greater than 6 feet, an incremental seismic pressure should be included into the wall design. Where needed, it is recommended that an equivalent fluid pressure of 20 pcf be included into the wall design to account for seismic loading conditions. This pressure may be applied as an inverted triangular distribution.

5.4.1.3 Retaining Wall Backfill and Drainage

The wall backfill should also include a minimum one (1) foot wide section of ¾- to 1-inch clean crushed rock (or an approved equivalent). The rock should be placed immediately adjacent to the back of the wall and extend up from a back drain to within approximately 24 inches of the finish grade. The upper 24 inches should consist of compacted on-site materials. The rock should be separated from the earth with filter fabric. The presence of other materials might necessitate revision to the parameters provided and modification of the wall designs. The backfill materials should be placed in lifts no greater than eight (8) inches in thickness and compacted to a minimum of 90% relative compaction as determined by ASTM D 1557 test procedures. Proper surface drainage needs to be provided and maintained.

As an alternative to the drain, rock and fabric, a pre-manufactured wall drainage product (example: Mira Drain 6000 or approved equivalent) may be used behind the retaining wall. The wall drainage product should extend from the base of the wall to within two (2) feet of the ground surface. The subdrain should be placed in direct contact with the wall drainage product.

Retaining walls should be provided with an adequate pipe and gravel back drain system to help prevent buildup of hydrostatic pressures. Backdrains should consist of a four (4)-inch diameter perforated collector pipe (Schedule 40, SDR 35, or approved equivalent) embedded in a minimum of one (1) cubic foot per linear foot of ¾- to 1-inch clean crushed rock or an approved equivalent, wrapped in filter fabric (Mirafi 140N or an approved equivalent). The drain system should be connected to a suitable outlet. Waterproofing of site walls should be performed where moisture migration through the walls is undesirable.

5.4.1.4 Restrained Retaining Walls

Retaining walls that will be restrained at the top that support level backfill or that have reentrant or male corners, should be designed for an equivalent at-rest fluid pressure of 60 pcf, plus any applicable surcharge loading. For areas of male or reentrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall laterally from the corner, or a distance otherwise determined by the project structural engineer.

5.4.1.5 Other Design Considerations

- Wall design should consider the additional surcharge loads from superjacent slopes and/or footings, where appropriate.
- No backfill should be placed against concrete until minimum design strengths are evident by compression tests of cylinders.
- The retaining wall footing excavations, backcuts, and backfill materials should be approved by the project geotechnical engineer or their authorized representative.
- Positive separations should be provided in garden walls at horizontal distances not exceeding 20 feet.

5.5 PRELIMINARY PAVEMENT DESIGN RECOMMENDATIONS

No on-site earth material has been tested to determine a preliminary R-Value for pavement design. A R-Value of 40 is assumed for the determination of preliminary pavement sections for this report. The final design should be based on R-Value testing of the soil subgrade following completion of rough grading operations. Project streets should be designed in accordance with County of Riverside requirements when final Traffic Indices and R-Value test results of the subgrade soil are completed.

Pavement design for proposed on-site and off-site street improvements was conducted per Caltrans *Highway Design Manual* guidelines for flexible pavements. Based on Traffic Indices (TIs) specified by the City of Moreno Valley (Standard Plan MVSI-100A-1, approved 9/14/18) and using a design R-value of 40, the following preliminary sections were calculated:

PRELIMINARY PAVEMENT SECTIONS			
TI	R-Value	Thickness of Asphalt Concrete (inches)	Thickness of Aggregate Base (inches)
6.0 (Local Street, Modified Local Street)	40	3.6*	6*
7.0 (Collector)		3.6*	7
9.0 (Minor Arterial)		5.4*	9*

*Minimum pavement structural section per City of Moreno Valley Standards

The TIs used in the above pavement analysis and design were designated by the City of Moreno Valley for the indicated street types and should provide a pavement life of approximately 20 years with a normal amount of flexible pavement maintenance. Irrigation adjacent to pavements, without a deep curb or other cutoff to separate landscaping from the paving may result in premature pavement failure. Traffic parameters used for design were selected based upon engineering judgment and not upon information furnished to us such as an equivalent wheel load analysis or a traffic study.

The recommended pavement sections provided are intended as a minimum guideline and final selection of pavement cross section parameters should be made by the project civil engineer, based upon the local laws and ordinates, expected subgrade and pavement response, and desired level of conservatism. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. Final pavement design should be checked by testing of soils exposed at subgrade (the upper 12 inches) after final grading has been completed.

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 ASTM D1557 laboratory maximum dry density (modified proctor).

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 City of Moreno Valley 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.

Deleterious material, excessive wet or dry pockets, oversized rock fragments, and other unsuitable yielding materials encountered during grading should be removed. Once existing compacted fill are brought to the proposed pavement subgrade elevations, the subgrade should be proof rolled in order to check for a uniform and unyielding surface. The upper 12 inches of pavement subgrade soils should be scarified, moisture conditioned at or near optimum moisture content, and recompacted to at least 95 percent of the laboratory maximum dry density as determined by ASTM D1557 test procedures. If loose or yielding materials are encountered during construction, additional evaluation of these areas should be carried out by GeoTek. All pavement section changes should be properly transitioned.

5.6 CONCRETE CONSTRUCTION

5.6.1 General

Concrete construction should follow the 2019 CBC and ACI guidelines regarding design, mix placement and curing of the concrete. If desired, GeoTek could provide quality control testing of the concrete during construction.

5.6.2 Concrete Mix Design

As discussed in Section 5.3.5, no special recommendations for concrete are required for this project due to soil sulfate exposure. Additional testing should be performed during grading so that specific recommendations can be formulated based on the as-graded conditions.

5.6.3 Concrete Flatwork

Exterior concrete flatwork is often one of the most visible aspects of site development. They are typically given the least level of quality control, being considered "non-structural" components. Cracking of these features is common due to various factors. While cracking usually does not affect the structural performance of the concrete, it is unsightly. It is recommended that the same standards of care be applied to these features as to the structure itself.

Flatwork should consist of a minimum four-inch (actual) thick concrete and the use of temperature and shrinkage control reinforcement is suggested. The project structural engineer should provide final design recommendations.

5.6.4 Concrete Performance

Concrete cracks should be expected. These cracks can vary from sizes that are hairline to more than 1/8 inch in width. Most cracks in concrete while unsightly do not significantly impact long-term performance. While it is possible to take measures (proper concrete mix, placement, curing, control joints, etc.) to reduce the extent and size of cracks that occur, some cracking will occur despite the best efforts to minimize it. Concrete undergoes chemical processes that are dependent on a wide range of variables, which are difficult, at best, to control. Concrete, while seemingly a stable material, is subject to internal expansion and contraction due to external changes over time.

One of the simplest means to control cracking is to provide weakened control joints for cracking to occur along. These do not prevent cracks from developing; they simply provide a relief point for the stresses that develop. These joints are a widely accepted means to control cracks but are not always effective. Control joints are more effective the more closely spaced they are. GeoTek suggests that control joints be placed in two orthogonal directions and located a distance apart approximately equal to 24 to 36 times the slab thickness.

5.7 PLAN REVIEW AND CONSTRUCTION OBSERVATIONS

It is recommended that site grading, specifications, and foundation plans be reviewed by this office prior to construction to check for conformance with the recommendations of this report. It is also recommended that GeoTek representatives be present during site grading and foundation construction to observe and document for proper implementation of the geotechnical recommendations. The owner/developer should have GeoTek perform at least the following duties:

- Observe site clearing and grubbing operations for proper removal of all 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 where necessary.
- Observe the fill for uniformity during placement, including utility trench excavation backfill. Also, test the fill for density, relative compaction and moisture content.

- Observe and probe foundation excavations to confirm suitability of bearing materials with respect to density.

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. INTENT

It is the intent of this report to aid in the design and construction of the proposed development. Implementation of the advice presented in this report is intended to reduce risk associated with construction projects. The professional opinions and geotechnical advice contained in this report are not intended to imply total performance of the project or guarantee that unusual or variable conditions will not be discovered during or after construction.

The scope of GeoTek's evaluation is limited to the area explored that is shown on the Exploration Location Map (Figure 2). This evaluation does not and should in no way be construed to encompass any areas beyond the specific area of the proposed construction as indicated to GeoTek 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-00409821-CR) dated April 27, 2021 and geotechnical engineering standards normally used on similar projects in this region.

7. LIMITATIONS

GeoTek's findings are based on site conditions observed and the stated sources. Thus, GeoTek's comments are professional opinions that are limited to the extent of the available data.

GeoTek has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering at this time and location and science professions currently practicing under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report.

Since GeoTek's recommendations are based on the site conditions observed and encountered at the stated times and laboratory testing. Thus, 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 of any kind is expressed or implied. Standards of care/practice are subject to change with time.

8. SELECTED REFERENCES

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Modified from Google Earth Pro Aerial Imagery

1500 Ft.

D.R. Horton a Los Angeles Holding Company
Discovery Church Project
APN 486-240-010-01
Moreno Valley, Riverside County, California

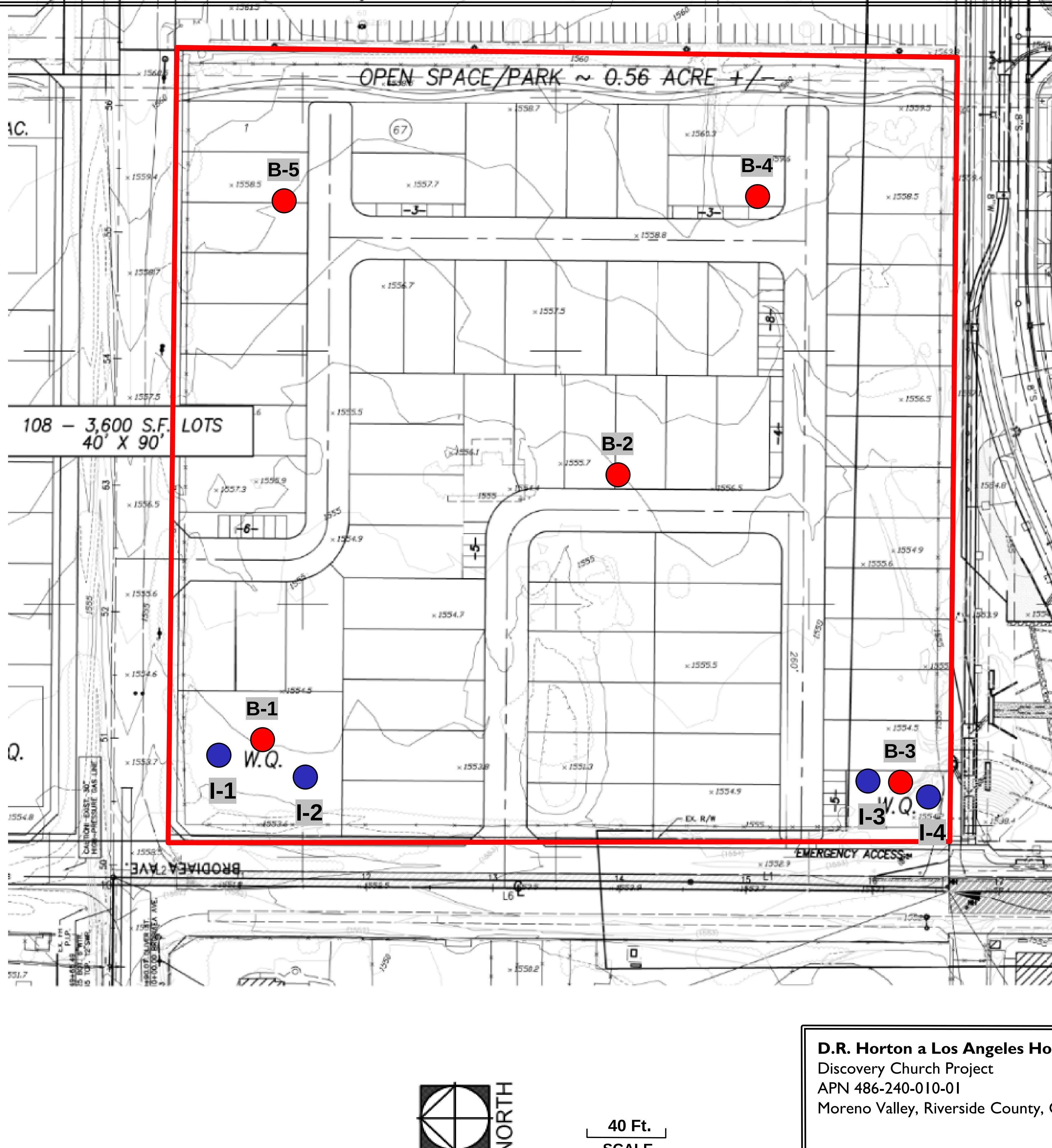
GeoTek Project No. 2867-CR

NORTH

Figure 1

Site Location Map

GEOTEK



Legend
(Locations are approximate)

B-5 - Exploratory Geotechnical Boring

I-3 - Percolation Test Boring

D.R. Horton a Los Angeles Holding Company
Discovery Church Project
APN 486-240-010-01
Moreno Valley, Riverside County, California

GeoTek Project No. 2867-CR

Figure 2

Exploration Location Map

GEOTEK

APPENDIX A

LOG OF EXPLORATORY BORINGS

Single-Family Residential Tract Development

Brodiaea Avenue and Oliver Street

Moreno Valley, Riverside County, California

Project No. 2867-CR

GEOTEK

A - FIELD TESTING AND SAMPLING PROCEDURES

The Modified Split-Barrel Sampler (Ring)
The Ring sampler is driven into the ground at various depths in accordance with ASTM D 3550 test procedures. The sampler, with an external diameter of 3.0 inches, is lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sampler is typically driven into the ground 12 or 18 inches with a 140-pound hammer free falling from a height of 30 inches. Blow counts are recorded for every 6 inches of penetration as indicated on the log of boring. The samples are removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

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 - BORING LOG LEGEND

The following abbreviations and symbols often appear in the classification and description of soil and rock on the log 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 boring

(Additional denotations and symbols are provided on the boring logs)

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	DR Horton	**DRILLER:**	2R Drilling	**LOGGED BY:**	D. Alvarez
PROJECT NAME:	Discovery Church Project 73 Lots	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Jeff/Cameron
PROJECT NO.:	2867-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley			**DATE:**	9/10/2021

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	Boring No.: B-1 — MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Disturbed soil/undocumented fill			
				SM	Silty f SAND, brown, dry, loose			
					Alluvium			
	Ring	9 11 11		SM	Silty f SAND, brown, medium dense	5.8	97.7	SR
5	Ring	7 10 14		SM-ML	Silty f SAND to sandy SILT, light brown, slightly moist, medium dense	6.5	99.5	
	Ring	14 28 42		SM	Silty f-m SAND, red-brown, slightly moist to moist, dense			
10	Ring	24 50/6			-Some f gravel, very dense @ 10 feet	4.1	120.9	
15	Ring	12 20 25			Silty f-m SAND, red-brown, moist, medium dense			
20	Ring	16 29 30			-Becomes dense @ 20 feet	6.7	124.0	
					BORING TERMINATED AT 21.5 FEET Boring backfilled with excavated soils. No groundwater encountered.			
25								
30								

END

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT: DR Horton | **DRILLER:** 2R Drilling | **LOGGED BY:** D. Alvarez

PROJECT NAME: Discovery Church Project 73 Lots | **DRILL METHOD:** Hollow Stem | **OPERATOR:** Jeff/Cameron

PROJECT NO.: 2867-CR | **HAMMER:** 140#/30" | **RIG TYPE:** CME 75

LOCATION: Moreno Valley | **DATE:** 9/10/2021

Boring No.: B-2

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
0					**Disturbed soil/undocumented fill**			
				SM	Silty f SAND, brown, dry, loose			
					Alluvium			
								MD, EI, SH, SR
		10		SM-ML	Silty f SAND to sandy SILT, brown, moist, medium dense to stiff	7.3	102.7	HC
		12						
		12						
5		7		SM	Silty f SAND, light brown, moist, medium dense	7.1	109.4	
		14						
		20						
		14			F-m SAND with some silt, red-brown, slightly moist, dense	3.3	123.6	
		26						
		34						
10		16			-Some f Gravel, brown, medium dense @ 10 feet			
		22						
		20						
15		10			-Becomes dense @ 15 feet	6.4	127.4	
		29						
		36						
20		15						
		30						
		40						
				SM-ML	Silty f SAND to sandy SILT, light brown, moist, medium dense to very stiff			
25		9				12.9		
		14						
		15						
30		11		ML	SILT with some sand, light brown, moist, hard			
		22						
		32						

END

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	DR Horton	**DRILLER:**	2R Drilling	**LOGGED BY:**	D. Alvarez
PROJECT NAME:	Discovery Church Project 73 Lots	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Jeff/Cameron
PROJECT NO.:	2867-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley			**DATE:**	9/10/2021

Boring No.: B-2 (Continued)

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
30								
35	SPT	7 12 19		ML	SILT, tan to light brown, moist to very moist, hard	19.3		
40	SPT	20 20 21		ML	SILT with some f sand, tan to light brown, moist to very moist, hard			
45	SPT	10 24 24		SM	Silty f-m SAND with some f gravel, brown, slightly moist, dense	4.9		
50	SPT	15 16 17		CL	CLAY with f sand, brown, very moist, hard			
					BORING TERMINATED AT 51.5 FEET			
					Boring backfilled with excavated soils. No groundwater encountered.			
55								
60								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits EI = Expansion Index SA = Sieve Analysis RV = R-Value Test

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	DR Horton	**DRILLER:**	2R Drilling	**LOGGED BY:**	D. Alvarez
PROJECT NAME:	Discovery Church Project 73 Lots	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Jeff/Cameron
PROJECT NO.:	2867-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley			**DATE:**	9/10/2021

Boring No.: B-3

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Disturbed soil/undocumented fill			
				SM	Silty f SAND, brown, dry, loose			
					Alluvium			
								EI
	Ring	4		SM-ML	Silty f SAND to sandy SILT, light brown, slightly moist, loose to medium stiff	4.3	98.4	SR
		4						
		3						
	Ring	4		ML	Sandy SILT, brown, moist, medium stiff	12.8	103.5	HC
5		3						
		4						
	Ring	4			-Becomes medium stiff to stiff	14.7	104.1	
		6						
		6						
	Ring	5			SILT with f sand, brown, medium stiff to stiff	20.5	104.8	HC
		5			Sandy SILT, gray, moist			
		7						
10								
	Ring	12		SC	F-m SAND with few clay, red-brown, moist, medium dense			
15		10						
		14						
					BORING TERMINATED AT 15.5 FEET			
					Boring backfilled with excavated soils.			
					No groundwater encountered.			
20								
25								
30								

END

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT: DR Horton
DRILLER: 2R Drilling
LOGGED BY: D. Alvarez
PROJECT NAME: Discovery Church Project 73 Lots
DRILL METHOD: Hollow Stem
OPERATOR: Jeff/Cameron
PROJECT NO.: 2867-CR
HAMMER: 140#/30"
RIG TYPE: CME 75
LOCATION: Moreno Valley
DATE: 9/10/2021

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	Boring No.: B-4 — MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Disturbed soil/undocumented fill			
				SM	Silty f SAND, light brown, dry, loose			
					Alluvium			
	Ring	5 5 6		SM-ML	Silty f SAND to sandy SILT, light brown, slightly moist, loose to medium stiff	1.9	102.7	HC, SR
5	Ring	4 5 8		SM	Silty f SAND, light brown, slightly moist, loose	8.9	104.0	
	Ring	10 7 8		SM-ML	Silty f SAND to sandy SILT, gray-brown, slightly moist, medium dense to stiff			
10	Ring	6 14 9		SM	Silty f-m SAND, moderate brown, moist, medium dense	19.2		
15	Ring	15 19 27		SC	Clayey f-m SAND, red-brown, moist to very moist, dense			
20	Ring	25 50/6		SM	Silty f-m SAND, red-brown, moist, very dense	12.5	124.1	
25					**BORING TERMINATED AT 21 FEET** Boring backfilled with excavated soils. No groundwater encountered.			
30								

END

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT: DR Horton
DRILLER: 2R Drilling
LOGGED BY: D. Alvarez
PROJECT NAME: Discovery Church Project 73 Lots
DRILL METHOD: Hollow Stem
OPERATOR: Jeff/Cameron
PROJECT NO.: 2867-CR
HAMMER: 140#/30"
RIG TYPE: CME 75
LOCATION: Moreno Valley
DATE: 9/10/2021

Boring No.: B-5

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Disturbed soil/undocumented fill			
				SM	Silty f SAND, brown, dry, loose			
					Alluvium			
	Ring	5 9 9		SM	Silty f SAND, light brown, slightly moist to moist, medium dense	7.3	102.5	SR
5	Ring	8 12 20			Same	7.0	109.1	
	Ring	24 50/6			Silty f-m SAND, brown, slightly moist, very dense			
	Ring	8 17 19			F-m SAND with some silt, brown, slightly moist, medium dense			
10								
15	Ring	12 27 30			Silty f-m SAND, brown, moist, dense	5.0	122.7	
					BORING TERMINATED AT 15.5 FEET			
					Boring backfilled with excavated soils. No groundwater encountered.			
20								
25								
30								

END

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	DR Horton	**DRILLER:**	2R Drilling	**LOGGED BY:**	D. Alvarez
PROJECT NAME:	Discovery Church Project 73 Lots	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Jeff/Cameron
PROJECT NO.:	2867-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley			**DATE:**	9/10/2021

Boring No.: I-1

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
				SM	**Disturbed soil/undocumented fill** Silty f SAND, brown, dry, loose			
				SM	**Alluvium** Silty f SAND, brown, dry			
				SM-ML	Silty f SAND to sandy SILT, brown, medium dense			
5					**BORING TERMINATED AT 5 FEET** Boring subsequently prepared for infiltration testing (pvc, pipe, filter sock, gravel) No groundwater encountered			
10								
15								
20								
25								
30								

END

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	DR Horton	**DRILLER:**	2R Drilling	**LOGGED BY:**	D. Alvarez
PROJECT NAME:	Discovery Church Project 73 Lots	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Jeff/Cameron
PROJECT NO.:	2867-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley			**DATE:**	9/10/2021

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	Boring No.: I-2 — MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
				SM	**Disturbed soil/undocumented fill** Silty f SAND, brown, dry, loose			
				SM	**Alluvium** Silty f SAND, brown, dry			
				SM-ML	Silty f SAND to sandy SILT, brown, medium dense			
5					**BORING TERMINATED AT 5 FEET** Boring subsequently prepared for infiltration testing (pvc, pipe, filter sock, gravel) No groundwater encountered			
10								
15								
20								
25								
30								

END **Sample type**: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	DR Horton	**DRILLER:**	2R Drilling	**LOGGED BY:**	D. Alvarez
PROJECT NAME:	Discovery Church Project 73 Lots	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Jeff/Cameron
PROJECT NO.:	2867-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley			**DATE:**	9/10/2021

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	Boring No.: I-3 MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					<u>Disturbed soil/undocumented fill</u>			
				SM	Silty f SAND, brown, dry, loose			
					<u>Alluvium</u>			
				SM	Silty f SAND, light brown, slightly moist, loose to medium stiff			
				ML	Sandy SILT, brown, moist, medium stiff			
5					**BORING TERMINATED AT 5 FEET**			
					Boring subsequently prepared for infiltration testing (pvc, pipe, filter sock, gravel)			
					No groundwater encountered			
10								
15								
20								
25								
30								

END

<u>Sample type</u>: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	DR Horton	**DRILLER:**	2R Drilling	**LOGGED BY:**	D. Alvarez
PROJECT NAME:	Discovery Church Project 73 Lots	**DRILL METHOD:**	Hollow Stem	**OPERATOR:**	Jeff/Cameron
PROJECT NO.:	2867-CR	**HAMMER:**	140#/30"	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley			**DATE:**	9/10/2021

Boring No.: I-4

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Disturbed soil/undocumented fill			
				SM	Silty f SAND, brown, dry, loose			
					Alluvium			
				SM	Silty f SAND, light brown, slightly moist, loose to medium stiff			
				ML	Sandy SILT, brown, moist, medium stiff			
5					**BORING TERMINATED AT 5 FEET**			
					Boring subsequently prepared for infiltration testing (pvc, pipe, filter sock, gravel)			
					No groundwater encountered			
10								
15								
20								
25								
30								

END

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

APPENDIX B

RESULTS OF LABORATORY TESTING

Single-Family Residential Tract Development

Brodiaea Avenue and Oliver Street

Moreno Valley, Riverside County, California

Project No. 2867-CR

GEOTEK

SUMMARY OF LABORATORY TESTING

Classification
Soils were classified visually in general accordance with the Unified Soil Classification System (ASTM Test Method D 2487). The soil classifications are shown on the logs of borings in Appendix A.

Collapse Test
Collapse tests were performed on selected samples of the site soils in general accordance with ASTM D 5333 test procedures. The results of this test are presented graphically in Appendix B.

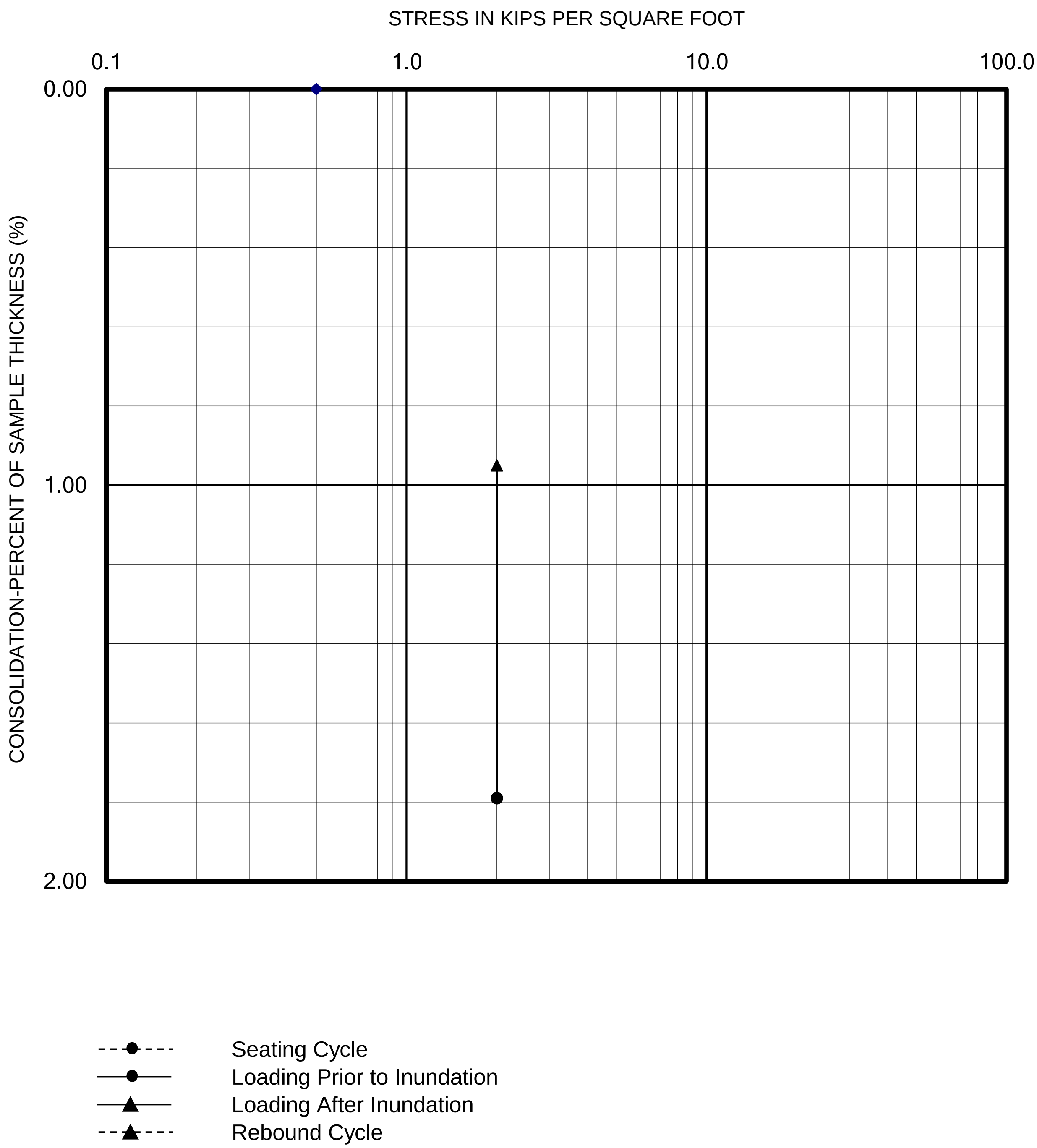
Direct Shear
Shear testing was performed in a direct shear machine of the strain-control type in general accordance with ASTM D 3080 test procedures. The rate of deformation was approximately 0.035 inch per minute. The sample was sheared under varying confining loads in order to determine the coulomb shear strength parameters, angle of internal friction and cohesion. The tests were performed on soil samples remolded to approximately 90 percent of maximum dry density as determined by ASTM D 1557 test procedures. The shear test results are presented in Appendix B.

Expansion Index
Expansion Index testing was performed two soil samples. Testing was performed in general accordance with ASTM Test Method D 4829. The results of the testing are provided below and in Appendix B.

Boring No.	Depth (ft.)	Description	Expansion Index	Classification
B-2	0-5	Sandy Silt	1	Very Low
B-12	0-5	Sandy Silt	2	Very Low

In-Situ Moisture and Density
The natural water content of sampled soils was determined in general accordance with ASTM D 2216 test procedures on samples of the materials recovered from the subsurface exploration. In addition, in-place dry density of the sampled soils was determined in general accordance with ASTM D 2937 test procedures on relatively undisturbed samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths in Appendix A.

Moisture-Density Relationship
Laboratory testing was performed on two samples collected during the subsurface exploration. The laboratory maximum dry density and optimum moisture content for the soil type was determined in general accordance with ASTM D 1557 test procedures. The results are presented in Appendix B.



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

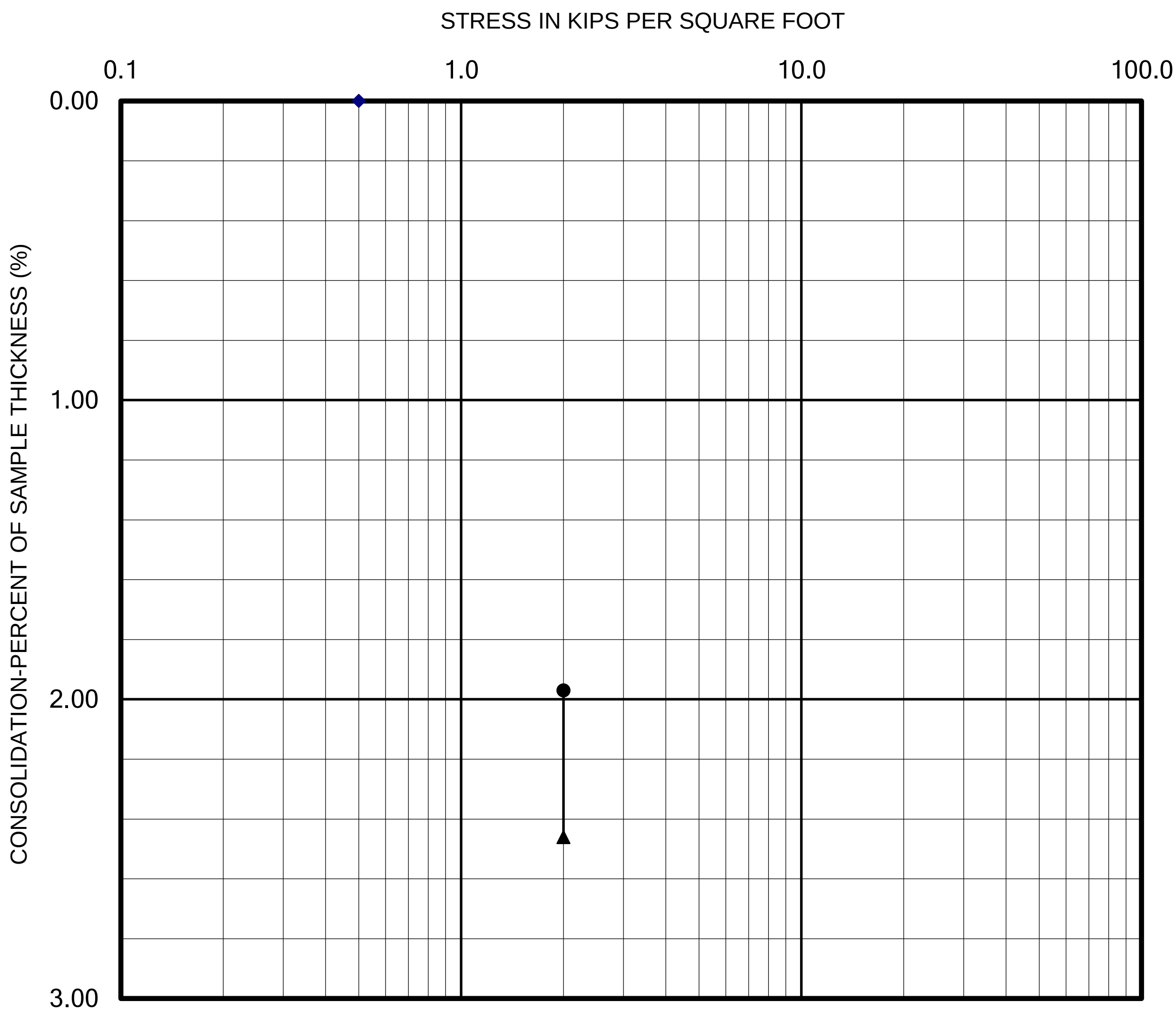
GEOTEK

COLLAPSE REPORT

Sample: B-2 @ 3 feet

Plate B-1

CHECKED BY: RJ	Lab: Corona
PROJECT NO.: 2867-CR	Date: 9-17-21



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

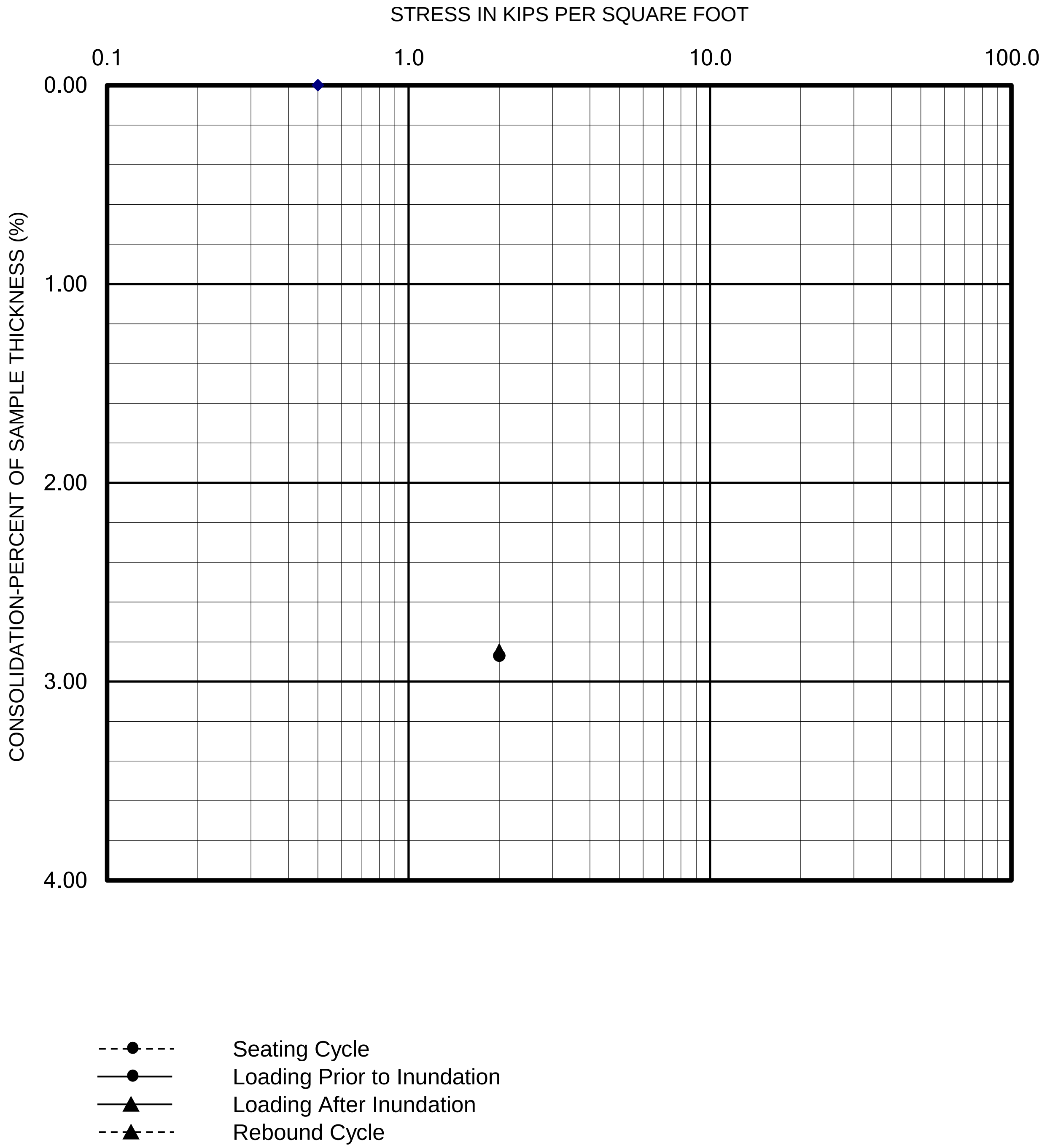
GEOTEK

COLLAPSE REPORT

Sample: B-4 @ 3 feet

Plate B-2

CHECKED BY: RJ	Lab: Corona
PROJECT NO.: 2867-CR	Date: 9-17-21



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

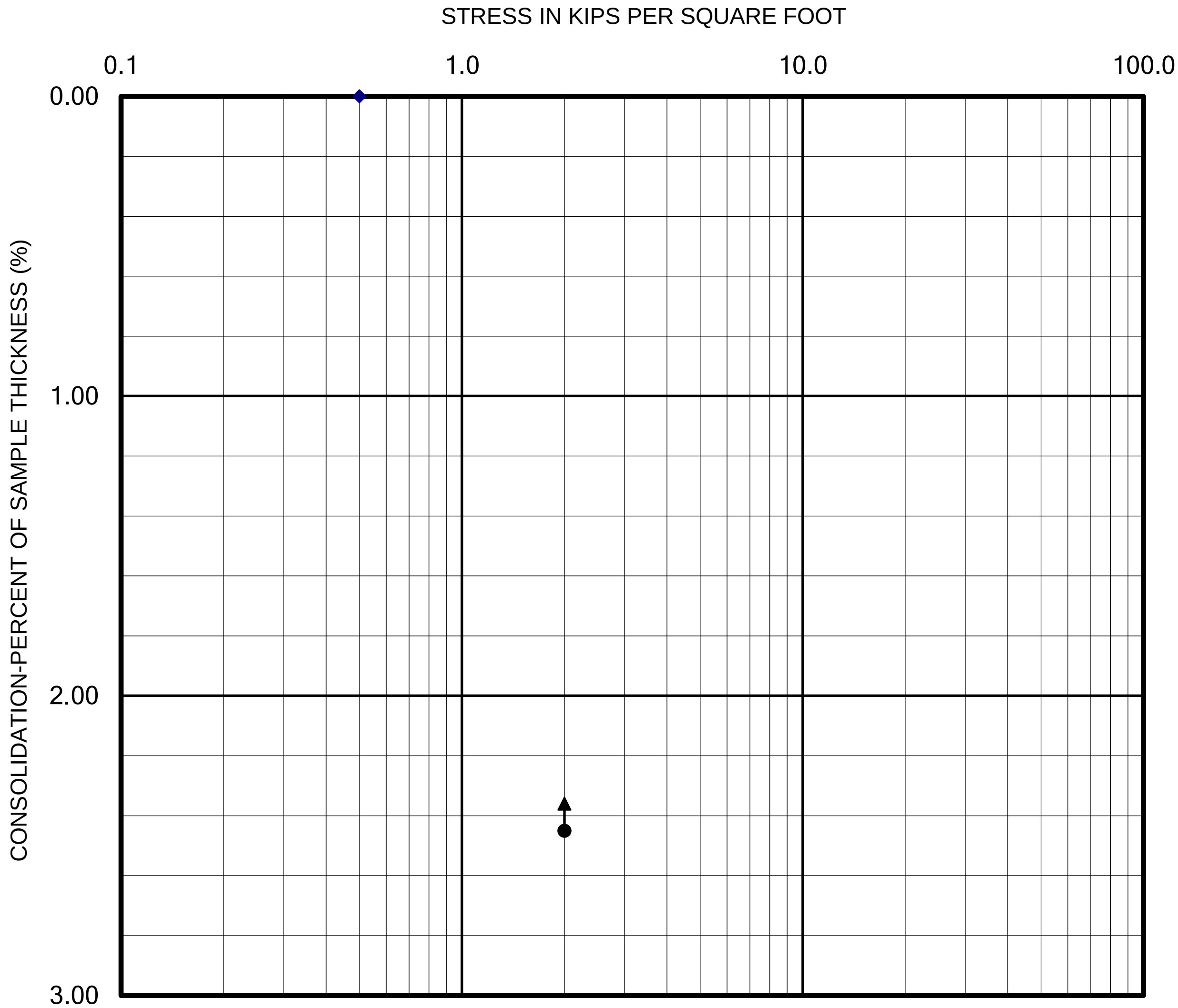
GEOTEK

COLLAPSE REPORT

Sample: B-3 @ 4 feet

Plate B-3

CHECKED BY: RJ	Lab: Corona
PROJECT NO.: 2867-CR	Date: 9-17-21



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

COLLAPSE REPORT

Sample: B-3 @ 8 feet

Plate B-4

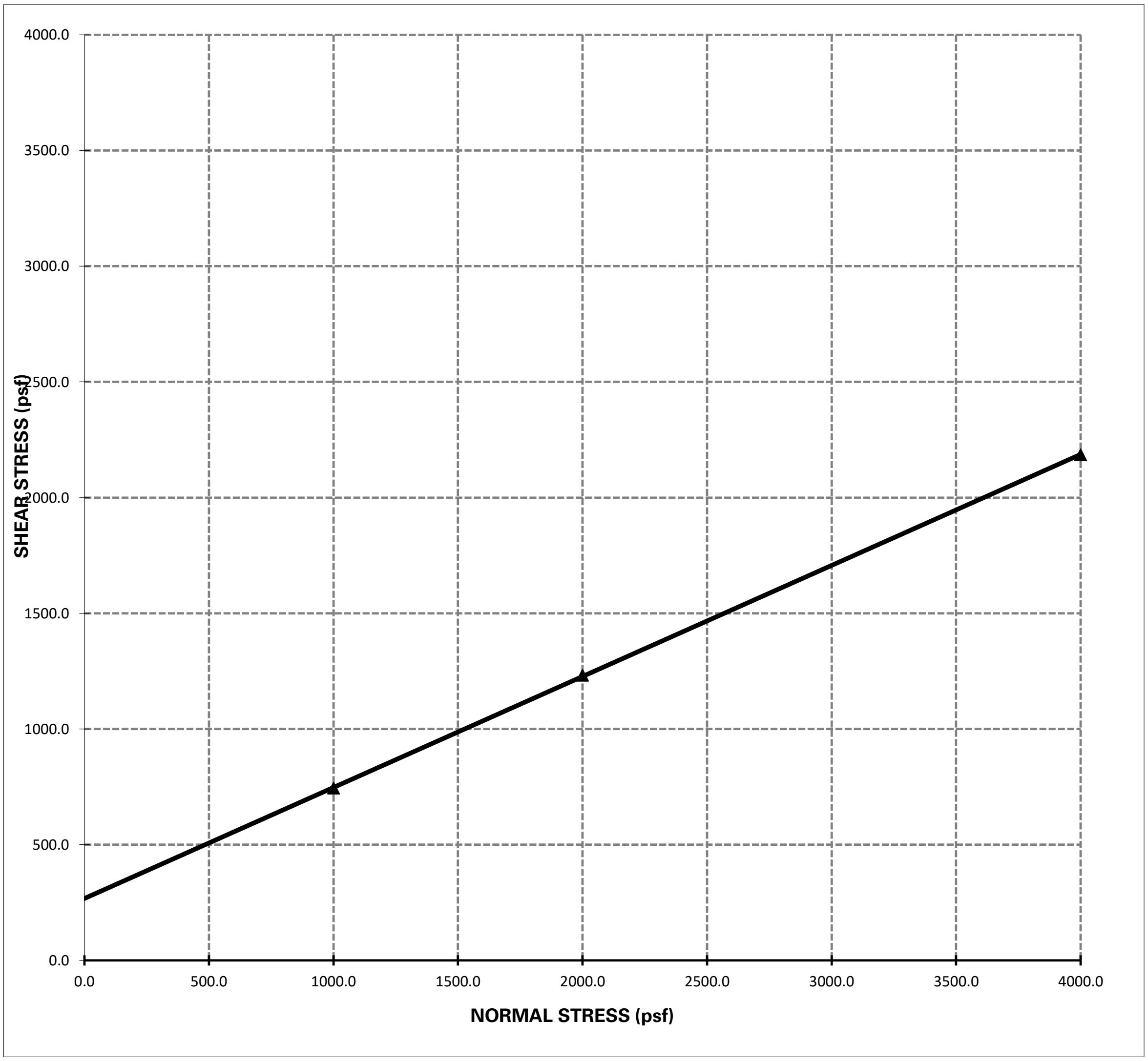
CHECKED BY: RJ	Lab: Corona
PROJECT NO.: 2867-CR	Date: 9-17-21

GEOTEK

DIRECT SHEAR TEST

Project Name: DR Horton
Project Number: 2867-CR
Sample Location: B2 @ 0-5'
Date Tested: 9/21/2020

PEAK VALUE



Shear Strength: $\Phi =$ **26** $^{\circ}$ **C = 268 psf**

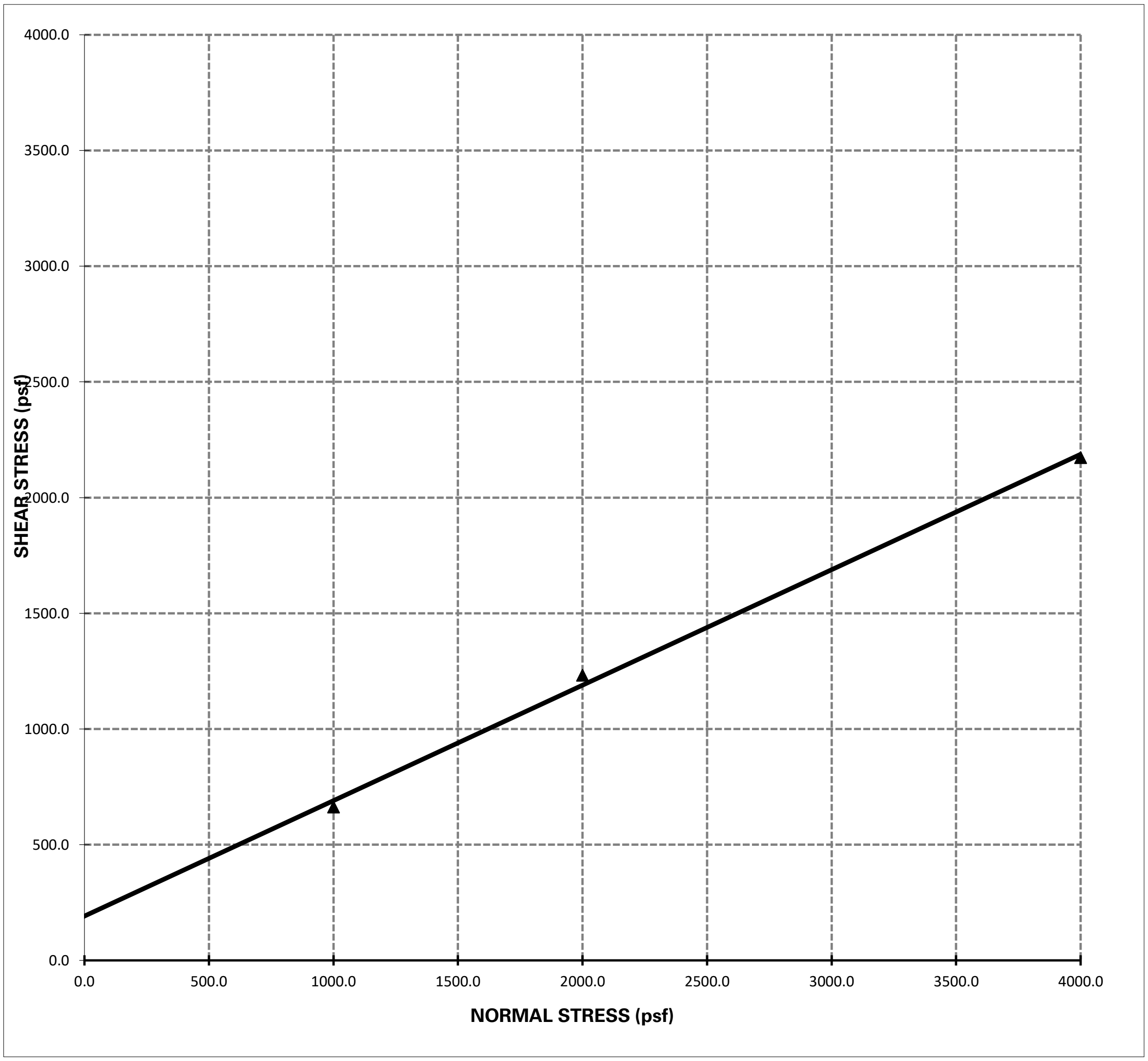
Notes: 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.

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:	DR Horton	***Sample Location:***	B2 @ 0-5'
Project Number:	2867-CR	***Date Tested:***	9/21/2020



Shear Strength: **Φ = 27 °** **C = 192 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.

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client: DR Horton
Project Number: 2867-CR
Project Location: Discovery Church Project 73 Lots Moreno Valley

Tested/ Checked By: CD Lab No Corona
Date Tested: 9/17/2021
Sample Source: B2 @ 0-5'
Sample Description:

Ring #:______ Ring Dia. : 4.01" Ring Ht.:1"

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	783.8
B	Weight of ring (gm)	362.8
C	Net weight of sample (gm)	**421.0**
D	Wet Density, lb / ft3 (C*0.3016)	**127.0**
E	Dry Density, lb / ft3 (D/1.F)	**117.8**

SATURATION DETERMINATION

F	Moisture Content, %	7.8
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
I	% Saturation	**48.9**

READINGS

DATE	TIME	READING	
9/17/2021		0.7240	Initial
9/17/2021		0.7250	10 min/Dry
9/18/2021		0.7600	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
817.4	**15.8**

EXPANSION INDEX = 35

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client: DR Horton
Project Number: 2867-CR
Project Location: Discovery Church Project 73 Lots Moreno Valley

Tested/ Checked By: CD Lab No Corona
Date Tested: 9/20/2021
Sample Source: B3 @ 0-10'
Sample Description:

Ring #:______ Ring Dia. : 4.01" Ring Ht.:1"

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	776.6
B	Weight of ring (gm)	366.5
C	Net weight of sample (gm)	**410.1**
D	Wet Density, lb / ft3 (C*0.3016)	**123.7**
E	Dry Density, lb / ft3 (D/1.F)	**113.1**

SATURATION DETERMINATION

F	Moisture Content, %	9.4
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
I	% Saturation	**51.8**

READINGS

DATE	TIME	READING	
9/20/2021		0.7430	Initial
9/20/2021		0.7420	10 min/Dry
9/21/2021		0.7670	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
803.4	**15.9**

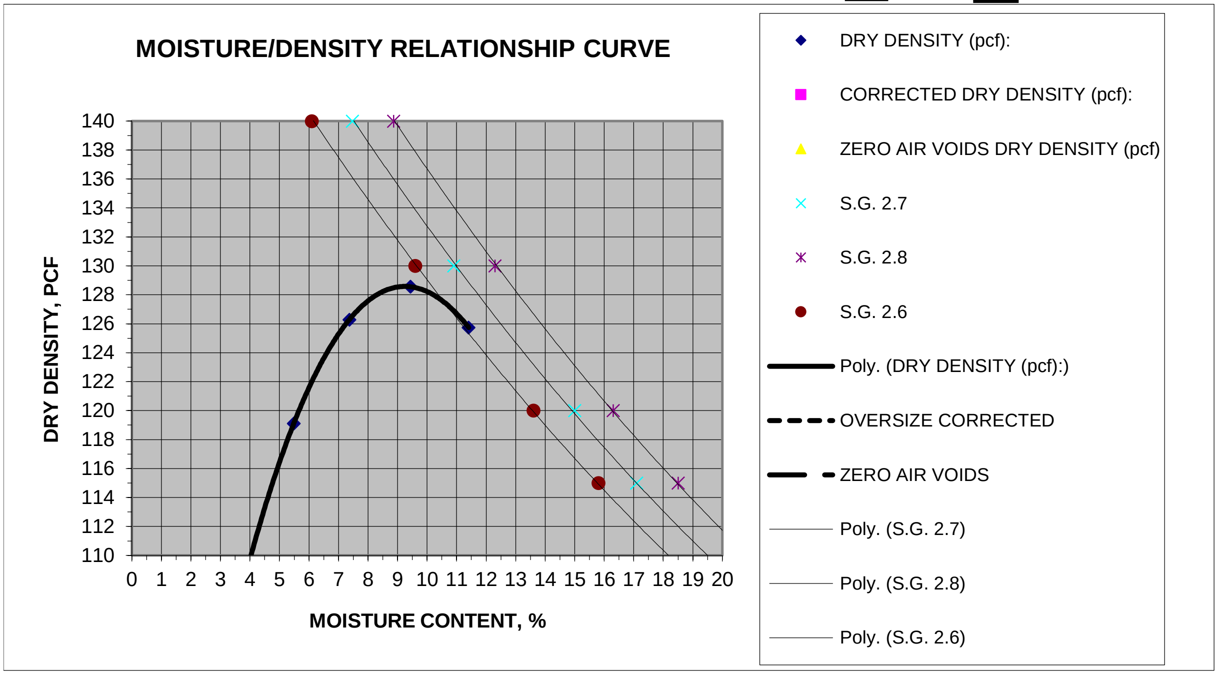
EXPANSION INDEX = 25

MOISTURE/DENSITY RELATIONSHIP

Client:	DR Horton	**Job No.:**	2867-CR
Project:	Discovery Church Project 73	**Lab No.:**	Corona
Location:	Moreno Vallley		
Material Type:	Brown Silty Sand		
Material Supplier:	-		
Material Source:	-		
Sample Location:	B2 @ 0-5'		
	-		
Sampled By:	DA	**Date Sampled:**	9/14/2021
Received By:	RJ	**Date Received:**	9/14/2021
Tested By:	AD	**Date Tested:**	9/20/2021
Reviewed By:	RJ	**Date Reviewed:**	9/20/2021

Test Procedure: ASTM D1557 **Method:** A

Oversized Material (%): 0.3 **Correction Required:** yes x no



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	129.0	**@ Optimum Moisture, %**	9.5
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: ________

APPENDIX C

PERCOLATION DATA SHEETS & PORCHET CALCULATIONS

Single-Family Residential Tract Development

Brodiaea Avenue and Oliver Street

Moreno Valley, Riverside County, California

Project No. 2867-CR

GEOTEK

Client:	**Griffin Residential**
Project:	**2020 East 1st Street, Santa Ana**
Project No:	**2881-CR**
Date:	**9/14/2021**

Boring No. **I-1**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	10
Final Depth to Water, D_F =	588.75
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	580
Total Test Hole Depth, D_T =	600

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

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	11.25
$\Delta H = \Delta D = H_O - H_F$ =	8.75
Havg = $(H_O+H_F)/2$ =	15.625

I_t = 5.96 **Inches pe**

GEOTEK

Client:	**Griffin Residential**
Project:	**2020 East 1st Street, Santa Ana**
Project No:	**2881-CR**
Date:	**9/14/2021**

Boring No. **I-2**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	10
Final Depth to Water, D_F =	588.5
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	580
Total Test Hole Depth, D_T =	600

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

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	11.5
$\Delta H = \Delta D = H_O - H_F$ =	8.5
Havg = $(H_O+H_F)/2$ =	15.75

I_t = 5.75 **Inches pe**

GEOTEK

Client:	**Griffin Residential**
Project:	**2020 East 1st Street, Santa Ana**
Project No:	**2881-CR**
Date:	**9/14/2021**

Boring No. **I-3**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	10
Final Depth to Water, D_F =	588.5
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	580
Total Test Hole Depth, D_T =	600

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

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	11.5
$\Delta H = \Delta D = H_O - H_F$ =	8.5
Havg = $(H_O+H_F)/2$ =	15.75

I_t = 5.75 **Inches pe**

GEOTEK

Client:	**D. R. Horton**
Project:	**Discovery Church**
Project No:	**2867-CR**
Date:	**9/13/2021**

Boring No. **I-4**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	10
Final Depth to Water, D_F =	38.5
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	36
Total Test Hole Depth, D_T =	60

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

$H_O = D_T - D_O$ =	24
$H_F = D_T - D_F$ =	21.5
$\Delta H = \Delta D = H_O - H_F$ =	2.5
Havg = $(H_O+H_F)/2$ =	22.75

I_t = 1.21 **Inches pe**

GEOTEK

Percolation Test Data Sheet

Project:	Discovery Church	Project No:	2867-OR	Date:	9/13
Test Hole No:	I-1	Tested By:	D. Alvarez		
Depth of Test Hole, D_T:	60	USCS Soil Classification:			

Test Hole Dimensions (inches)			Length	Width
Diameter (if round)=	8	Sides (if rectangular)=		

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	8:29	8:54	25	36	41.5	5.5	N
2	8:54	9:19	25	36	40.75	4.75	N

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

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	9:19	9:49	30	36	41.25	5.25	
2	9:49	10:19	30	36	41.25	5.25	
3	10:19	10:49	30	36	41.0	5.0	
4	10:49	11:19	30	36	40.75	4.75	
5	11:19	11:49	30	36	40.75	4.75	
6	11:49	12:19	30	36	40.75	4.75	
7	12:19	12:49	30	36	40.5	4.5	
8	12:49	13:19	30	36	40.5	4.5	
9	13:19	13:49	30	36	40.5	4.5	
10	13:49	14:19	30	36	40.5	4.5	
11	14:19	14:49	30	36	40.5	4.5	
12	14:49	15:19	30	36	40.5	4.5	
13							
14							
15							

COMMENTS:

Percolation Test Data Sheet

Project:	Discovery Church	Project No:	2867-02	Date:	9/13
Test Hole No:	I-2	Tested By:	D. Alvarez		
Depth of Test Hole, D_T:	60	USCS Soil Classification:			
Test Hole Dimensions (inches)			Length	Width	
Diameter (if round)=	8	Sides (if rectangular)=			

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	8:32	8:57	25	36	41.25	5.25	N
2	8:57	9:22	25	36	41.25	5.25	N

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

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	9:22	9:52	30	36	41.25	5.25	
2	9:52	10:22			41.25	5.25	
3	10:22	10:52			41.25	5.25	
4	10:52	11:22			40.75	4.75	
5	11:22	11:52			40.5	4.5	
6	11:52	12:22			40.5	4.5	
7	12:22	12:52			40.25	4.25	
8	12:52	13:22			40.0	4.0	
9	13:22	13:52			40.0	4.0	
10	13:52	14:22			40.0	4.0	
11	14:22	14:52			40.0	4.0	
12	14:52	15:22			40.0	4.0	
13							
14							
15							

COMMENTS:

Percolation Test Data Sheet

Project:	Discovery Church	Project No:	2867-CR	Date:	9/13
Test Hole No:	I-3	Tested By:	D. Alvarez		
Depth of Test Hole, D_T:	60	USCS Soil Classification:			

Test Hole Dimensions (inches)				Length	Width	
Diameter (if round)=	8	Sides (if rectangular)=				

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	9:26	9:51	25	36	49.25	13.25	y
2	9:51	10:16	25	36	44.75	8.75	y

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

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_0 Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	10:16	10:26	10	36	40.25	4.25	
2	10:26	10:36			39.5	3.5	
3	10:36	10:46			39.5	3.5	
4	10:46	10:56			39.5	3.5	
5	10:56	11:06			39.5	3.5	
6	11:06	11:16			39.5	3.5	
7							
8							
9							
10							
11							
12							
13							
14							
15							

COMMENTS:

Percolation Test Data Sheet

Project:	Discovery Church	Project No: 2867-CR	Date: 9/13
Test Hole No:	I-4	Tested By: D. Alvarez	
Depth of Test Hole, D_T:	60	USCS Soil Classification:	

Test Hole Dimensions (inches)			Length	Width
Diameter (if round)=	8	Sides (if rectangular)=		

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	9:28	9:53	25	36	45	9.0	y
2	10:18	10:43	25	36	42.25	6.25	y

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

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	ΔD Change in Water Level (in.)	Percolation Rate (min./in.)
1	10:43	10:53	10	36	39	3.0	
2	10:53	11:03			39	3.0	
3	11:03	11:13			38.5	2.5	
4	11:13	11:23			38.5	2.5	
5	11:23	11:33			38.5	2.5	
6	11:33	11:43			38.5	2.5	
7	~~11:43~~						
8							
9							
10							
11							
12							
13							
14							
15							

COMMENTS:

APPENDIX D

SEISMIC SETTLEMENT ANALYSIS

Single-Family Residential Tract Development
Brodiaea Avenue and Oliver Street
Moreno Valley, Riverside County, California
Project No. 2867-CR

GEOTEK

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                                LIQUEFACTION ANALYSIS CALCULATION DETAILS

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        Input File Name: UNTITLED
        Title:  Discovery Church 73 Lots Project
        Subtitle:  Seismic Settlement

 Input Data:
        Surface Elev.=
        Hole No.=B-2
        Depth of Hole=50.00 ft
        Water Table during Earthquake= 50.00 ft
        Water Table during In-Situ Testing= 50.00 ft
        Max. Acceleration=0.88 g
        Earthquake Magnitude=7.00
        No-Liquefiable Soils:   CL, OL are Non-Liq. Soil
        1. SPT or BPT Calculation.
        2. Settlement Analysis Method: Ishihara / Yoshimine
        3. Fines Correction for Liquefaction: Idriss/Seed
        4. Fine Correction for Settlement: During Liquefaction*
        5. Settlement Calculation in: All zones*
        6. Hammer Energy Ratio,                                  Ce = 1.25
        7. Borehole Diameter,                                       Cb= 1
        8. Sampling Method,                                        Cs= 1
        9. User request factor of safety (apply to CSR) ,   User= 1
           Plot one CSR curve (fs1=User)
        10. Average two input data between two Depths: Yes*
        * Recommended Options

        In-Situ Test Data:
        Depth   SPT     Gamma   Fines
        ft              pcf     %
        ____________________________________
        1.00    50.00   127.00  NoLiq
        3.00    50.00   127.00  NoLiq
        5.00    50.00   127.00  NoLiq
        7.50    38.00   127.00  25.00
        10.00   26.00   127.00  25.00
        15.00   41.00   135.00  25.00
        20.00   44.00   135.00  25.00
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25.00	29.00	127.00	50.00
30.00	54.00	124.00	55.00
35.00	31.00	124.00	55.00
40.00	41.00	124.00	55.00
45.00	48.00	135.00	25.00
50.00	33.00	130.00	NoLiq

Output Results:
Calculation segment, dz=0.050 ft
User defined Print Interval, dp=1.00 ft

Peak Ground Acceleration (PGA), a_max = 0.88g

CSR Calculation:

Depth ft	gamma pcf	sigma atm	gamma' pcf	sigma' atm	rd	mZ g	a(z) g	CSR	x	fs1 =CSRfs
1.00	127.00	0.060	127.00	0.060	1.00	0.000	0.885	0.57	1.00	0.57
2.00	127.00	0.120	127.00	0.120	1.00	0.000	0.885	0.57	1.00	0.57
3.00	127.00	0.180	127.00	0.180	0.99	0.000	0.885	0.57	1.00	0.57
4.00	127.00	0.240	127.00	0.240	0.99	0.000	0.885	0.57	1.00	0.57
5.00	127.00	0.300	127.00	0.300	0.99	0.000	0.885	0.57	1.00	0.57
6.00	127.00	0.360	127.00	0.360	0.99	0.000	0.885	0.57	1.00	0.57
7.00	127.00	0.420	127.00	0.420	0.98	0.000	0.885	0.57	1.00	0.57
8.00	127.00	0.480	127.00	0.480	0.98	0.000	0.885	0.56	1.00	0.56
9.00	127.00	0.540	127.00	0.540	0.98	0.000	0.885	0.56	1.00	0.56
10.00	127.00	0.600	127.00	0.600	0.98	0.000	0.885	0.56	1.00	0.56
11.00	128.60	0.661	128.60	0.661	0.97	0.000	0.885	0.56	1.00	0.56
12.00	130.20	0.722	130.20	0.722	0.97	0.000	0.885	0.56	1.00	0.56
13.00	131.80	0.784	131.80	0.784	0.97	0.000	0.885	0.56	1.00	0.56
14.00	133.40	0.846	133.40	0.846	0.97	0.000	0.885	0.56	1.00	0.56

15.00	135.00	0.910	135.00	0.910	0.97	0.000	0.885	0.56	1.00	0.56
16.00	135.00	0.973	135.00	0.973	0.96	0.000	0.885	0.55	1.00	0.55
17.00	135.00	1.037	135.00	1.037	0.96	0.000	0.885	0.55	1.00	0.55
18.00	135.00	1.101	135.00	1.101	0.96	0.000	0.885	0.55	1.00	0.55
19.00	135.00	1.165	135.00	1.165	0.96	0.000	0.885	0.55	1.00	0.55
20.00	135.00	1.229	135.00	1.229	0.95	0.000	0.885	0.55	1.00	0.55
21.00	133.40	1.292	133.40	1.292	0.95	0.000	0.885	0.55	1.00	0.55
22.00	131.80	1.355	131.80	1.355	0.95	0.000	0.885	0.55	1.00	0.55
23.00	130.20	1.417	130.20	1.417	0.95	0.000	0.885	0.54	1.00	0.54
24.00	128.60	1.478	128.60	1.478	0.94	0.000	0.885	0.54	1.00	0.54
25.00	127.00	1.538	127.00	1.538	0.94	0.000	0.885	0.54	1.00	0.54
26.00	126.40	1.598	126.40	1.598	0.94	0.000	0.885	0.54	1.00	0.54
27.00	125.80	1.658	125.80	1.658	0.94	0.000	0.885	0.54	1.00	0.54
28.00	125.20	1.717	125.20	1.717	0.93	0.000	0.885	0.54	1.00	0.54
29.00	124.60	1.776	124.60	1.776	0.93	0.000	0.885	0.54	1.00	0.54
30.00	124.00	1.835	124.00	1.835	0.93	0.000	0.885	0.54	1.00	0.54
31.00	124.00	1.893	124.00	1.893	0.92	0.000	0.885	0.53	1.00	0.53
32.00	124.00	1.952	124.00	1.952	0.91	0.000	0.885	0.53	1.00	0.53
33.00	124.00	2.010	124.00	2.010	0.91	0.000	0.885	0.52	1.00	0.52
34.00	124.00	2.069	124.00	2.069	0.90	0.000	0.885	0.52	1.00	0.52
35.00	124.00	2.128	124.00	2.128	0.89	0.000	0.885	0.51	1.00	0.51
36.00	124.00	2.186	124.00	2.186	0.88	0.000	0.885	0.51	1.00	0.51
37.00	124.00	2.245	124.00	2.245	0.87	0.000	0.885	0.50	1.00	0.50
38.00	124.00	2.303	124.00	2.303	0.86	0.000	0.885	0.50	1.00	0.50
39.00	124.00	2.362	124.00	2.362	0.86	0.000	0.885	0.49	1.00	0.49

40.00	124.00	2.421	124.00	2.421	0.85	0.000	0.885	0.49	1.00	0.49
41.00	126.20	2.480	126.20	2.480	0.84	0.000	0.885	0.48	1.00	0.48
42.00	128.40	2.540	128.40	2.540	0.83	0.000	0.885	0.48	1.00	0.48
43.00	130.60	2.601	130.60	2.601	0.82	0.000	0.885	0.47	1.00	0.47
44.00	132.80	2.663	132.80	2.663	0.82	0.000	0.885	0.47	1.00	0.47
45.00	135.00	2.726	135.00	2.726	0.81	0.000	0.885	0.46	1.00	0.46
46.00	134.00	2.790	134.00	2.790	0.80	0.000	0.885	0.46	1.00	0.46
47.00	133.00	2.853	133.00	2.853	0.79	0.000	0.885	0.46	1.00	0.46
48.00	132.00	2.916	132.00	2.916	0.78	0.000	0.885	0.45	1.00	0.45
49.00	131.00	2.978	131.00	2.978	0.78	0.000	0.885	0.45	1.00	0.45
50.00	130.00	3.040	130.00	3.040	0.77	0.000	0.885	0.44	1.00	0.44

CSR is based on water table at 50.00 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
1.00	50.00	1.25	0.75	0.060	1.70	79.69	NoLiq	20.94	100.63	0.50
2.00	50.00	1.25	0.75	0.120	1.70	79.69	NoLiq	20.94	100.63	0.50
3.00	50.00	1.25	0.75	0.180	1.70	79.69	NoLiq	20.94	100.63	0.50
4.00	50.00	1.25	0.75	0.240	1.70	79.69	NoLiq	20.94	100.63	0.50
5.00	50.00	1.25	0.75	0.300	1.70	79.69	NoLiq	20.94	100.62	0.50
6.00	45.20	1.25	0.75	0.360	1.67	70.62	70.60	19.12	89.74	0.50
7.00	40.40	1.25	0.75	0.420	1.54	58.44	40.20	16.69	75.12	0.50
8.00	35.60	1.25	0.75	0.480	1.44	48.17	25.00	9.83	58.00	0.50

9.00	30.80	1.25	0.85	0.540	1.36	44.53	25.00	9.41	53.94	0.50
10.00	26.00	1.25	0.85	0.600	1.29	35.66	25.00	8.39	44.05	0.50
11.00	29.00	1.25	0.85	0.661	1.23	37.91	25.00	8.65	46.56	0.50
12.00	32.00	1.25	0.85	0.722	1.18	40.02	25.00	8.89	48.92	0.50
13.00	35.00	1.25	0.85	0.784	1.13	42.01	25.00	9.12	51.13	0.50
14.00	38.00	1.25	0.85	0.846	1.09	43.89	25.00	9.34	53.23	0.50
15.00	41.00	1.25	0.95	0.910	1.05	51.05	25.00	10.16	61.21	0.50
16.00	41.60	1.25	0.95	0.973	1.01	50.07	25.00	10.05	60.12	0.50
17.00	42.20	1.25	0.95	1.037	0.98	49.21	25.00	9.95	59.15	0.50
18.00	42.80	1.25	0.95	1.101	0.95	48.44	25.00	9.86	58.30	0.50
19.00	43.40	1.25	0.95	1.165	0.93	47.75	25.00	9.78	57.53	0.50
20.00	44.00	1.25	0.95	1.229	0.90	47.14	25.00	9.71	56.85	0.50
21.00	41.00	1.25	0.95	1.292	0.88	42.83	30.00	11.32	54.15	0.50
22.00	38.00	1.25	0.95	1.355	0.86	38.77	35.00	12.62	51.39	0.50
23.00	35.00	1.25	0.95	1.417	0.84	34.92	40.00	11.98	46.91	0.50
24.00	32.00	1.25	0.95	1.478	0.82	31.26	45.00	11.25	42.51	0.50
25.00	29.00	1.25	0.95	1.538	0.81	27.77	50.00	10.55	38.32	0.50
26.00	34.00	1.25	0.95	1.598	0.79	31.94	51.00	11.39	43.33	0.50
27.00	39.00	1.25	0.95	1.658	0.78	35.97	52.00	12.19	48.16	0.50
28.00	44.00	1.25	1.00	1.717	0.76	41.97	53.00	13.39	55.37	0.50
29.00	49.00	1.25	1.00	1.776	0.75	45.96	54.00	14.19	60.15	0.50
30.00	54.00	1.25	1.00	1.835	0.74	49.83	55.00	14.97	64.80	0.50
31.00	49.40	1.25	1.00	1.893	0.73	44.88	55.00	13.98	58.85	0.50
32.00	44.80	1.25	1.00	1.952	0.72	40.08	55.00	13.02	53.10	0.50
33.00	40.20	1.25	1.00	2.010	0.71	35.44	55.00	12.09	47.53	0.50

34.00	35.60	1.25	1.00	2.069	0.70	30.94	55.00	11.19	42.12	0.50
35.00	31.00	1.25	1.00	2.128	0.69	26.57	55.00	10.31	36.88	0.50
36.00	33.00	1.25	1.00	2.186	0.68	27.90	55.00	10.58	38.48	0.50
37.00	35.00	1.25	1.00	2.245	0.67	29.20	55.00	10.84	40.04	0.50
38.00	37.00	1.25	1.00	2.303	0.66	30.47	55.00	11.09	41.57	0.50
39.00	39.00	1.25	1.00	2.362	0.65	31.72	55.00	11.34	43.06	0.50
40.00	41.00	1.25	1.00	2.421	0.64	32.94	55.00	11.59	44.53	0.50
41.00	42.40	1.25	1.00	2.480	0.64	33.66	49.00	11.73	45.39	0.50
42.00	43.80	1.25	1.00	2.540	0.63	34.35	43.00	11.87	46.22	0.50
43.00	45.20	1.25	1.00	2.601	0.62	35.03	37.00	12.01	47.04	0.50
44.00	46.60	1.25	1.00	2.663	0.61	35.69	31.00	10.57	46.27	0.50
45.00	48.00	1.25	1.00	2.726	0.61	36.34	25.00	8.47	44.81	0.50
46.00	45.00	1.25	1.00	2.790	0.60	33.68	40.19	11.74	45.41	0.50
47.00	42.00	1.25	1.00	2.853	0.59	31.08	55.39	11.22	42.30	0.50
48.00	39.00	1.25	1.00	2.916	0.59	28.55	70.59	10.71	39.26	0.50
49.00	36.00	1.25	1.00	2.978	0.58	26.08	85.79	10.22	36.29	0.50
50.00	33.00	1.25	1.00	3.040	0.57	23.66	NoLiq	9.73	33.39	0.50

CRR is based on water table at 50.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 7.00:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
1.00	0.04	0.50	1.00	0.50	1.19	2.00	0.57	5.00 ^
2.00	0.08	0.50	1.00	0.50	1.19	2.00	0.57	5.00 ^
3.00	0.12	0.50	1.00	0.50	1.19	2.00	0.57	5.00 ^
4.00	0.16	0.50	1.00	0.50	1.19	2.00	0.57	5.00 ^
5.00	0.20	0.50	1.00	0.50	1.19	0.60	0.57	5.00
6.00	0.23	0.50	1.00	0.50	1.19	0.60	0.57	5.00

7.00	0.27	0.50	1.00	0.50	1.19	0.60	0.57	5.00
8.00	0.31	0.50	1.00	0.50	1.19	0.60	0.56	5.00
9.00	0.35	0.50	1.00	0.50	1.19	0.60	0.56	5.00
10.00	0.39	0.50	1.00	0.50	1.19	0.60	0.56	5.00
11.00	0.43	0.50	1.00	0.50	1.19	0.60	0.56	5.00
12.00	0.47	0.50	1.00	0.50	1.19	0.60	0.56	5.00
13.00	0.51	0.50	1.00	0.50	1.19	0.60	0.56	5.00
14.00	0.55	0.50	1.00	0.50	1.19	0.60	0.56	5.00
15.00	0.59	0.50	1.00	0.50	1.19	0.60	0.56	5.00
16.00	0.63	0.50	1.00	0.50	1.19	0.60	0.55	5.00
17.00	0.67	0.50	1.00	0.50	1.19	0.60	0.55	5.00
18.00	0.72	0.50	1.00	0.50	1.19	0.60	0.55	5.00
19.00	0.76	0.50	1.00	0.50	1.19	0.60	0.55	5.00
20.00	0.80	0.50	1.00	0.50	1.19	0.60	0.55	5.00
21.00	0.84	0.50	1.00	0.50	1.19	0.60	0.55	5.00
22.00	0.88	0.50	1.00	0.50	1.19	0.60	0.55	5.00
23.00	0.92	0.50	1.00	0.50	1.19	0.60	0.54	5.00
24.00	0.96	0.50	1.00	0.50	1.19	0.60	0.54	5.00
25.00	1.00	0.50	1.00	0.50	1.19	0.60	0.54	5.00
26.00	1.04	0.50	1.00	0.50	1.19	0.60	0.54	5.00
27.00	1.08	0.50	0.99	0.50	1.19	0.59	0.54	5.00
28.00	1.12	0.50	0.99	0.49	1.19	0.59	0.54	5.00
29.00	1.15	0.50	0.98	0.49	1.19	0.59	0.54	5.00
30.00	1.19	0.50	0.98	0.49	1.19	0.58	0.54	5.00
31.00	1.23	0.50	0.97	0.49	1.19	0.58	0.53	5.00
32.00	1.27	0.50	0.97	0.48	1.19	0.58	0.53	5.00
33.00	1.31	0.50	0.96	0.48	1.19	0.57	0.52	5.00
34.00	1.34	0.50	0.95	0.48	1.19	0.57	0.52	5.00
35.00	1.38	0.50	0.95	0.47	1.19	0.57	0.51	5.00
36.00	1.42	0.50	0.94	0.47	1.19	0.56	0.51	5.00
37.00	1.46	0.50	0.94	0.47	1.19	0.56	0.50	5.00
38.00	1.50	0.50	0.93	0.47	1.19	0.56	0.50	5.00
39.00	1.54	0.50	0.93	0.46	1.19	0.55	0.49	5.00
40.00	1.57	0.50	0.92	0.46	1.19	0.55	0.49	5.00
41.00	1.61	0.50	0.92	0.46	1.19	0.55	0.48	5.00
42.00	1.65	0.50	0.91	0.46	1.19	0.54	0.48	5.00
43.00	1.69	0.50	0.91	0.45	1.19	0.54	0.47	5.00
44.00	1.73	0.50	0.90	0.45	1.19	0.54	0.47	5.00
45.00	1.77	0.50	0.90	0.45	1.19	0.54	0.46	5.00
46.00	1.81	0.50	0.89	0.45	1.19	0.53	0.46	5.00
47.00	1.85	0.50	0.89	0.44	1.19	0.53	0.46	5.00
48.00	1.90	0.50	0.88	0.44	1.19	0.53	0.45	5.00
49.00	1.94	0.50	0.88	0.44	1.19	0.52	0.45	5.00
50.00	1.98	0.50	0.87	0.44	1.19	0.52	0.44	5.00

* F.S.<1: Liquefaction Potential Zone. (If above water table: F.S.=5)
^ No-liquefiable Soils or above Water Table.
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

CPT convert to SPT for Settlement Analysis:
Fines Correction for Settlement Analysis:

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
1.00	-	-	-	100.00	NoLiq	0.00	100.00
2.00	-	-	-	100.00	NoLiq	0.00	100.00
3.00	-	-	-	100.00	NoLiq	0.00	100.00
4.00	-	-	-	100.00	NoLiq	0.00	100.00
5.00	-	-	-	100.00	NoLiq	0.00	100.00
6.00	-	-	-	89.74	70.60	0.00	89.74
7.00	-	-	-	75.12	40.20	0.00	75.12
8.00	-	-	-	58.00	25.00	0.00	58.00
9.00	-	-	-	53.94	25.00	0.00	53.94
10.00	-	-	-	44.05	25.00	0.00	44.05
11.00	-	-	-	46.56	25.00	0.00	46.56
12.00	-	-	-	48.92	25.00	0.00	48.92
13.00	-	-	-	51.13	25.00	0.00	51.13
14.00	-	-	-	53.23	25.00	0.00	53.23
15.00	-	-	-	61.21	25.00	0.00	61.21
16.00	-	-	-	60.12	25.00	0.00	60.12
17.00	-	-	-	59.15	25.00	0.00	59.15
18.00	-	-	-	58.30	25.00	0.00	58.30
19.00	-	-	-	57.53	25.00	0.00	57.53
20.00	-	-	-	56.85	25.00	0.00	56.85
21.00	-	-	-	54.15	30.00	0.00	54.15
22.00	-	-	-	51.39	35.00	0.00	51.39
23.00	-	-	-	46.91	40.00	0.00	46.91
24.00	-	-	-	42.51	45.00	0.00	42.51
25.00	-	-	-	38.32	50.00	0.00	38.32
26.00	-	-	-	43.33	51.00	0.00	43.33
27.00	-	-	-	48.16	52.00	0.00	48.16
28.00	-	-	-	55.37	53.00	0.00	55.37
29.00	-	-	-	60.15	54.00	0.00	60.15
30.00	-	-	-	64.80	55.00	0.00	64.80
31.00	-	-	-	58.85	55.00	0.00	58.85
32.00	-	-	-	53.10	55.00	0.00	53.10
33.00	-	-	-	47.53	55.00	0.00	47.53
34.00	-	-	-	42.12	55.00	0.00	42.12
35.00	-	-	-	36.88	55.00	0.00	36.88
36.00	-	-	-	38.48	55.00	0.00	38.48
37.00	-	-	-	40.04	55.00	0.00	40.04
38.00	-	-	-	41.57	55.00	0.00	41.57
39.00	-	-	-	43.06	55.00	0.00	43.06
40.00	-	-	-	44.53	55.00	0.00	44.53
41.00	-	-	-	45.39	49.00	0.00	45.39
42.00	-	-	-	46.22	43.00	0.00	46.22
43.00	-	-	-	47.04	37.00	0.00	47.04
44.00	-	-	-	46.27	31.00	0.00	46.27
45.00	-	-	-	44.81	25.00	0.00	44.81

46.00	-	-	-	45.41	40.19	0.00	45.41
47.00	-	-	-	42.30	55.39	0.00	42.30
48.00	-	-	-	39.26	70.59	0.00	39.26
49.00	-	-	-	36.29	85.79	0.00	36.29
50.00	-	-	-	33.39	NoLiq	0.00	33.39

(N1)60s has been fines corrected in liquefaction analysis, therefore d(N1)60=0.
Fines=NoLiq means the soils are not liquefiable.

Settlement of Saturated Sands:
Settlement Analysis Method: Ishihara / Yoshimine

Depth	CSRsf	/ MSF*	=CSRm	F.S.	Fines	(N1)60s	Dr	ec	dsz	dsp	S
ft					%		%	%	in.	in.	in.

No Settlement of Saturated Sands

Settlement of Saturated Sands=0.000 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=1.00 ft
S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

Depth	sigma'	sigC'	(N1)60s	CSRsf	Gmax	g*Ge/Gm	g_eff	ec7.5	Cec	ec	dsz	dsp	S
ft	atm	atm			atm			%		%	in.	in.	in.
49.95	3.04	1.97	33.54	0.44	2023.76	6.6E-4	0.2754	0.1303	0.93	0.1210	0.00E0	0.000	0.000
49.00	2.98	1.94	36.29	0.45	2057.55	6.5E-4	0.2547	0.1028	0.93	0.0955	1.15E-3	0.024	0.024
48.00	2.92	1.90	39.26	0.45	2089.95	6.3E-4	0.2358	0.0784	0.93	0.0728	8.74E-4	0.020	0.044
47.00	2.85	1.85	42.30	0.46	2119.34	6.1E-4	0.2192	0.0693	0.93	0.0644	7.72E-4	0.016	0.060
46.00	2.79	1.81	45.41	0.46	2145.93	6.0E-4	0.2045	0.0647	0.93	0.0600	7.21E-4	0.015	0.075
45.00	2.73	1.77	44.81	0.46	2111.86	6.0E-4	0.2062	0.0652	0.93	0.0606	7.27E-4	0.014	0.090

44.00	2.66	1.73	46.27	0.47	2109.66	5.9E-4	0.1992	0.0630	0.93	0.0585	7.02E-4	0.014	0.104
43.00	2.60	1.69	47.04	0.47	2096.39	5.9E-4	0.1952	0.0617	0.93	0.0573	6.88E-4	0.014	0.118
42.00	2.54	1.65	46.22	0.48	2059.58	5.9E-4	0.1972	0.0624	0.93	0.0579	6.95E-4	0.014	0.132
41.00	2.48	1.61	45.39	0.48	2022.71	5.9E-4	0.1993	0.0630	0.93	0.0585	7.02E-4	0.014	0.146
40.00	2.42	1.57	44.53	0.49	1985.78	5.9E-4	0.2015	0.0637	0.93	0.0592	7.10E-4	0.014	0.160
39.00	2.36	1.54	43.06	0.49	1939.87	6.0E-4	0.2063	0.0652	0.93	0.0606	7.27E-4	0.014	0.174
38.00	2.30	1.50	41.57	0.50	1893.24	6.1E-4	0.4714	0.1491	0.93	0.1384	1.66E-3	0.017	0.191
37.00	2.24	1.46	40.04	0.50	1845.84	6.1E-4	0.4917	0.1555	0.93	0.1444	1.73E-3	0.034	0.225
36.00	2.19	1.42	38.48	0.51	1797.61	6.2E-4	0.5142	0.1805	0.93	0.1676	2.01E-3	0.037	0.262
35.00	2.13	1.38	36.88	0.51	1748.50	6.2E-4	0.5392	0.2099	0.93	0.1950	2.34E-3	0.044	0.306
34.00	2.07	1.34	42.12	0.52	1802.32	5.9E-4	0.4278	0.1353	0.93	0.1256	1.51E-3	0.036	0.342
33.00	2.01	1.31	47.53	0.52	1849.47	5.7E-4	0.3512	0.1110	0.93	0.1031	1.24E-3	0.027	0.369
32.00	1.95	1.27	53.10	0.53	1890.85	5.4E-4	0.2956	0.0935	0.93	0.0868	1.04E-3	0.023	0.391
31.00	1.89	1.23	58.85	0.53	1927.15	5.2E-4	0.2537	0.0802	0.93	0.0745	8.94E-4	0.019	0.411
30.00	1.83	1.19	64.80	0.54	1958.87	5.0E-4	0.2211	0.0699	0.93	0.0649	7.79E-4	0.017	0.427
29.00	1.78	1.15	60.15	0.54	1880.09	5.1E-4	0.2297	0.0726	0.93	0.0675	8.09E-4	0.016	0.443
28.00	1.72	1.12	55.37	0.54	1798.27	5.1E-4	0.2407	0.0761	0.93	0.0707	8.48E-4	0.017	0.460
27.00	1.66	1.08	48.16	0.54	1686.80	5.3E-4	0.2699	0.0853	0.93	0.0793	9.51E-4	0.018	0.478
26.00	1.60	1.04	43.33	0.54	1598.82	5.4E-4	0.2906	0.0919	0.93	0.0853	1.02E-3	0.020	0.498
25.00	1.54	1.00	38.32	0.54	1505.75	5.5E-4	0.3197	0.1134	0.93	0.1053	1.26E-3	0.022	0.520
24.00	1.48	0.96	42.51	0.54	1527.79	5.3E-4	0.2616	0.0827	0.93	0.0768	9.22E-4	0.021	0.540
23.00	1.42	0.92	46.91	0.54	1545.63	5.0E-4	0.2178	0.0689	0.93	0.0640	7.68E-4	0.017	0.557
22.00	1.35	0.88	51.39	0.55	1558.12	4.7E-4	0.1844	0.0583	0.93	0.0542	6.50E-4	0.014	0.571
21.00	1.29	0.84	54.15	0.55	1548.41	4.6E-4	0.1634	0.0517	0.93	0.0480	5.76E-4	0.012	0.583
20.00	1.23	0.80	56.85	0.55	1534.58	4.4E-4	0.1455	0.0460	0.93	0.0427	5.13E-4	0.011	0.594

19.00	1.16	0.76	57.53	0.55	1500.17	4.3E-4	0.1343	0.0425	0.93	
0.0394	4.73E-4	0.010	0.604							
18.00	1.10	0.72	58.30	0.55	1464.93	4.1E-4	0.3363	0.1063	0.93	
0.0988	1.19E-3	0.023	0.627							
17.00	1.04	0.67	59.15	0.55	1428.77	4.0E-4	0.2852	0.0902	0.93	
0.0838	1.01E-3	0.022	0.649							
16.00	0.97	0.63	60.12	0.55	1391.61	3.9E-4	0.2408	0.0762	0.93	
0.0707	8.49E-4	0.018	0.667							
15.00	0.91	0.59	61.21	0.56	1353.32	3.7E-4	0.2025	0.0641	0.93	
0.0595	7.14E-4	0.016	0.683							
14.00	0.85	0.55	53.23	0.56	1245.96	3.8E-4	0.2147	0.0679	0.93	
0.0630	7.57E-4	0.015	0.698							
13.00	0.78	0.51	51.13	0.56	1183.02	3.7E-4	0.1938	0.0613	0.93	
0.0569	6.83E-4	0.014	0.713							
12.00	0.72	0.47	48.92	0.56	1118.71	3.6E-4	0.1746	0.0552	0.93	
0.0513	6.15E-4	0.013	0.725							
11.00	0.66	0.43	46.56	0.56	1052.84	3.5E-4	0.1569	0.0496	0.93	
0.0461	5.53E-4	0.012	0.737							
10.00	0.60	0.39	44.05	0.56	985.21	3.4E-4	0.1407	0.0445	0.93	
0.0413	4.96E-4	0.010	0.748							
9.00	0.54	0.35	53.94	0.56	999.85	3.0E-4	0.0919	0.0291	0.93	
0.0270	3.24E-4	0.008	0.756							
8.00	0.48	0.31	58.00	0.56	965.72	2.8E-4	0.1713	0.0542	0.93	
0.0503	6.04E-4	0.017	0.772							
7.00	0.42	0.27	75.12	0.57	984.64	2.4E-4	0.0636	0.0201	0.93	
0.0187	2.24E-4	0.007	0.780							
6.00	0.36	0.23	89.74	0.57	967.20	2.1E-4	0.0425	0.0134	0.93	
0.0125	1.50E-4	0.004	0.783							
5.00	0.30	0.20	100.00	0.57	915.34	1.9E-4	0.0358	0.0113	0.93	
0.0105	0.00E0	0.003	0.786							
4.00	0.24	0.16	100.00	0.57	818.70	1.7E-4	0.0306	0.0097	0.93	
0.0090	0.00E0	0.000	0.786							
3.00	0.18	0.12	100.00	0.57	709.02	1.5E-4	0.0278	0.0088	0.93	
0.0082	0.00E0	0.000	0.786							
2.00	0.12	0.08	100.00	0.57	578.91	1.2E-4	0.0254	0.0080	0.93	
0.0075	0.00E0	0.000	0.786							
1.00	0.06	0.04	100.00	0.57	409.35	8.4E-5	0.0151	0.0048	0.93	
0.0044	0.00E0	0.000	0.786							

Settlement of Unsaturated Sands=0.786 in.
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=1.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Unsaturated Sands=0.786 in.
Differential Settlement=0.393 to 0.519 in.

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight =

pcf; Depth = ft; Settlement = in.

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1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)

SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT) [atm (tsf)]
fs	Friction from CPT testing [atm (tsf)]
Rf	Ratio of fs/qc (%)
gamma	Total unit weight of soil
gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size
Dr	Relative Density
sigma	Total vertical stress [atm]
sigma'	Effective vertical stress [atm]
sigC'	Effective confining pressure [atm]
rd	Acceleration reduction coefficient by Seed
a_max.	Peak Ground Acceleration (PGA) in ground surface
mZ	Linear acceleration reduction coefficient X depth
a_min.	Minimum acceleration under linear reduction, mZ
CRRv	CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksig	Overburden stress correction factor for CRR7.5
CRRm	After magnitude scaling correction CRRm=CRRv * MSF
MSF	Magnitude scaling factor from M=7.5 to user input M
CSR	Cyclic stress ratio induced by earthquake
CSRfs	CSRfs=CSR*fs1 (Default fs1=1)
fs1	First CSR curve in graphic defined in #9 of Advanced page
fs2	2nd CSR curve in graphic defined in #9 of Advanced page
F.S.	Calculated factor of safety against liquefaction
F.S.=CRRm/CSRsf	
Cebs	Energy Ratio, Borehole Dia., and Sampling Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60	Fines correction of SPT
(N1)60f	(N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq	Overburden stress correction factor
qc1	CPT after Overburden stress correction
dqc1	Fines correction of CPT
qc1f	CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n	CPT after normalization in Robertson's method
Kc	Fine correction factor in Robertson's Method
qc1f	CPT after Fines correction in Robertson's Method
Ic	Soil type index in Suzuki's and Robertson's Methods
(N1)60s	(N1)60 after settlement fines corrections
CSRm	After magnitude scaling correction for Settlement

calculation CSRm=CSRsf / MSF*

CSRfs	Cyclic stress ratio induced by earthquake with user inputed fs
MSF*	Scaling factor from CSR, MSF*=1, based on Item 2 of Page C.
ec	Volumetric strain for saturated sands
dz	Calculation segment, dz=0.050 ft
dsz	Settlement in each segment, dz
dp	User defined print interval
dsp	Settlement in each print interval, dp
Gmax	Shear Modulus at low strain
g_eff	gamma_eff, Effective shear Strain
g*Ge/Gm	gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5	Volumetric Strain for magnitude=7.5
Cec	Magnitude correction factor for any magnitude
ec	Volumetric strain for unsaturated sands, ec=Cec * ec7.5
NoLiq	No-Liquefy Soils

References:

1. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.

SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.

2. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING AND SEISMIC SITE RESPONSE EVALUATION, Paper No. SPL-2, PROCEEDINGS: Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA, March 2001.

3. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING: A UNIFIED AND CONSISTENT FRAMEWORK, Earthquake Engineering Research Center, Report No. EERC 2003-06 by R.B Seed and etc. April 2003.

Note: Print Interval you selected does not show complete results. To get complete results, you should select 'Segment' in Print Interval (Item 12, Page C).

APPENDIX E

SOIL CORROSIVITY STUDY

Single-Family Residential Tract Development

Brodiaea Avenue and Oliver Street

Moreno Valley, Riverside County, California

Project No. 2867-CR

September 28, 2021 via email: bhick@geotekusa.com

Geotek, Inc
1548 North Maple
Corona, CA, 92614

Attention: Mr. Bruce A. Hicks

Re: Soil Corrosivity Study
Discovery Church Project
Moreno Valley, CA
HDR #21-0862SCS, Geotek #2867-CR

Introduction

Laboratory tests have been completed on five soil sample provided to HDR for the Discovery Church project. The purpose of these tests was to determine whether the soils are likely to have deleterious effects on underground utility piping. HDR assumes that the provided samples are representative of the most corrosive soils at the site.

The proposed structure has two stories and no subterranean levels. The site is located at the North East corner of Brodiaea Avenue and Olive Street in Moreno Valley, California, and the water table is reportedly greater than 50 feet deep.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. HDR's recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, HDR will be happy to work with them as a separate phase of this project.

Soil Corrosivity Testing

Laboratory Testing

The electrical resistivity of each sample was measured in a soil box per *ASTM International (ASTM)* G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured per ASTM G51. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327, ASTM D6919, and *American Water Works Association (AWWA)* Standard Method 2320-B.

The laboratory analyses were performed under HDR laboratory number 21-0862SCS. The full set of test results are shown in the attached Table 1.

Discussion

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil. A correlation between electrical resistivity and corrosivity toward ferrous metals is shown in Table 1.[1]

Table 1: Soil Corrosivity Categories.

Soil Resistivity (ohm-cm)	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,001 to 10,000	Moderately Corrosive
1,001 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

Electrical resistivities were in the mildly corrosive category with as-received moisture. When saturated, the resistivities were in the moderately corrosive category. The resistivities dropped considerably with added moisture because the samples were dry as-received.

Soil pH values varied from 7.8 to 8.0. This range is mildly to moderately alkaline.[2] These values do not particularly increase soil corrosivity.

The soluble salt content of the samples were low.

Per ACI-318, the soil is classified as S0 with respect to sulfate concentration.[3]

The nitrate concentration was high enough to be aggressive to copper. Ammonium was not detected.

Tests were not made for sulfide and oxidation-reduction (redox) potential because these samples did not exhibit characteristics typically associated with anaerobic conditions.

In conclusion, this soil is classified as moderately corrosive to ferrous metals, aggressive to copper, and negligible (S0) for sulfate attack on concrete.

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[1] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.*
[2] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*
[3] *American Concrete Institute (ACI) 318-19 Table 19.3.1.1.*

Corrosion Control Recommendations

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion. The following recommendations are based on the evaluation of soil corrosivity described above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.
2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.
3. To prevent differential aeration corrosion cells, provide at least 2 inches of pipe bedding or backfill material all around metallic piping, including the bottom. Do not lay pipe directly on undisturbed soil.

Steel Pipe

1. Underground steel pipe with rubber gasketed, mechanical, grooved end, or other nonconductive type joints should be bonded for electrical continuity. Electrical continuity is necessary for corrosion monitoring and the possible future application of cathodic protection.
2. Install corrosion monitoring test stations to facilitate corrosion monitoring and the possible future application of cathodic protection:
    a. At each end of the pipeline.
    b. At each end of all casings.
    c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.
3. To prevent dissimilar metal corrosion cells and to facilitate the possible future application of cathodic protection, electrically isolate each buried steel pipeline per NACE SP0286 from:
    a. Dissimilar metals.
    b. Dissimilarly coated piping (cement-mortar vs. dielectric).
    c. Above ground steel pipe.
    d. All existing piping.

   Insulated joints should be placed above grade or in vaults where possible. Wrap all buried insulators with wax tape per AWWA C217.

4. Choose one of the following corrosion control options:

OPTION 1

a. Apply a suitable dielectric coating intended for underground use such as:

i. Polyurethane per AWWA C222 *or*

ii. Extruded polyethylene per AWWA C215 *or*

iii. A tape coating system per AWWA C214 *or*

iv. Hot applied coal tar enamel per AWWA C203 *or*

v. Fusion bonded epoxy per AWWA C213.

b. Although it is customary to cathodically protect bonded dielectrically coated structures, cathodic protection is not recommended at this time because the soil is considered only moderately corrosive to ferrous materials. Install joint bonds, test stations, and insulated joints to provide for corrosion monitoring and/or the future application of cathodic protection to control leaks if needed.

OPTION 2

As an alternative to the coating systems described in Option 1 and possible future cathodic protection, apply a ¾-inch cement mortar coating per AWWA C205 or encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement. Install joint bonds, test stations, and insulated joints to provide for corrosion monitoring and/or the future application of cathodic protection if needed.

NOTE: Some steel piping systems, such as for oil, gas, and high-pressure piping systems, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Ductile Iron Pipe

1. To prevent dissimilar metal corrosion cells and to facilitate the possible future application of cathodic protection, electrically insulate underground iron pipe from dissimilar metals and from above ground iron pipe with insulating joints per NACE SP0286.

2. Bond all nonconductive type joints for electrical continuity. Electrical continuity is necessary for corrosion monitoring and possible future application of cathodic protection.

3. Install corrosion monitoring test stations to facilitate corrosion monitoring and the possible future application of cathodic protection:

   a. At each end of the pipeline.

   b. At each end of any casings.

   c. Other locations as necessary so the interval between test stations does not exceed 1,200 feet.

4. Choose one of the following corrosion control options:

   OPTION 1

   a. Apply a suitable coating intended for underground use such as:

      i. Polyethylene encasement per AWWA C105; *or*

      ii. Epoxy coating; *or*

      iii. Polyurethane; *or*

      iv. Wax tape.

      NOTE: The thin factory-applied asphaltic coating applied to ductile iron pipe for transportation and aesthetic purposes does not constitute a corrosion control coating.

   b. Although it is customary to cathodically protect coated structures, cathodic protection is not recommended at this time because the soil is considered only moderately corrosive to ferrous materials. Install joint bonds, test stations, and insulated joints to provide for corrosion monitoring and/or the future application of cathodic protection to control leaks if needed.

   OPTION 2

   As an alternative to the coating systems described in Option 1 and possible future cathodic protection, encase all buried portions of metallic piping so that there is a minimum of 3 inches of concrete cover provided over and around surfaces of pipe, fittings, and valves using any type of ASTM C150 cement. Install joint bonds, test stations, and insulated joints to provide for corrosion monitoring and/or the future application of cathodic protection if needed.

NOTE: Some iron piping systems, such as for fire water piping, have special corrosion and cathodic protection requirements that must be evaluated for each specific application.

Cast Iron Soil Pipe

1. Protect cast iron soil pipe with either a double wrap 4-mil or single wrap 8-mil polyethylene encasement per AWWA C105.

2. It is not necessary to bond the pipe joints or apply cathodic protection.

3. Provide 6 inches of clean sand backfill all around the pipe. Use the following parameters for clean sand backfill:

   a. Minimum saturated resistivity of no less than 3,000 ohm-cm; *and*

   b. pH between 6.0 and 8.0.

   c. All backfill testing should be performed by a corrosion engineering laboratory.

Copper Tubing

1. Use Type K or Type L copper tubing as required by the applicable local plumbing code. Type M tubing should not be used for buried applications.[4]

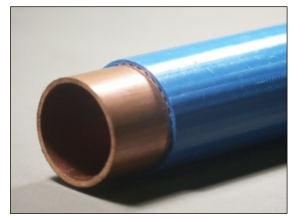
2. Electrically insulate underground copper pipe from dissimilar metals and from above ground copper pipe with insulating devices per NACE SP0286.

3. Electrically insulate cold water piping from hot water piping systems.

4. Protect buried copper tubing by one of the following measures:

    a. Prevent soil contact. Soil contact may be prevented by placing the tubing above ground or encasing the tubing using PVC pipe with solvent-welded joints. Either seal the PVC pipe at both ends or terminate both ends above-grade in a manner that doesn't allow water to infiltrate; or

    b. Install copper pipe with a factory-applied coating that is at least 25 mils in thickness. Use Kamco's Aqua Shield™, Mueller Streamline's Plumbshield™, or equal. The coating must be continuous with no cuts or defects.



    c. Insulate the pipe by installing 12-mil polyethylene pipe wrapping tape with butyl rubber mastic over a suitable primer. Protect wrapped copper tubing by applying cathodic protection per NACE SP0169.

Plastic and Vitrified Clay Pipe

1. No special corrosion control measures are required for plastic and vitrified clay piping placed underground.

2. Protect all metallic fittings and valves with wax tape per AWWA C217, or with epoxy and appropriately designed cathodic protection system per NACE SP0169.

Concrete Structures and Pipe

1. From a corrosion standpoint, any type of ASTM C150 cement may be used for concrete structures and pipe because the sulfate concentration is negligible (S0), from 0 to 0.10 percent. Use a minimum strength of 2,500 psi per applicable codes.[5,6,7]

2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentrations found on site.[8] Limit the water-soluble chloride ion content in the concrete mix design to less than 0.3 percent by weight of cement.

[4] 2016 California Plumbing Code (CPC), July 1, 2018 Supplement, Section 604.3.
[5] 2018 International Building Code (IBC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1
[6] 2015 International Residential Code (IRC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1
[7] 2016 California Building Code (CBC) which refers to American Concrete Institute (ACI) 318-19 Table 19.3.2.1
[8] Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65

Closure

The analysis and recommendations presented in this report are based upon data obtained from the laboratory samples. This report does not reflect variations that may occur across the site or due to the modifying effects of construction. If variations appear, HDR should be notified immediately so that further evaluation and supplemental recommendations can be provided.

HDR's services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

Respectfully Submitted,
HDR Engineering, Inc.

Bradley Stuart, PE
Corrosion Engineer

Marc E N Wegner, PE
Sr. Corrosion Project Manager

Enc: Table 1

21-0862SCS SCS JTK_BS_MW.docx

Table 1 - Laboratory Tests on Soil Samples

Geotek, Inc.
Discovery Church Project
Your #2867-CR, HDR Lab #21-0862SCS
22-Sep-21

Sample ID			B-1 @ 3'	B-2 @ 0-5'	B-3 @ 0-10'	B-4 @ 3'	B-5 @ 2'
Resistivity		**Units**					
as-received		ohm-cm	40,000	40,000	18,000	52,000	28,400
saturated		ohm-cm	3,440	3,600	3,480	4,800	3,200
pH			7.9	8.0	7.9	7.8	7.8
Electrical Conductivity		mS/cm	0.06	0.09	0.08	0.10	0.07
Chemical Analyses							
Cations							
calcium	Ca^{2+}	mg/kg	44	77	96	72	60
magnesium	Mg^{2+}	mg/kg	12	16	8.7	17	19
sodium	Na^{1+}	mg/kg	54	48	18	19	45
potassium	K^{1+}	mg/kg	6.6	9.8	12	30	8.2
ammonium	NH_4^{1+}	mg/kg	ND	ND	ND	ND	ND
Anions							
carbonate	CO_3^{2-}	mg/kg	ND	50	41	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	192	146	207	284	189
fluoride	F^{1-}	mg/kg	7.5	6.3	8.3	11	11
chloride	Cl^{1-}	mg/kg	7.2	15	7.5	10	14
sulfate	SO_4^{2-}	mg/kg	30	15	11	13	17
nitrate	NO_3^{1-}	mg/kg	24	7.6	29	91	87
phosphate	PO_4^{3-}	mg/kg	ND	ND	ND	ND	ND
Other Tests							
sulfide	S^{2-}	qual	na	na	na	na	na
Redox		mV	na	na	na	na	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

APPENDIX F

GENERAL EARTHWORK GRADING GUIDELINES

Single-Family Residential Tract Development
Brodiaea Avenue and Oliver Street
Moreno Valley, Riverside County, California
Project No. 2867-CR

GEOTEK

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, Chapters 18 and 33 of the Uniform 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 oversize 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

It is anticipated that 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 recommend 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.

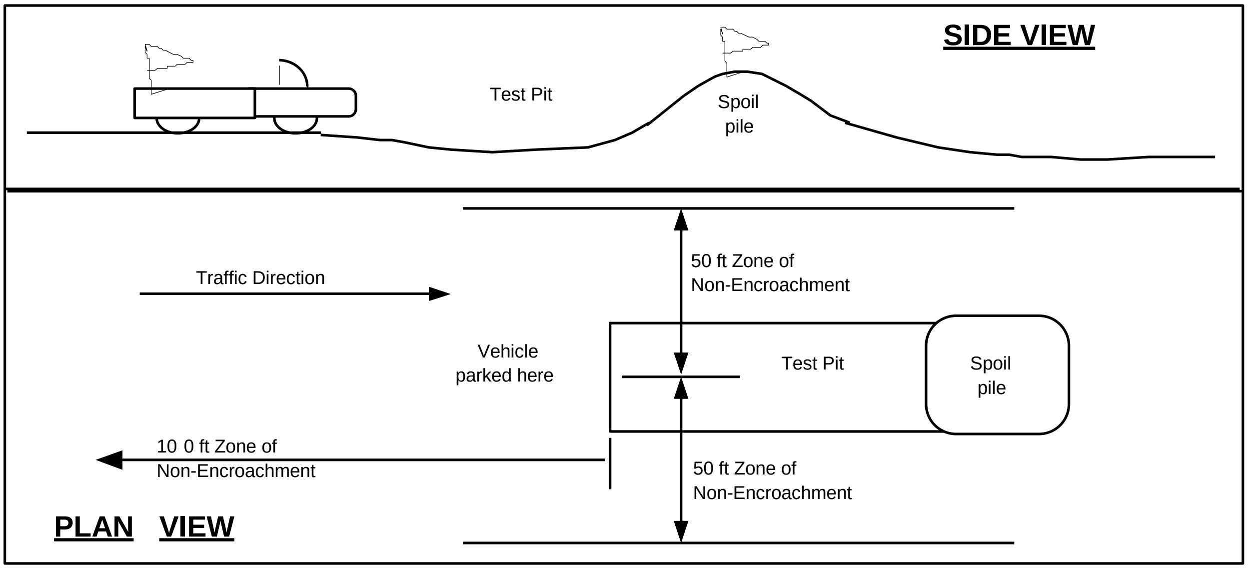
Test Pits Location, Orientation and Clearance

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.

TEST PIT SAFETY PLAN



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.

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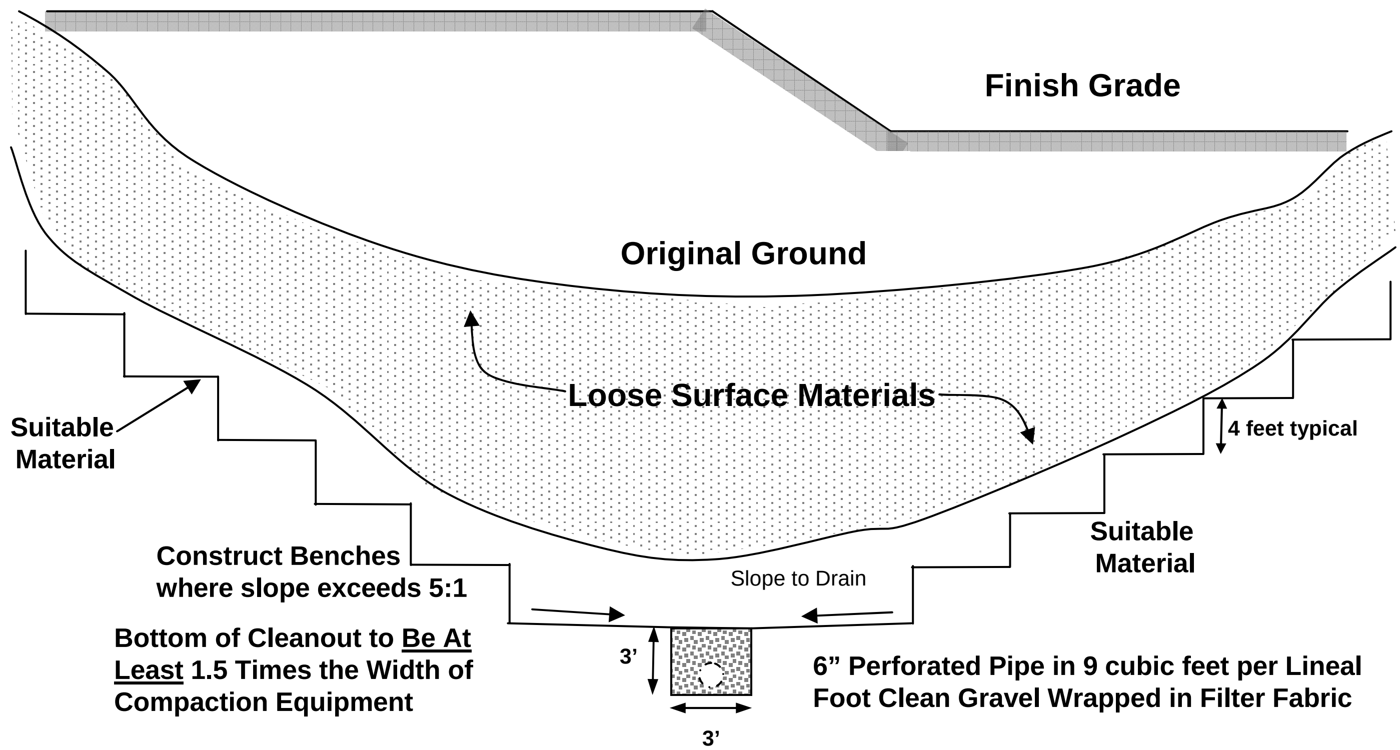
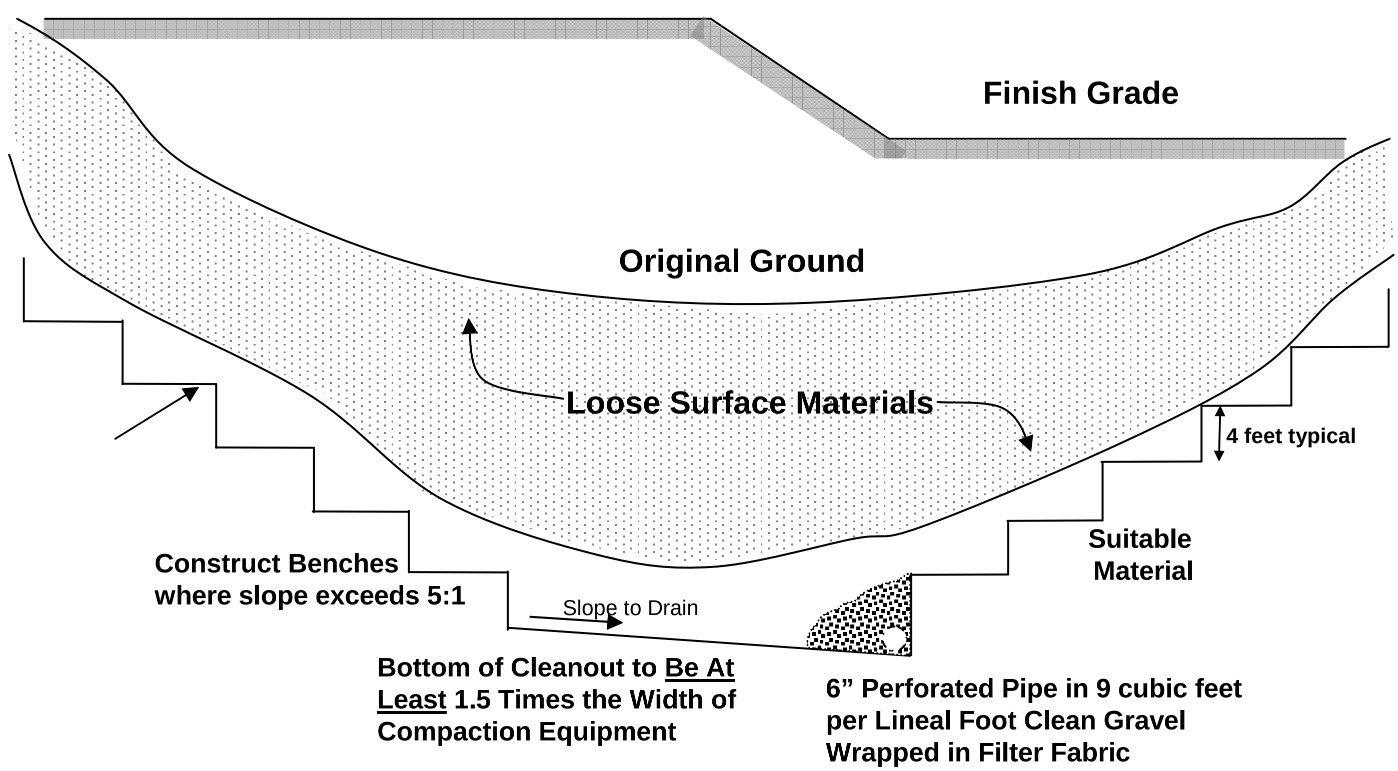
ALTERNATES





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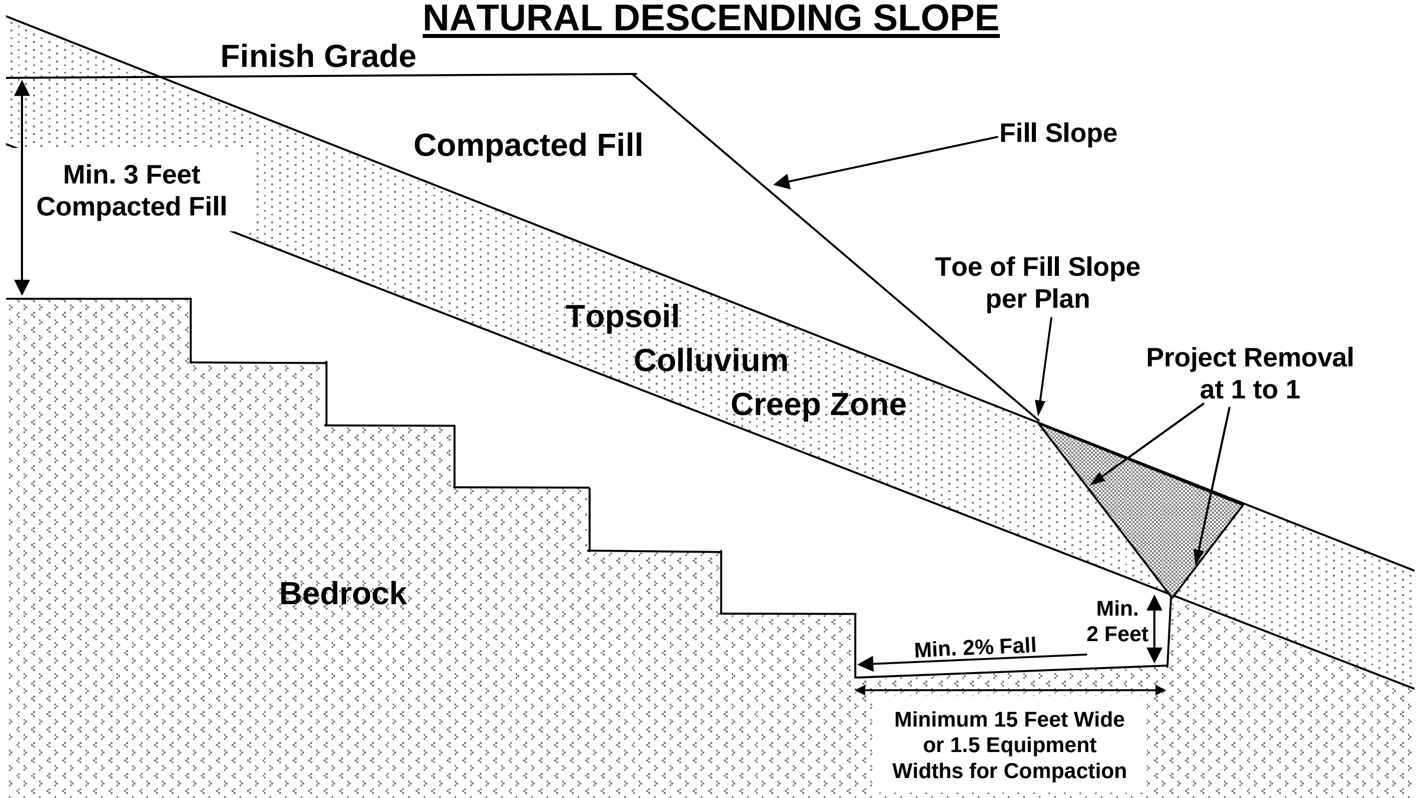
TYPICAL CANYON CLEANOUT

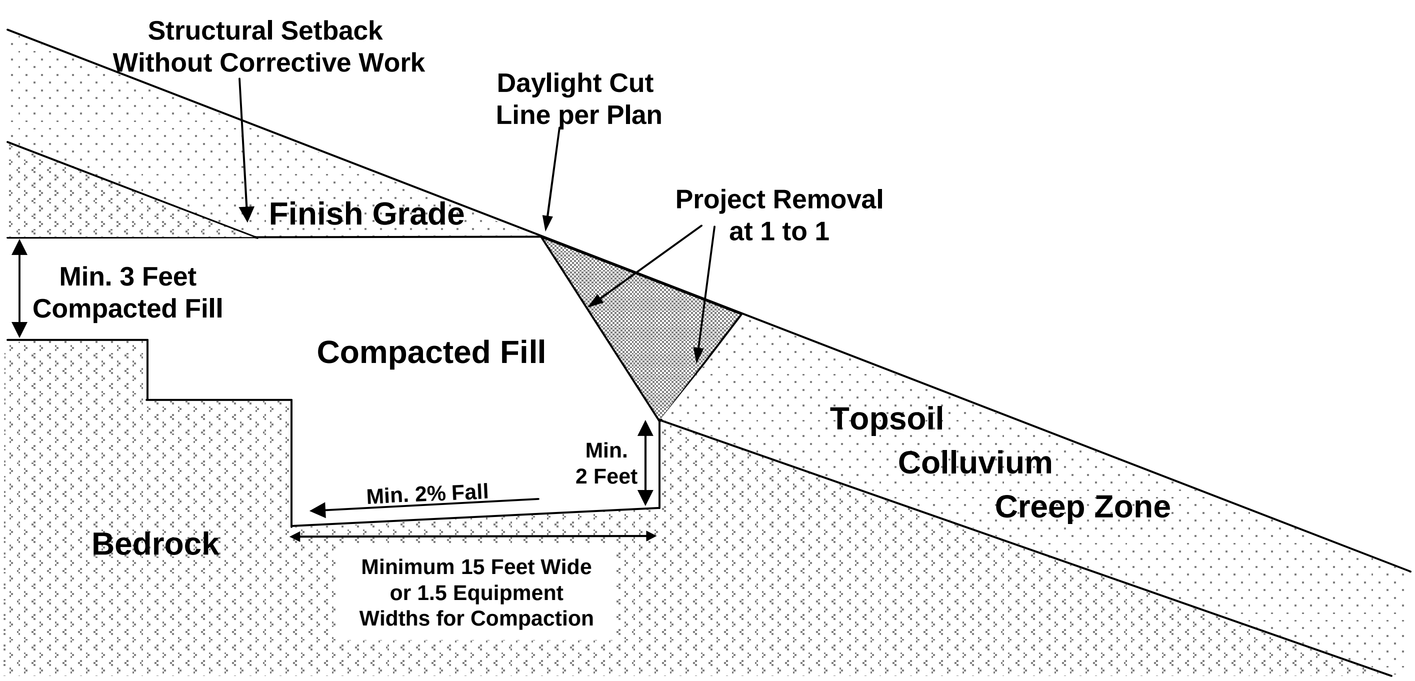
STANDARD GRADING GUIDELINES

PLATE F-1

TYPICAL FILL SLOPE OVER NATURAL DESCENDING SLOPE



DAYLIGHT CUT AREA OVER NATURAL DESCENDING SLOPE



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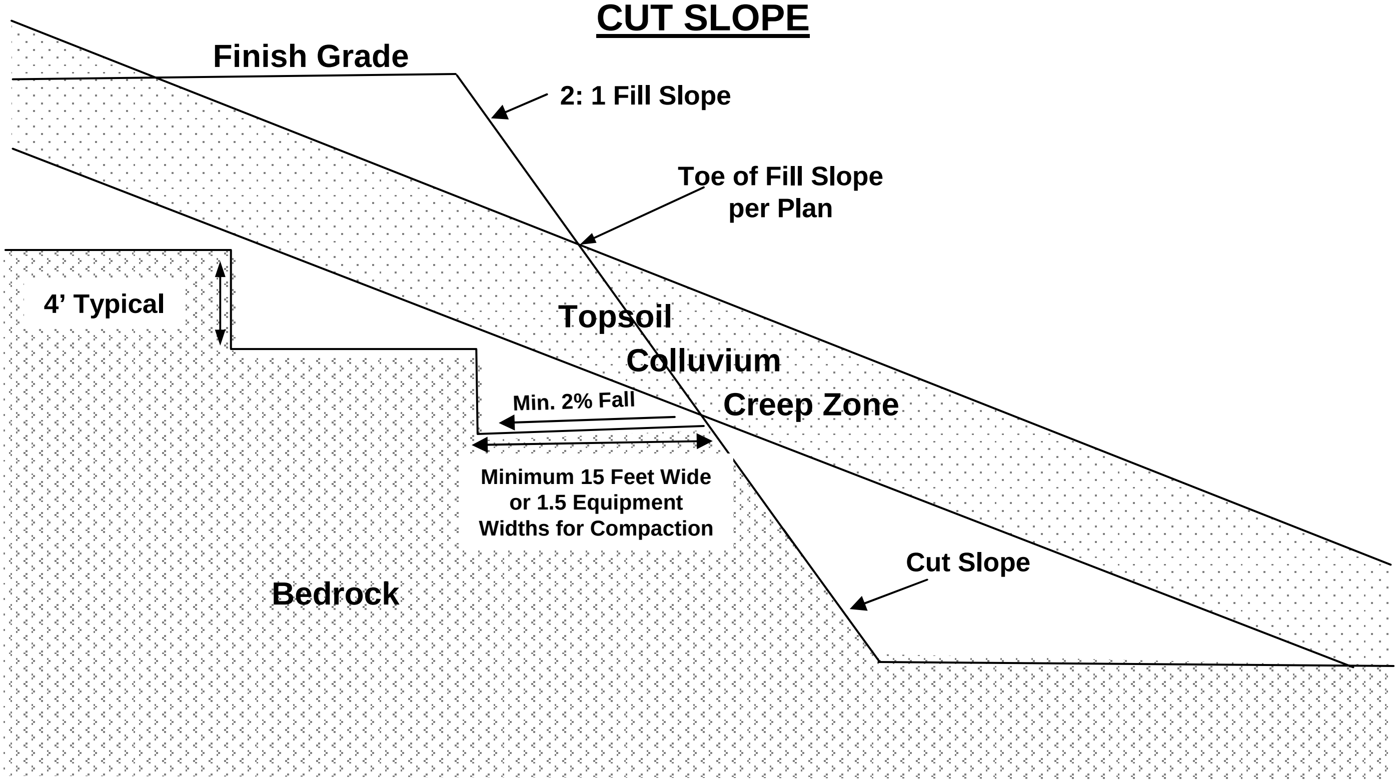
TREATMENT ABOVE NATURAL SLOPES

STANDARD GRADING GUIDELINES

PLATE F-2

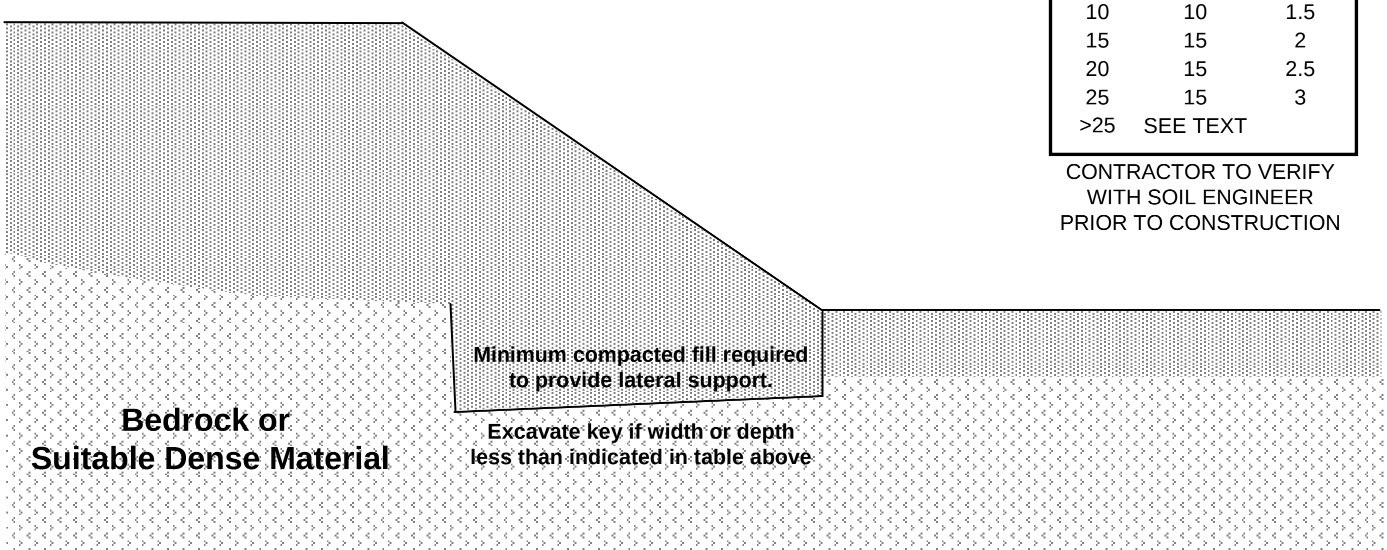
TYPICAL FILL SLOPE OVER CUT SLOPE



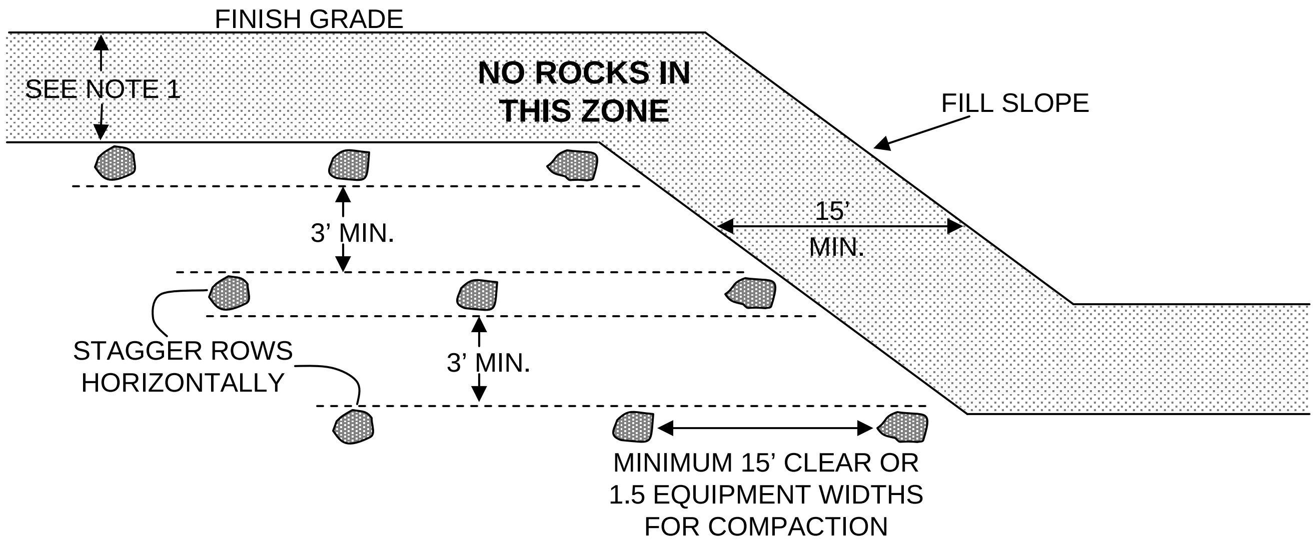
TYPICAL FILL SLOPE

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	

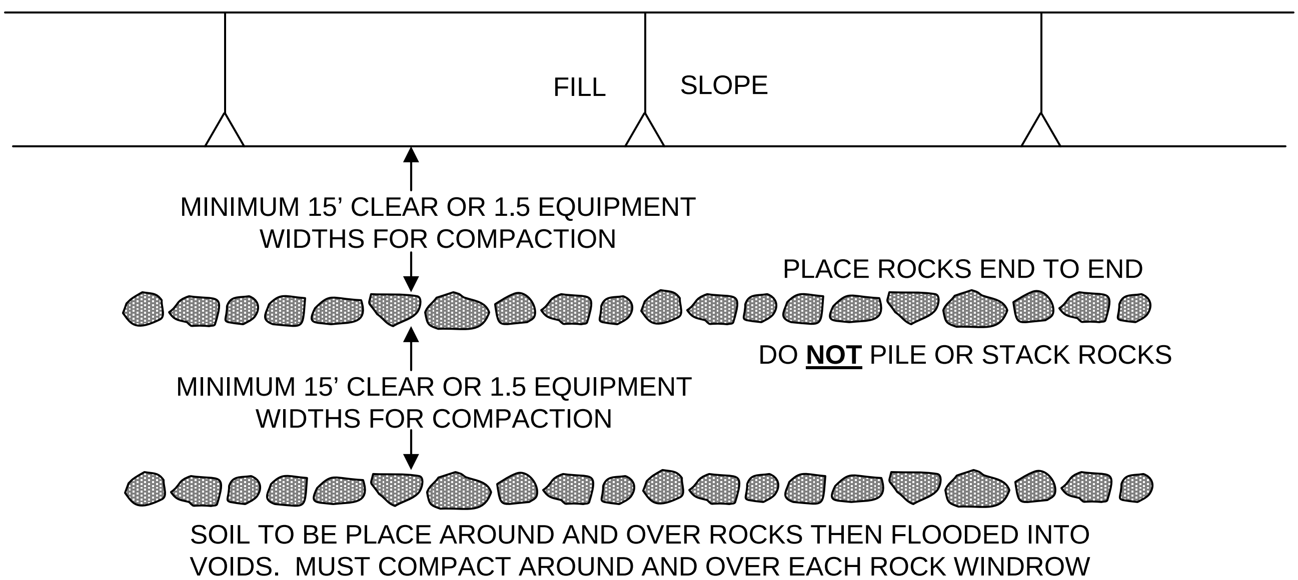
CONTRACTOR TO VERIFY WITH SOIL ENGINEER PRIOR TO CONSTRUCTION



CROSS SECTIONAL VIEW

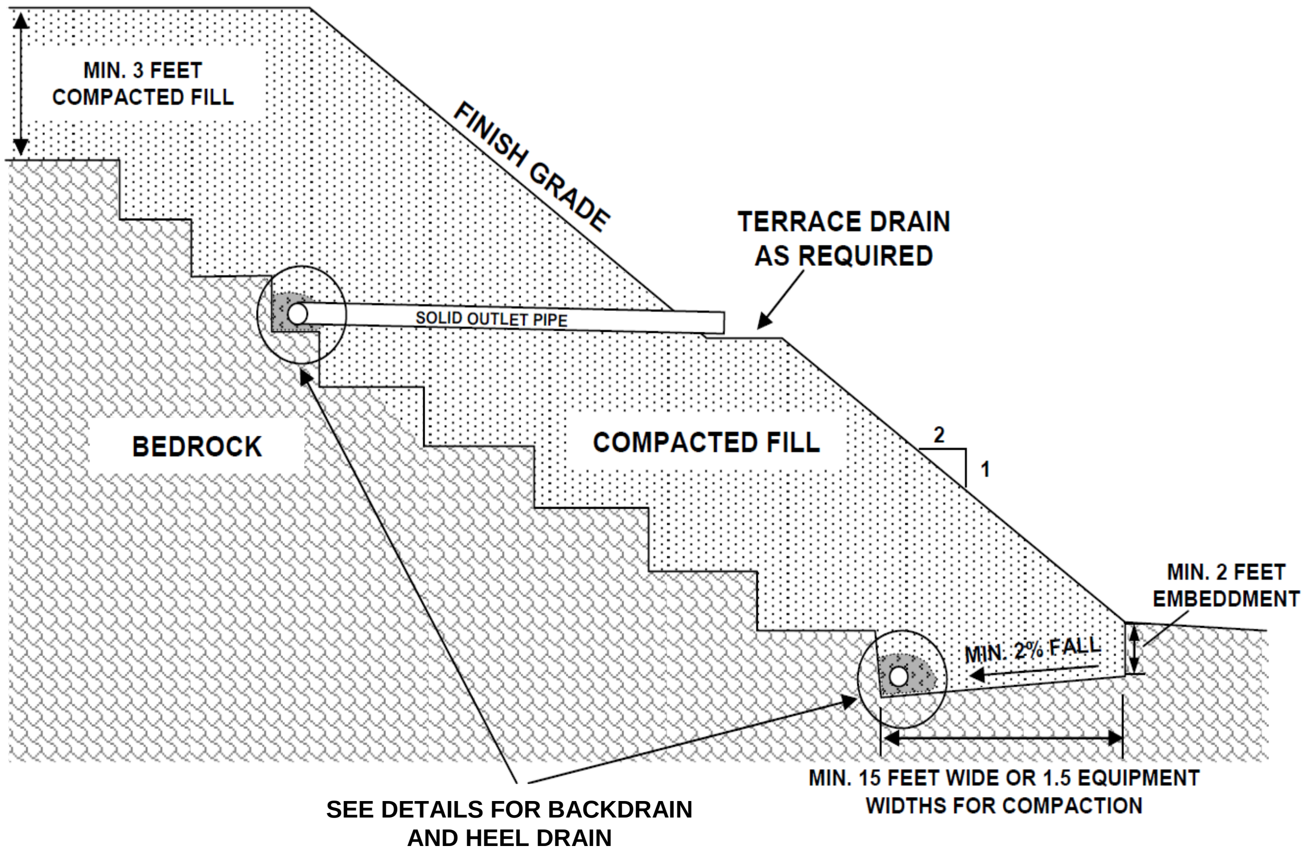


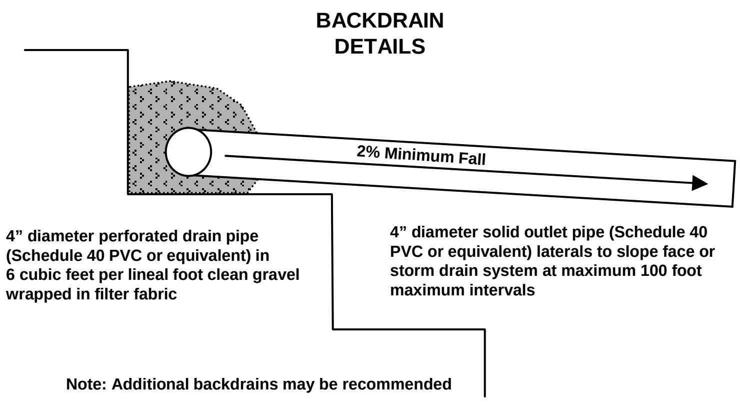
PLAN VIEW

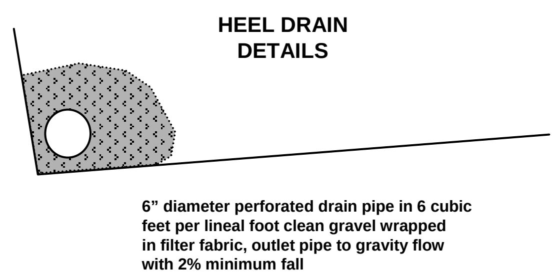


NOTES:

1) SOIL FILL OVER WINDROW SHOULD BE 7 FEET OR PER JURISDUICTIONAL STANDARDS AND SUFFICIENT FOR FUTURE EXCAVATIONS TO AVOID ROCKS
2) MAXIMUM ROCK SIZE IN WINDROWS IS 4 FEET IN 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) INDIVDUAL LARGE ROCKS MAY BE BURIED IN PITS.







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TYPICAL BUTTRESS AND STABILIZATION FILL

STANDARD GRADING GUIDELINES

PLATE F-5