

**GEOTECHNICAL AND INFILTRATION EVALUATION  
PROPOSED SINGLE-FAMILY RESIDENTIAL TRACT DEVELOPMENT  
WHITNEY 162 AND VALLEY CHURCH (80 LOTS) PROJECTS  
ALESSANDRO BOULEVARD AND OLIVER STREET  
MORENO VALLEY, RIVERSIDE COUNTY, CALIFORNIA**

**PREPARED FOR**

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**PROJECT No. 2868-CR**

**SEPTEMBER 29, 2021**

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September 29, 2021  
Project No. 2868-CR

**D. R. Horton Los Angeles Holding Company, Inc.**  
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Attention: Ms. Megan Whieldon

Subject: Geotechnical and Infiltration Evaluation  
Proposed Single-Family Residential Tract Development  
Whitney 162 and Valley Church (80 Lots) Project  
Alessandro Boulevard 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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Appendix A – Log of Exploratory Borings

Appendix B – Results of Laboratory Testing

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Appendix D – Seismic Settlement Analysis

Appendix E – Soil Corrosion Report

Appendix F – General Earthwork Grading Guidelines

## **I. 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-0802021-CR, dated August 6, 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 twelve (12) exploratory test borings extending to depths ranging from 16.5 to 51.5 feet below grade,
- Excavation of eight (8) 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 29-acre "L"-shaped project site is located adjacent to the southwest corner of Alessandro Boulevard and Oliver Street, in the City of Moreno Valley, Riverside County, California (See Figure 1). Access to the site is available from Alessandro Boulevard and Oliver Street, both paved, improved streets located adjacent to the northern and eastern boundaries of the site, respectively. Brodiaea Avenue, currently a dirt trail, forms the southern boundary of the site adjacent to existing single-family developments. Vacant land is located adjacent to the western boundary of the site.

Topographically, the site slopes gently downward to the southeast at an approximate two (2) percent gradient. Elevation of the northwestern portion of the the site is approximately 1,585 feet with approximately 30 feet of elevation differential across the site.

The site was vacant land at the time of the field exploration. The site appears to have been previously utilized for agriculture. The majority of the project site appears to have been disced for vegetation contraol but a light covering of grass and small brush was present in the northern portions of the site along with scattered domestic trees adjacent to edges of the property. A high-pressure natural gas lines trends in an east-west direction along the extension of Brodiaaea Avenue.

## **2.2 PROJECT DESCRIPTION**

Based upon discussions with representatives of D. R. Horton, the project consists of two previously separated sites referred to as the “Whitney 162 Project” and the “Valley Church Project” (80 Lots). These projects are to be combined into one Tentative Tract Map.

In addition to the planned 242 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: three are planned on the Whitney 162 project site with one on the Valley Church project site. 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 2 and 3, 2021 and consisted of excavating twelve (12) geotechnical exploratory borings with a hollow-stem drill rig to depths ranging from about 16.5 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-12 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-12.

#### **Percolation Testing**

In addition to the geotechnical exploratory borings, eight (8) percolation test borings (I-1 through I-8) 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:

<b>SUMMARY OF INFILTRATION RATES</b>		
<b>Boring</b>	<b>Depth of Test (Feet)</b>	<b>Infiltration Rate (Inches per hour)</b>
I-1	5.0	1.76
I-2	5.0	1.39
I-3	5.0	1.03
I-4	5.0	1.14
I-5	5.0	1.45
I-6	5.0	1.45
I-7	5.0	0.91
I-8	5.0	1.03

The results of the conversions indicate infiltration rate range from about 0.91 to 1.76 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 in the extension of Brodiaea Avenue along the southern boundary of the site. This fill is most likely from locally derived locations. 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 highly 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 “very low” 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 33.9155 degrees West Latitude and -117.1850 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,  $C_s$ , is conservatively calculated by Eq 12.8-2 of ASCE 7-16 for values of  $T \leq 1.5T_L$  and taken as equal to 1.5 times the value computed in accordance with either Eq. 12.8-3 for  $T_L \geq T > 1.5T_L$  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.

<b>SITE SEISMIC PARAMETERS</b>	
Mapped 0.2 sec Period Spectral Acceleration, $S_s$	1.887g
Mapped 1.0 sec Period Spectral Acceleration, $S_1$	0.743g
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, $S_{MS}$	1.887g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, $S_{M1}$	1.263g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, $S_{DS}$	1.258g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, $S_{D1}$	0.842g
Peak Ground Acceleration ( $PGA_M$ )	0.877g
Seismic Design Category	D

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-12, a ground acceleration ( $PGA_M$ ) of 0.877g and a mean earthquake magnitude of 7.3. 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 one inch. It is estimated a seismic differential settlement of 1/2 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 the upper 1 foot of existing materials across the site, and 10 to 20 percent for the natural materials necessitating remedial grading and/or over-excavation 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 0.1 to 0.2 foot is also estimated for the site. Subsidence will occur below the processed removal bottoms at 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 “very low” (0-20) 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:

<b>Design Parameter</b>	<b>“Very Low” Expansion Potential (<math>0 \leq EI \leq 20</math>)</b>
Foundation Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent grade)	12 - One- and -two Stories
Minimum Foundation Width (Inches)*	12
Minimum Slab Thickness (actual)	4 inches
Minimum Slab Reinforcing	6" x 6" – W1.4/W1.4 welded wire fabric placed in middle of slab or No. 3 bars at 24-inch centers.
Minimum Footing Reinforcement	Two No. 4 Reinforcing Bars, one top and one bottom
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum 100% 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 350 psf per foot of depth, to a maximum earth pressure of 2,500 psf for footings founded on engineered fill or competent native soil. A coefficient of friction between soil and concrete of 0.35 may be used with dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third. 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.

<b>ACTIVE EARTH PRESSURES</b>	
Surface Slope of Retained Materials (horizontal:vertical)	Equivalent Fluid Pressure (pcf) Select Backfill* and Native Soils
Level	38
2:1	60

\*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  $\frac{3}{4}$ - 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  $\frac{3}{4}$ - 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 MVS1-100A-1, approved 9/14/18) and using a design R-value of 40, the following preliminary sections were calculated:

<b>PRELIMINARY PAVEMENT SECTIONS</b>			
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-0802021-CR) dated August 6, 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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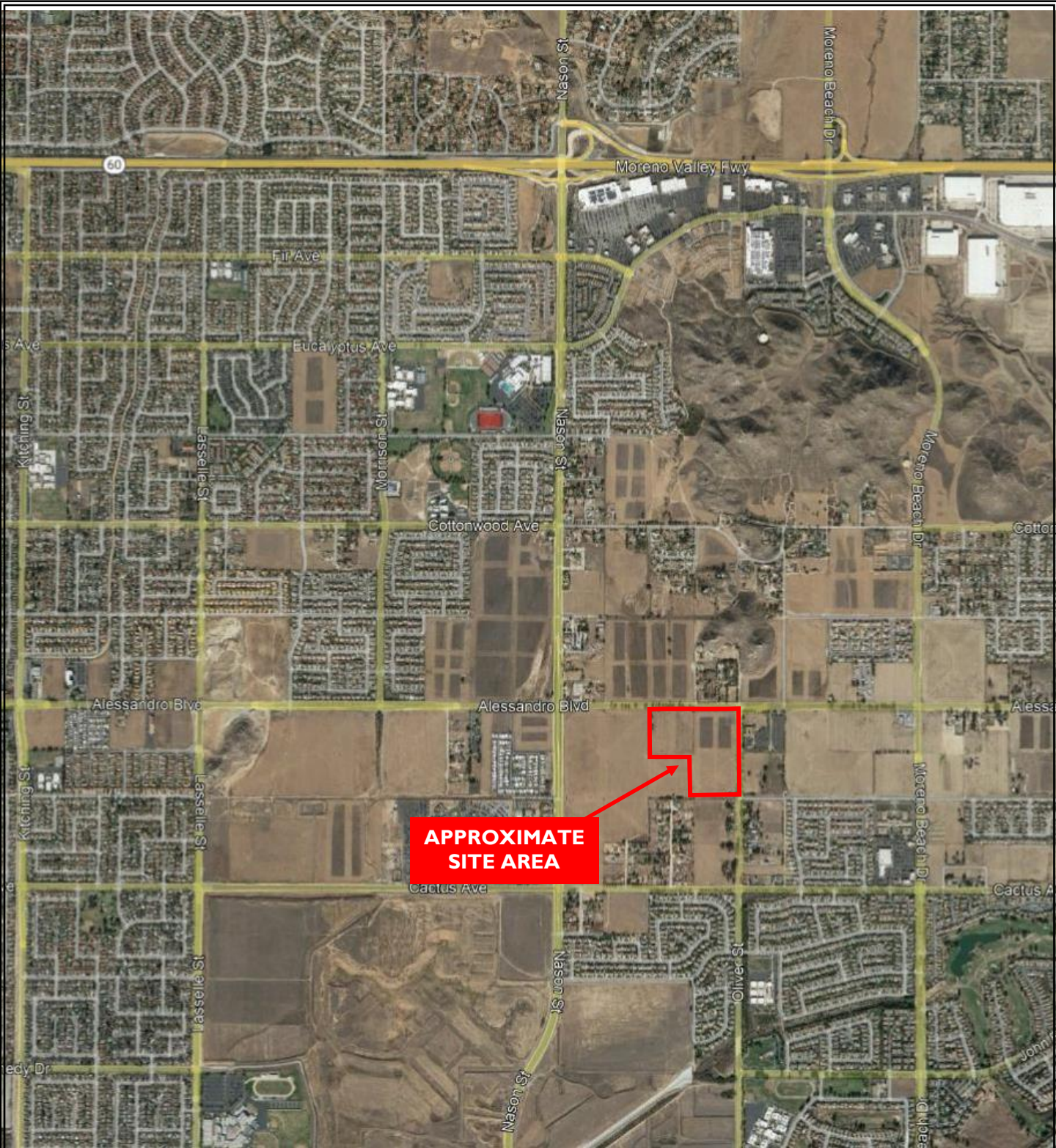
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Modified from Google Earth Pro Aerial Imagery

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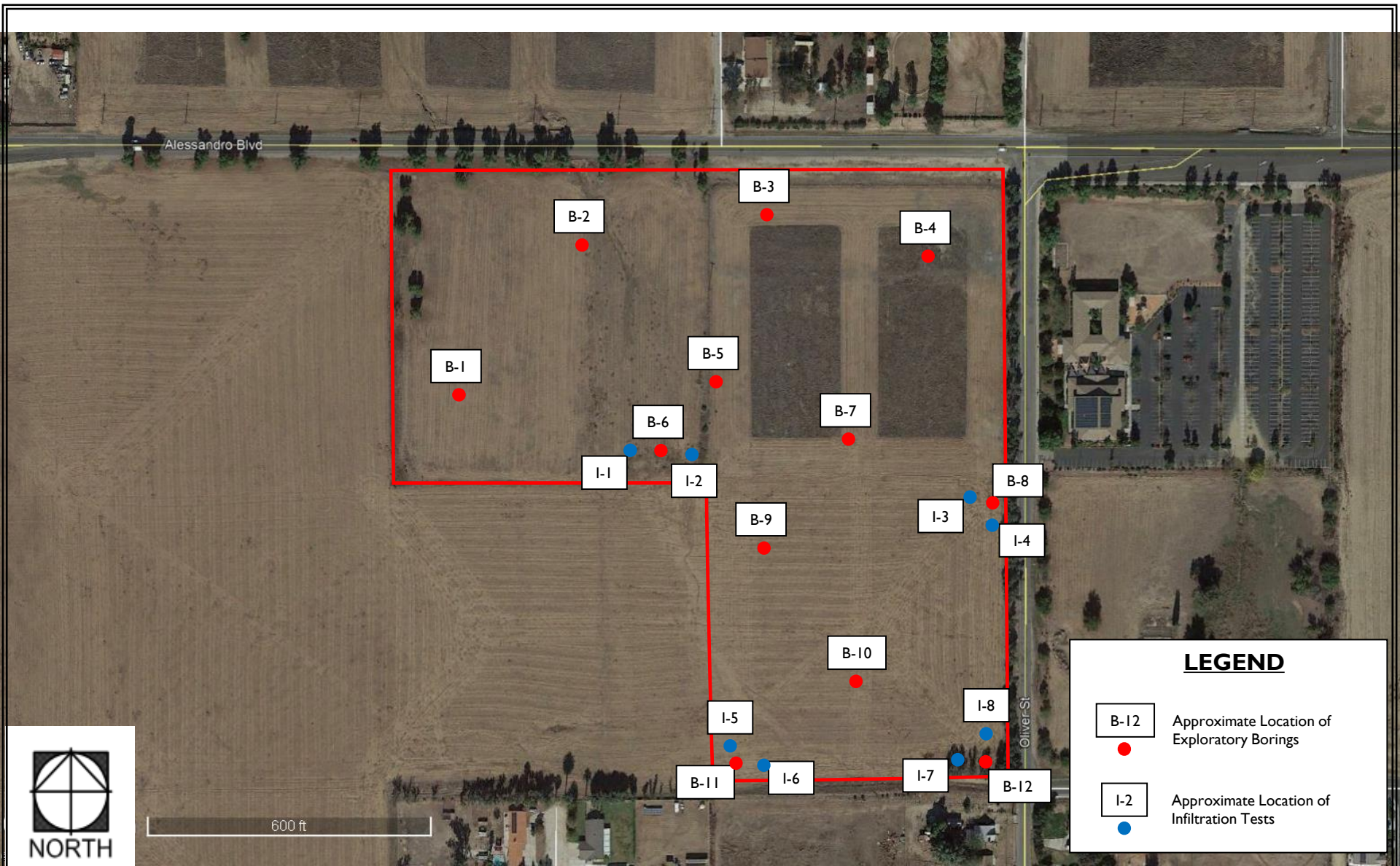
**D.R. Horton Los Angeles Holding Company**  
 Single-Family Residential Tract Development  
 APNs 486-260-003-7 and 486-260-004-8  
 Moreno Valley, Riverside County, California

GeoTek Project No. 2868-CR



**Figure I**  
**Site Location Map**





**LEGEND**

- B-12 ● Approximate Location of Exploratory Borings
- I-2 ● Approximate Location of Infiltration Tests



600 ft

**D.R. Horton Los Angeles Holding Company**  
 Single-Family Residential Tract Development  
 APNs 486-260-003-7 and 486-260-004-8  
 Moreno Valley, Riverside County, California

GeoTek Project No. 2868-CR

Scale As Shown

**Figure 2**  
 Exploration Location Map



**APPENDIX A**

**LOG OF EXPLORATORY BORINGS**

**Single-Family Residential Tract Development  
Alessandro Boulevard and Oliver Street  
Moreno Valley, Riverside County, California  
Project No. 2868-CR**





## **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 LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/ JB  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/3/2021

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-1  MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
0				ML	<b>Alluvium:</b> F sandy SILT, brown, slightly moist, soft			MD SH EI
2		2			F sandy SILT, brown, slightly moist, medium stiff	4.6	114.7	
3		3						
5		5						
5		5		F sandy SILT, brown, slightly moist, stiff	3.8	114.6		
6		6						
7		7						
5		5						
8		8						
12		12						
10		10		SM	Silty f-m SAND, brown, slightly moist, very dense	2.3	124.0	
		22						
		43						
15		10		SM	Silty f SAND, brown, moist, medium dense	8.5	123.1	
		10						
		21						
20		12			Silty f SAND w/ trace clay, brown, slightly moist, dense	4.8	129.6	
		25						
		37						
<b>BORING TERMINATED AT 21.5 FEET</b>								
No groundwater encountered Boring backfilled with spoils								
25								
30		9						
		12						
		16						

**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  
SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/JB  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/3/2021

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-2	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0				ML	<b>Alluvium:</b> Sandy SILT, brown, slightly moist, medium stiff			MD SH EI  HC
5		5 6 9 4 5 11				Same as above	2.9	
10		8 9 15		SM-ML	Silty f SAND to sandy SILT, brown, slightly moist, medium dense/stiff	4.4	112.3	
15		12 22 22		SM	Silty f SAND, brown, slightly moist, dense	4.6	124.5	
20		10 16 25			Silty f-m SAND with trace CLAY, brown, slightly moist, dense	5.6	130.8	
<b>BORING TERMINATED AT 16.5 FEET</b>								
25					No groundwater encountered Boring backfilled with spoils			
30		9 12 16						

**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  
SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/JB  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/3/2021

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-3	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0				ML	<b>Alluvium:</b> Sandy SILT, brown, dry to slightly moist, very stiff	3.5	114.0	MD SH EI
		5 14 23						
		10 29 17		SM	Silty f-m SAND, brown, slightly moist, dense	4.6	113.6	
5		5 8 11		SP	F SAND with SILT, brown, slightly moist, medium dense			
		9 20 30		SM-SC	Silty to clayey f SAND, brown, moist, dense	8.9	131.1	
10		12 18 28		SM	Silty f SAND, light brown/gray, slightly moist, dense, trace gravel	1.8	123.4	
15	<b>BORING TERMINATED AT 16.5 FEET</b>							
					No groundwater encountered Boring backfilled with spoils			
20								
25								
30		9 12 16						

**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  
SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/JB  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/3/2021

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-4	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0				ML	<b>Alluvium:</b> Sandy SILT w/ trace clay, brown, slightly moist, stiff			MD SH EI
5		4 5 11		SM	Silty f SAND, brown, moist, medium dense	6.9	123.1	
		11 18 16			Silty f-m SAND, brown/gray, slightly moist, medium dense	4.3	120.6	
10		8 10 14		SM	Silty f SAND brown, slightly moist, dense	6.9	126.4	
		16 20 26			Silty f-m SAND, brown/gray, slightly moist, medium dense	3.8	124.5	
15		12 18 20			<b>BORING TERMINATED AT 16.5 FEET</b>			
20					No groundwater encountered Boring backfilled with spoils			
25								
30		9 12 16						

**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  
SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/JB  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/3/2021

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-5	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)
0				ML	<b>Alluvium:</b> Sandy SILT, brown/gray, dry, stiff, trace gravel			MD SH EI
6		6						
7		7						
7								
4		4			Sandy SILT, brown, slightly moist to moist, stiff	4.5	101.9	HC
4		4						
7		7						
5				SM-ML	Silty f SAND to sandy SILT, dark brown, slightly moist, medium dense/stiff			
4		4						
7		7						
9		9						
10				SP	F SAND with trace SILT, brown/gray, slightly moist, medium dense	2.3	121.2	
9		9						
9		9						
15				ML	Sandy SILT, dark brown, moist, very stiff	8.7	99.7	
14		14						
19		19						
22		22						
<b>BORING TERMINATED AT 16.5 FEET</b>								
No groundwater encountered Boring backfilled with spoils								
20								
25								
30								
9		9						
12		12						
16		16						

**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  
SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/CD  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/2/2021

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-6	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0				SM	<b>Alluvium:</b> Silty f SAND, brown, dry, medium dense			MD SH EI
		14 14 11				1.6	116.9	
5		10 11 14		SM-ML	Silty f SAND to Sandy SILT, brown, slightly moist, medium dense/stiff			
		6 7 8		SM	F sandy SILT, brown, slightly moist, medium stiff	6.5	117.9	HC
10		6 7 8		SM-ML	F silty SAND to sandy SILT, brown, moist, medium stiff	8.7	124.6	
15		5 7 9		SM-SC	Silty to clayey f SAND, brown, moist, medium stiff			
20		10 15 22		SM	Silty f SAND with CLAY, brown, moist, very stiff	8.8	127.7	
<b>BORING TERMINATED AT 21.5 FEET</b>								
					No groundwater encountered Boring backfilled with spoils			
25								
30		9 12 16						

**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  
SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/JB  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/3/2021

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-7	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0				ML	<b>Alluvium:</b> F sandy SILT, brown, dry, stiff	3.7	102.0	MD SH EI
		6 11 12						
		9 7 10			F sandy SILT brown, slightly moist to moist, stiff	4.4	108.8	
5				SM	Silty f SAND, brown, slightly moist, medium dense			
		5 10 20						
10				SM	Silty f-c SAND, light brown/gray, slightly moist, medium dense	3.8	120.5	
		10 17 20						
15				SM	Silty f SAND with trace CLAY, brown, moist, dense	8.8	132.2	
		10 22 30						
20				SP	F-m SAND, lightly brown/gray, slightly moist, medium dense, trace gravel	2.2	115.8	
		11 18 19						
<b>BORING TERMINATED AT 21.5 FEET</b>								
					No groundwater encountered Boring backfilled with spoils			
25								
30								
		9 12 16						

**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  
SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density



**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/CD  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/2/2021

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-8	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0				ML	<b>Alluvium:</b> Sandy SILT, brown, slightly moist, stiff, trace organics			MD SH EI
		9 14 18			Same, very stiff	4.0	106.9	
		16 22 50/5			Sandy SILT, brown, slightly moist, hard, trace organics	2.9	121.8	
5				SM	Silty f SAND, brown/gray, slightly moist, dense, trace gravel			
		38 28 21						
		10 16 26			Same as above	2.0	117.2	
15					Silty f SAND, brown/gray, slightly moist, very dense	4.8	113.8	
		18 50/5						
20					Silty f-m SAND, brown/gray, slightly moist, very dense	4.7	113.5	
		18 50/6						
<b>BORING TERMINATED AT 21 FEET</b>								
					No groundwater encountered Boring backfilled with spoils			
25								
		9 12 16						
30								

**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  
SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/CD  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/2/2021

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-9	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0				SM	<b>Alluvium:</b> Silty f SAND, brown/gray, slightly moist, medium dense	2.2	113.1	MD SH EI
		8 12 14						
		8 6 8			Silty f SAND, light brown, slightly moist, medium dense, trace pinhole pores and root hairs	2.4	116	
5				SM-ML	Silty f SAND to sandy SILT, brown, slightly moist, medium dense/stiff			
		7 9 12						
		6 8 11		ML	F sandy SILT, brown, slightly moist to moist, stiff	5.0	108.9	HC
10				SM	Silty f SAND, brown, slightly moist to moist, medium dense	5.0	126.8	
		10 15 15						
15					Silty f SAND, brown, slightly moist to moist, dense	4.3	128.3	
		15 24 30						
<b>BORING TERMINATED AT 16.5 FEET</b>								
					No groundwater encountered Boring backfilled with spoils			
20								
25								
30								
		9 12 16						

**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  
SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/CD  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/2/2021

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-10	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0				ML	<b>Alluvium:</b> F sandy SILT, yellow/brown, slightly moist, stiff			MD SH EI
		7			Same as above	2.9	109.1	
		10						
		10						
		7			F sandy SILT, yellow/brown, slightly moist, stiff	3.5	111.5	
		9						
		12						
		5			Same as above			
		8						
		9						
10		18		SM-ML	Silty f SAND to sandy SILT, yellow/brown, slightly moist, dense/hard	5.6	121.4	
		27						
		32						
15		15		SM	Silty f SAND, yellow/brown, moist, dense	6.8	122.9	
		30						
		29						
20		7			Silty f-m SAND, brown, moist, dense	5.9	127.3	
		17						
		22						
<b>BORING TERMINATED AT 21.5 FEET</b>								
					No groundwater encountered Boring backfilled with spoils			
25								
30		9						
		12						
		16						

**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  
SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/CD  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/2/2021

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-11  MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing					
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others			
0				ML	<b>Alluvium:</b> F sandy SILT, light brown, slightly moist, stiff	3.6	106.7	MD SH EI			
6			Same as above						3.8	107.6	HC
7											
10				SM	Silty f-c SAND, gray/brown, slightly moist, medium dense						
15			SM					Silty f SAND, dark brown, moist, dense, micaceous	5.0	125.3	
20											
21.5	<b>BORING TERMINATED AT 21.5 FEET</b>										
25	No groundwater encountered Boring backfilled with spoils										
30											

**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  
SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/CD  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/2/2021

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-12  MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
0				ML	<b>Alluvium:</b> F sandy SILT, brown, dry, very stiff			MD SH EI
		9 16 18			Same, slightly moist	4.0	109.4	
5		12 16 19			F sandy SILT, brown, slightly moist, very stiff	4.3	110.1	
		14 18 24			Same as above			
10		15 26 32		SM	Silty f-m SAND, brown, slightly moist, dense	3.2	120.6	
15		30 50/5		ML	F-m sandy SILT, brown, slightly moist, hard	3.7	116.7	
20		50/6			Same as above	4.4	114.6	
25		50/5		SM	Silty f-m SAND, brown, moist, very dense	6.8	122.8	
30		9 12 16			Silty f-m SAND, light brown, slightly moist, medium dense	2.0		

**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  
SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/CD  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/2/2021

Depth (ft)	SAMPLES			USCS Symbol	Boring No.: B-12 (continued)	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number			Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
35	7 14 14			ML	F sandy SILT, light brown, slightly moist, very stiff			MD SH EI
40	6 11 14				Clayey SILT, light brown/gray, slightly moist, very stiff			
45	7 15 27				F sandy SILT, light brown/gray, slightly moist, hard			
50	20 21 20			SM	Silty f-c SAND, light brown, slightly moist, dense	3.8		
<b>BORING TERMINATED AT 51.5 FEET</b>								
55					No groundwater encountered Boring backfilled with spoils			
60	9 12 16							

**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  
SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/CD  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/2/2021

Depth (ft)	SAMPLES				Boring No.: I-1	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol		Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0				SM	<b>Alluvium:</b> Silty f-c SAND, brown, dry, medium dense			MD SH EI
5					<b>BORING TERMINATED AT 5 FEET</b>			
					No groundwater encountered Boring prepped with pipe, filter sock and gravel for infiltration testing			
10								
15								
20								
25								
30		9 12 16						

<b>LEGEND</b>	<b>Sample type:</b>	<input type="checkbox"/>	---Ring	<input type="checkbox"/>	---SPT	<input checked="" type="checkbox"/>	---Small Bulk	<input checked="" type="checkbox"/>	---Large Bulk	<input type="checkbox"/>	---No Recovery	<input type="checkbox"/>	---Water Table
	<b>Lab testing:</b>	AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test	SR = Sulfate/Resistivity Test	SH = Shear Test	HC = Consolidation	MD = Maximum Density				

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/CD  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/2/2021

Depth (ft)	SAMPLES				Boring No.: I-2	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol		Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0				SM	<b>Alluvium:</b> Silty f-c SAND, brown, dry, medium dense			MD SH EI
5					<b>BORING TERMINATED AT 5 FEET</b>			
					No groundwater encountered Boring prepped with pipe, filter sock and gravel for infiltration testing			
10								
15								
20								
25								
30		9 12 16						

**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  
SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density



**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/CD  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/2/2021

Depth (ft)	SAMPLES				Boring No.: I-3	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol		Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0				ML	<b>Alluvium:</b> Sandy SILT, brown, slightly moist, stiff			MD SH EI
5					<b>BORING TERMINATED AT 5 FEET</b>			
10					No groundwater encountered Boring prepped with pipe, filter sock and gravel for infiltration testing			
15								
20								
25								
30		9 12 16						

**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  
SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/CD  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/2/2021

Depth (ft)	SAMPLES				USCS Symbol	Boring No.: I-4  MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol			Water Content (%)	Dry Density (pcf)	Others
0					ML	<b>Alluvium:</b> Sandy SILT, brown, slightly moist, stiff			MD SH EI
5							<b>BORING TERMINATED AT 5 FEET</b>  No groundwater encountered Boring prepped with pipe, filter sock and gravel for infiltration testing		
10									
15									
20									
25									
30		9 12 16							

**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  
 SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/CD  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/2/2021

Depth (ft)	SAMPLES				Boring No.: I-5	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol		Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0				ML	<b>Alluvium:</b> F sandy SILT, light brown, slightly moist, medium stiff			MD SH EI
5					<b>BORING TERMINATED AT 5 FEET</b>			
					No groundwater encountered Boring prepped with pipe, filter sock and gravel for infiltration testing			
10								
15								
20								
25								
30		9 12 16						

**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  
 SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/CD  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/2/2021

Depth (ft)	SAMPLES				Boring No.: I-6	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol		Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0				ML	<b>Alluvium:</b> F sandy SILT, light brown, slightly moist, medium stiff			MD SH EI
5					<b>BORING TERMINATED AT 5 FEET</b>			
					No groundwater encountered Boring prepped with pipe, filter sock and gravel for infiltration testing			
10								
15								
20								
25								
30		9 12 16						

<b>LEGEND</b>	<b>Sample type:</b>	<input type="checkbox"/>	---Ring	<input type="checkbox"/>	---SPT	<input checked="" type="checkbox"/>	---Small Bulk	<input checked="" type="checkbox"/>	---Large Bulk	<input type="checkbox"/>	---No Recovery	<input checked="" type="checkbox"/>	---Water Table
	<b>Lab testing:</b>	AL = Atterberg Limits	EI = Expansion Index	SA = Sieve Analysis	RV = R-Value Test	SR = Sulfate/Resistivity Test	SH = Shear Test	HC = Consolidation	MD = Maximum Density				

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/CD  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/2/2021

Depth (ft)	SAMPLES				Boring No.: I-7	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol		Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0				ML	<b>Alluvium:</b> F sandy SILT, brown, dry, stiff			MD SH EI
5					<b>BORING TERMINATED AT 5 FEET</b>			
					No groundwater encountered Boring prepped with pipe, filter sock and gravel for infiltration testing			
10								
15								
20								
25								
30		9 12 16						

**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  
 SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**GeoTek, Inc.**  
**LOG OF EXPLORATORY BORING**

**CLIENT:** DR Horton LA Holding Co.  
**PROJECT NAME:** Whitney 162 & Valley Church  
**PROJECT NO.:** 2868-CR  
**LOCATION:** Moreno Valley, CA

**DRILLER:** 2R Drilling  
**DRILL METHOD:** Hollow Stem  
**HAMMER:** 140#/30"

**LOGGED BY:** GP/CD  
**OPERATOR:** Jeff/Matt  
**RIG TYPE:** CME 75  
**DATE:** 9/2/2021

Depth (ft)	SAMPLES				Boring No.: I-8	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol		Water Content (%)	Dry Density (pcf)	Others
<b>MATERIAL DESCRIPTION AND COMMENTS</b>								
0				ML	<b>Alluvium:</b> F sandy SILT, brown, dry, stiff			MD SH EI
5					<b>BORING TERMINATED AT 5 FEET</b>			
					No groundwater encountered Boring prepped with pipe, filter sock and gravel for infiltration testing			
10								
15								
20								
25								
30		9 12 16						

**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  
 SR = Sulfate/Resistivity Test      SH = Shear Test      HC = Consolidation      MD = Maximum Density

**APPENDIX B**

**RESULTS OF LABORATORY TESTING**

**Single-Family Residential Tract Development  
Alessandro Boulevard and Oliver Street  
Moreno Valley, Riverside County, California  
Project No. 2868-CR**



## 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

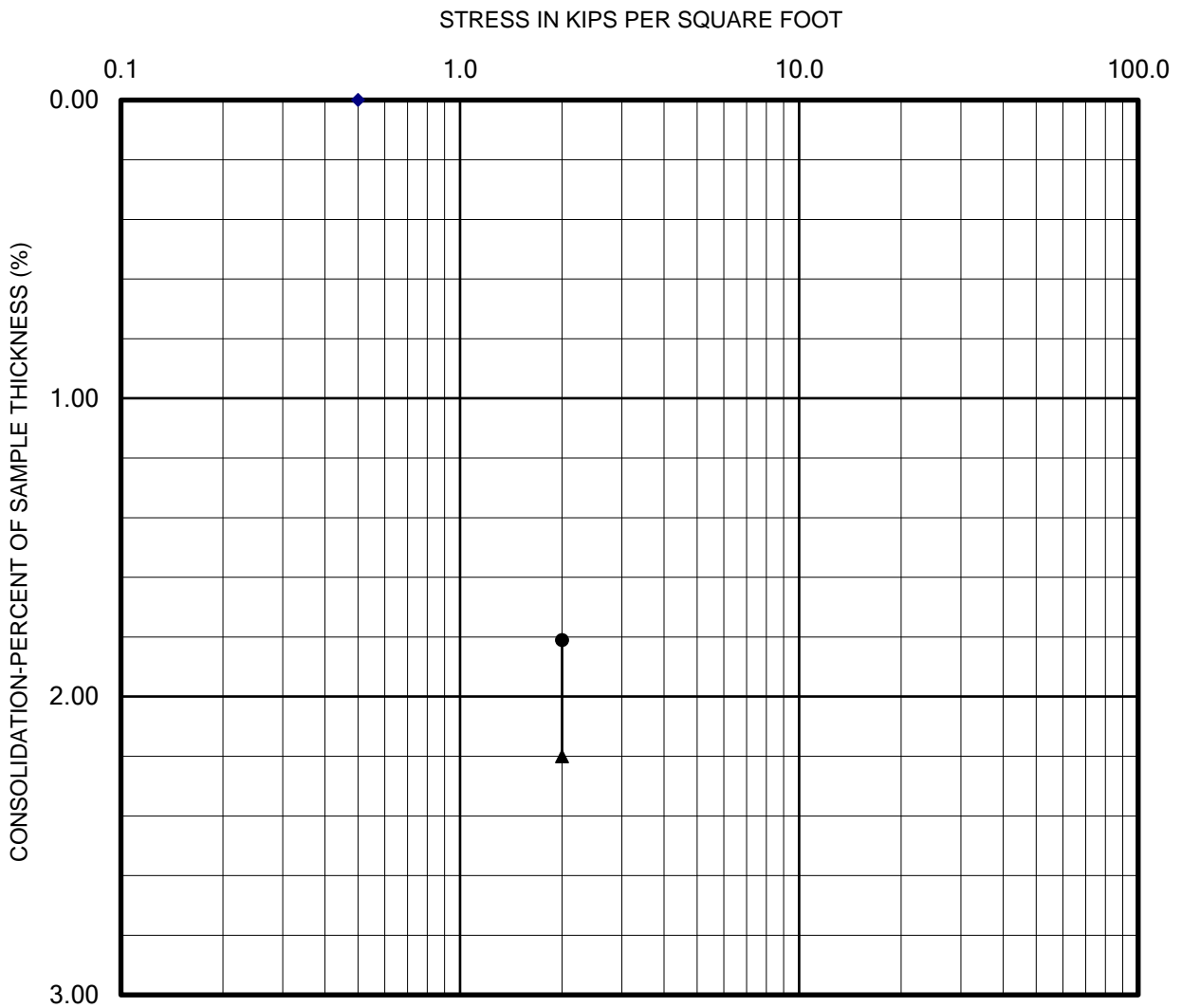
### In-Situ Moisture and Density

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.





- Seating Cycle
- Loading Prior to Inundation
- ▲— Loading After Inundation
- ▲--- Rebound Cycle

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546



## COLLAPSE REPORT

**Sample: B-2 @ 4 feet**

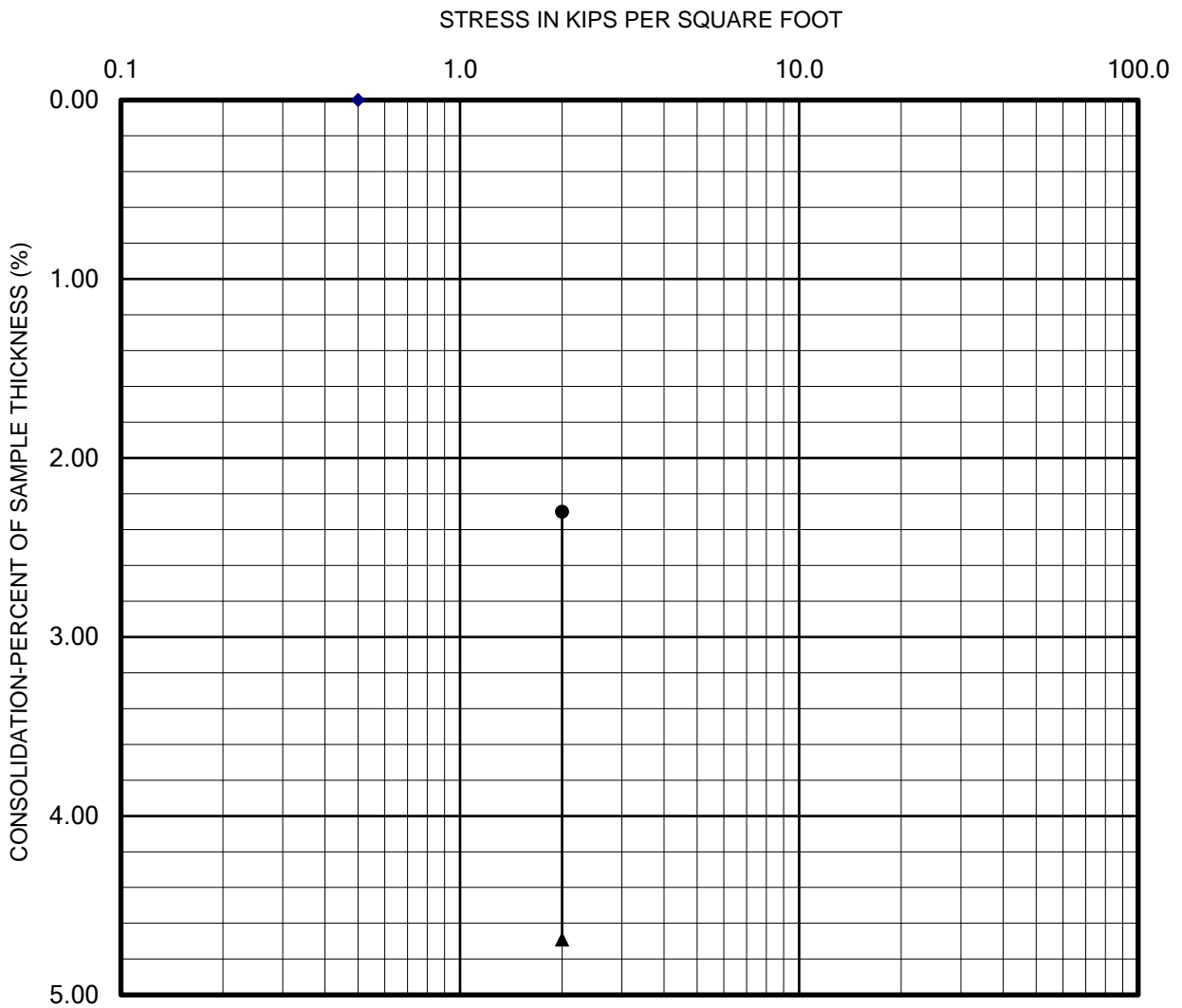
**Plate B-2**

CHECKED BY: RJ

Lab: Corona

PROJECT NO.: 2868-CR

Date: 9-15-21



- Seating Cycle
- Loading Prior to Inundation
- ▲— Loading After Inundation
- ▲--- Rebound Cycle

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546



## COLLAPSE REPORT

**Sample: B-5 @ 3 feet**

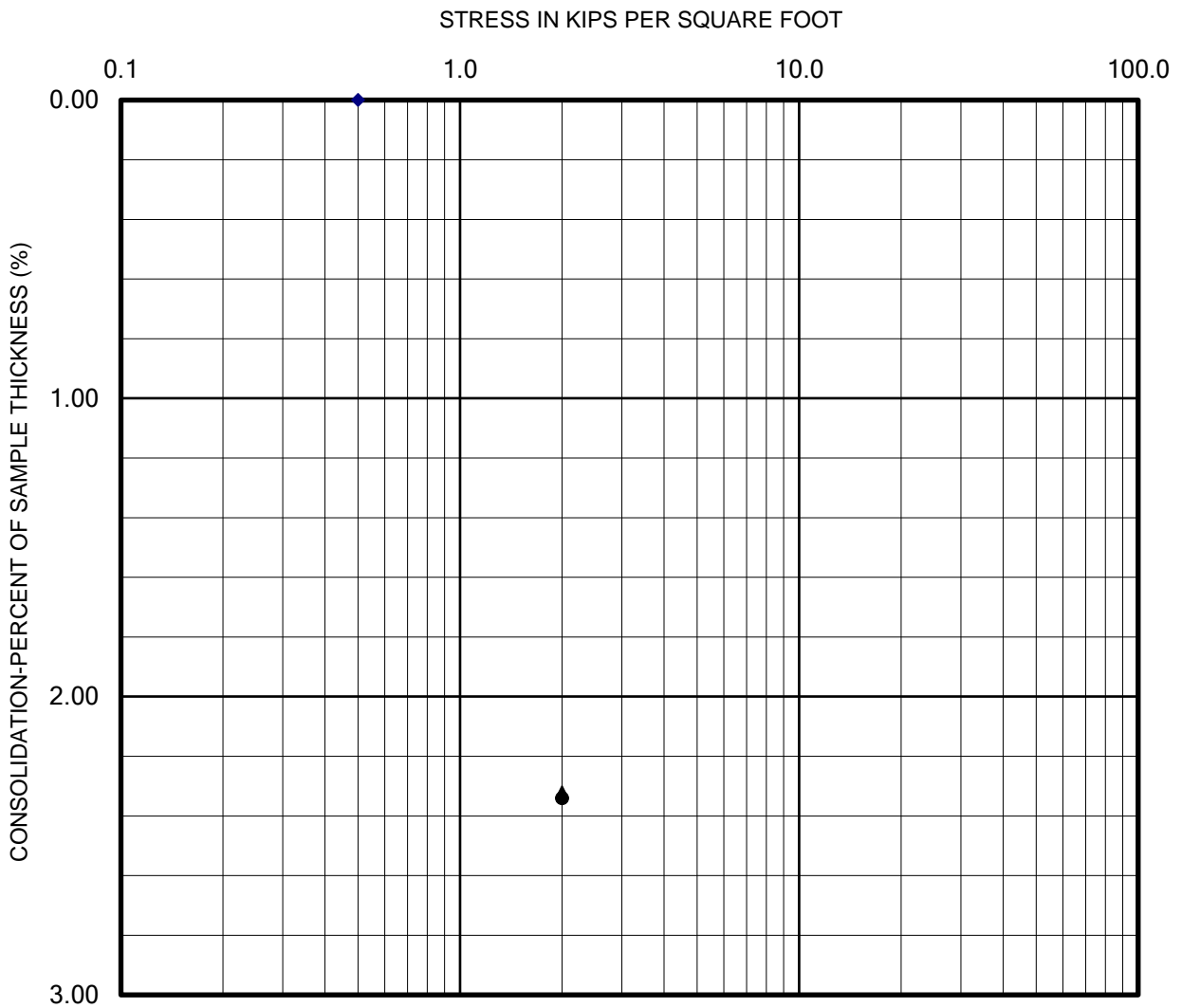
**Plate B-1**

CHECKED BY: RJ

Lab: Corona

PROJECT NO.: 2868-CR

Date: 9-15-21



- Seating Cycle
- Loading Prior to Inundation
- ▲— Loading After Inundation
- ▲--- Rebound Cycle

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546



## COLLAPSE REPORT

**Sample: B-6 @ 6 feet**

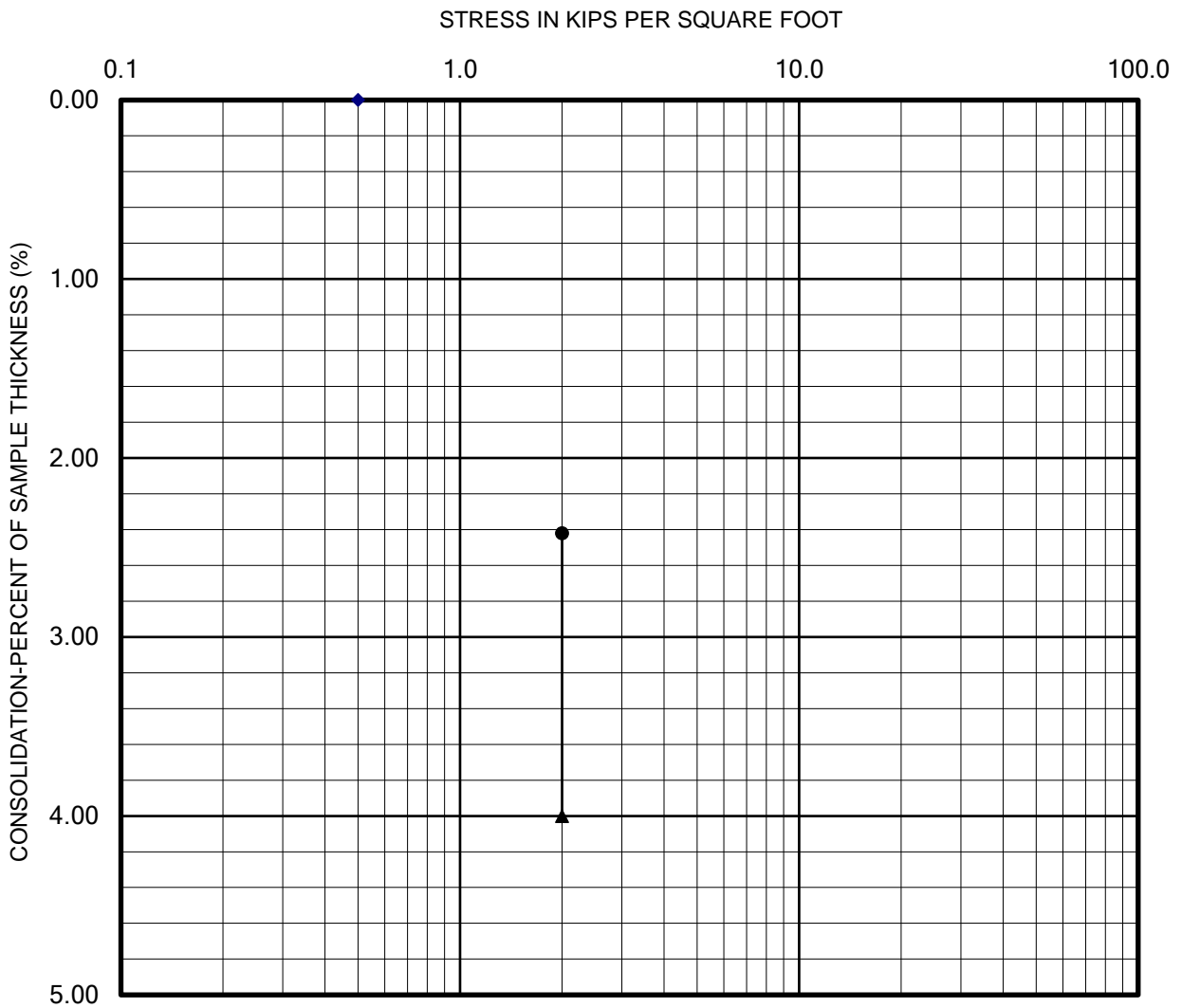
**Plate B-4**

CHECKED BY: RJ

Lab: Corona

PROJECT NO.: 2868-CR

Date: 9-15-21



- Seating Cycle
- Loading Prior to Inundation
- ▲— Loading After Inundation
- ▲--- Rebound Cycle

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546



## COLLAPSE REPORT

**Sample: B-9 @ 7 feet**

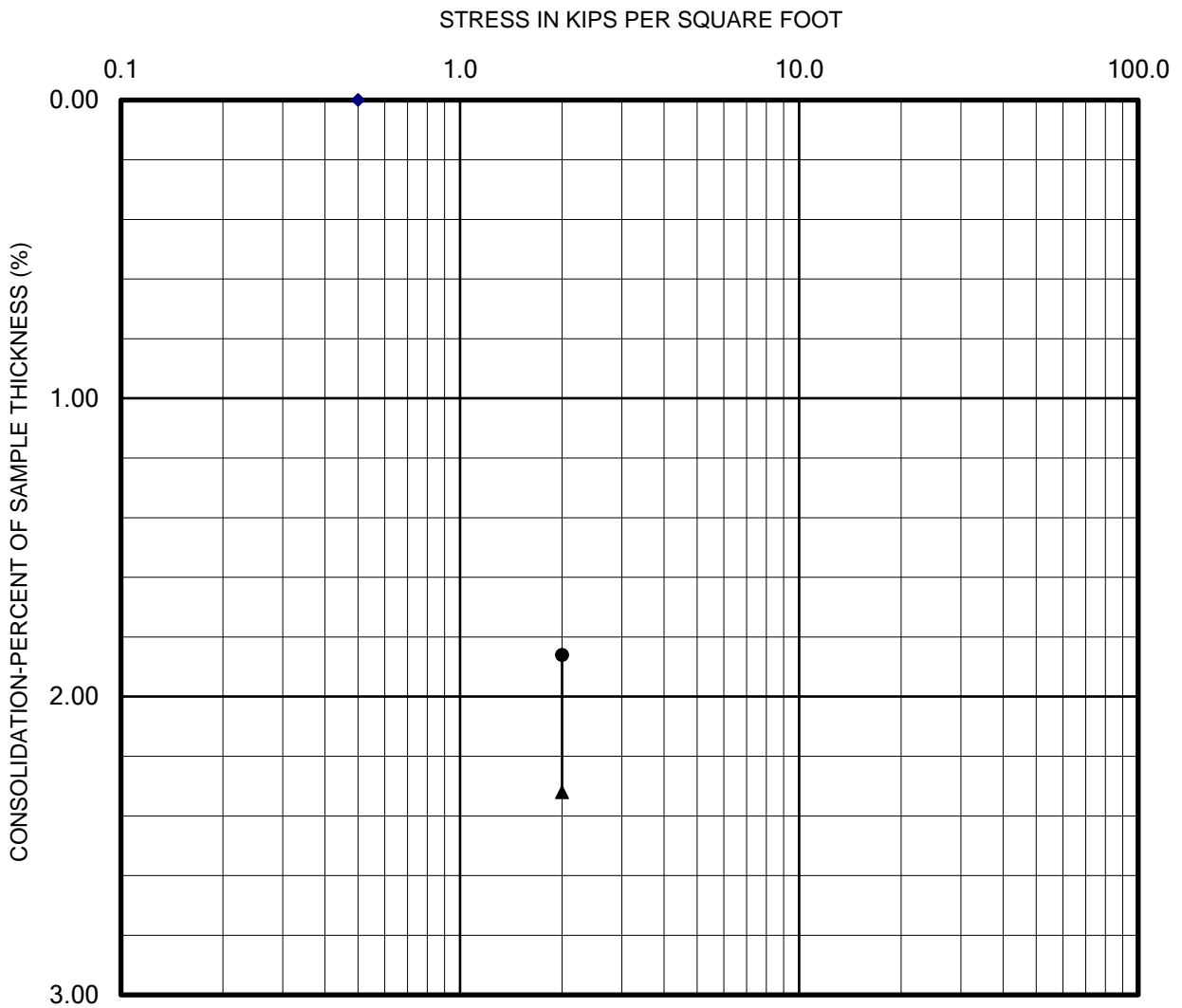
**Plate B-5**

CHECKED BY: RJ

Lab: Corona

PROJECT NO.: 2868-CR

Date: 9-15-21



- Seating Cycle
- Loading Prior to Inundation
- ▲— Loading After Inundation
- ▲--- Rebound Cycle

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546



## COLLAPSE REPORT

**Sample: B-11 @ 5 feet**

**Plate B-3**

CHECKED BY: RJ

Lab: Corona

PROJECT NO.: 2868-CR

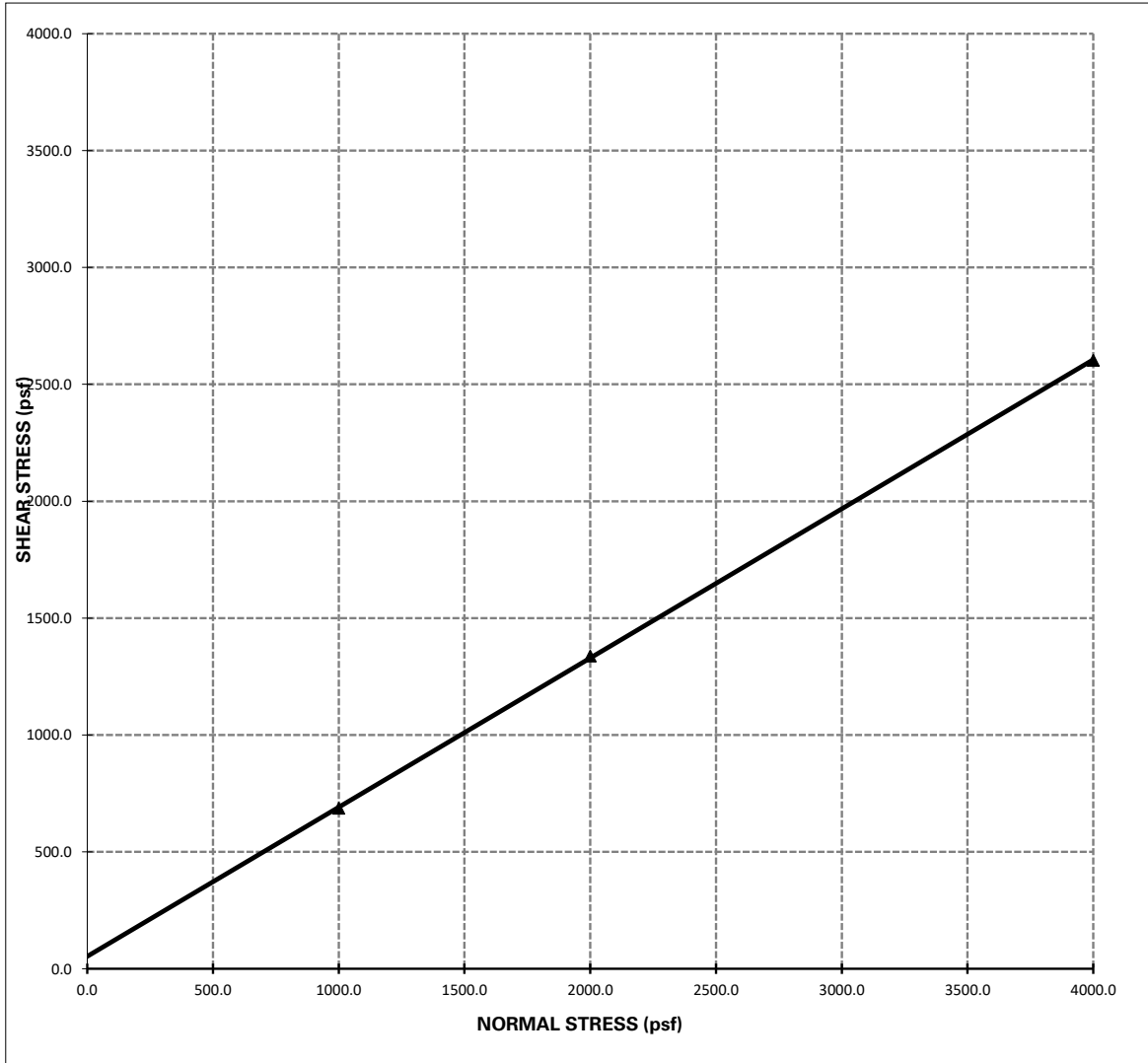
Date: 9-15-21



# DIRECT SHEAR TEST

**Project Name:** DR Horton  
**Project Number:** 2868-CR

**Sample Location:** B2 @ 0-5'  
**Date Tested:** 9/20/2021



**Shear Strength:**  $\Phi = 33^\circ$  ; **C = 53 psf**

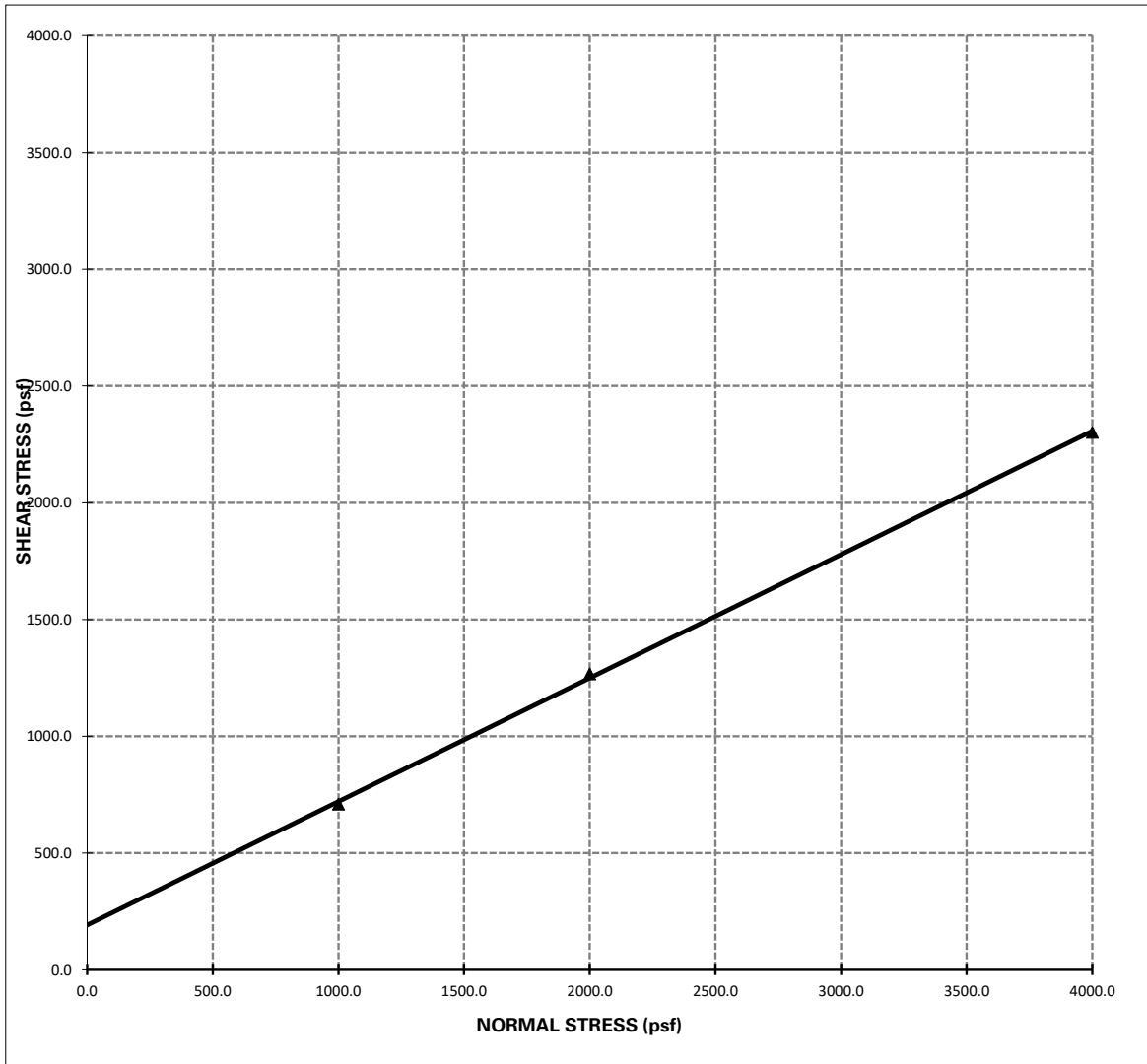
- Notes:**
- 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.
  - 2 - The above reflect direct shear strength at saturated conditions.
  - 3 - The tests were run at a shear rate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



# DIRECT SHEAR TEST

**Project Name:** DR Horton  
**Project Number:** 2868-CR

**Sample Location:** B12 @ 0-5'  
**Date Tested:** 9/20/2021



**Shear Strength:**  $\Phi = 28^{\circ}$  ; **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.



# EXPANSION INDEX TEST

(ASTM D4829)

**Client:** DR Horton CD Lab No Corona  
**Project Number:** 2868-CR Date Tested: 9/17/2021  
**Project Location:** Whitney 162 & Valley Church Moreno Valley ,CA Sample Source: B2 @ 0-5'  
Sample Description: \_\_\_\_\_

Ring #: \_\_\_\_\_ Ring Dia. : 4.01" Ring Ht.: 1"

### DENSITY DETERMINATION

<b>A</b>	Weight of compacted sample & ring (gm)	788.1
<b>B</b>	Weight of ring (gm)	362.0
<b>C</b>	Net weight of sample (gm)	<b>426.1</b>
<b>D</b>	Wet Density, lb / ft3 (C*0.3016)	<b>128.5</b>
<b>E</b>	Dry Density, lb / ft3 (D/1.F)	<b>119.2</b>

### SATURATION DETERMINATION

<b>F</b>	Moisture Content, %	7.8
<b>G</b>	Specific Gravity, assumed	<b>2.70</b>
<b>H</b>	Unit Wt. of Water @ 20 °C, (pcf)	<b>62.4</b>
<b>I</b>	% Saturation	<b>51.0</b>

READINGS		
DATE	TIME	READING
9/17/2021		0.1330
9/17/2021		0.1320
9/18/2021		0.1330

Initial  
10 min/Dry

Final

FINAL MOISTURE	
Final Weight of wet sample & tare	
807.7	% Moisture <b>12.4</b>

**EXPANSION INDEX = 1**





# EXPANSION INDEX TEST

(ASTM D4829)

**Client:** DR Horton CD Lab No Corona  
**Project Number:** 2868-CR Date Tested: 9/17/2021  
**Project Location:** Whitney 162 & Valley Church Moreno Valley ,CA Sample Source: B12 @ 0-5'  
Sample Description: \_\_\_\_\_

Ring #: \_\_\_\_\_ Ring Dia. : 4.01" Ring Ht.: 1.1"

### DENSITY DETERMINATION

<b>A</b>	Weight of compacted sample & ring (gm)	783.0
<b>B</b>	Weight of ring (gm)	362.2
<b>C</b>	Net weight of sample (gm)	<b>420.8</b>
<b>D</b>	Wet Density, lb / ft3 (C*0.3016)	<b>126.9</b>
<b>E</b>	Dry Density, lb / ft3 (D/1.F)	<b>117.7</b>

### SATURATION DETERMINATION

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

READINGS		
DATE	TIME	READING
9/17/2021		0.1480
9/17/2021		0.1490
9/18/2021		0.1510

Initial  
10 min/Dry

Final

FINAL MOISTURE	
Final Weight of wet sample & tare	
808.2	% Moisture
	<b>13.8</b>

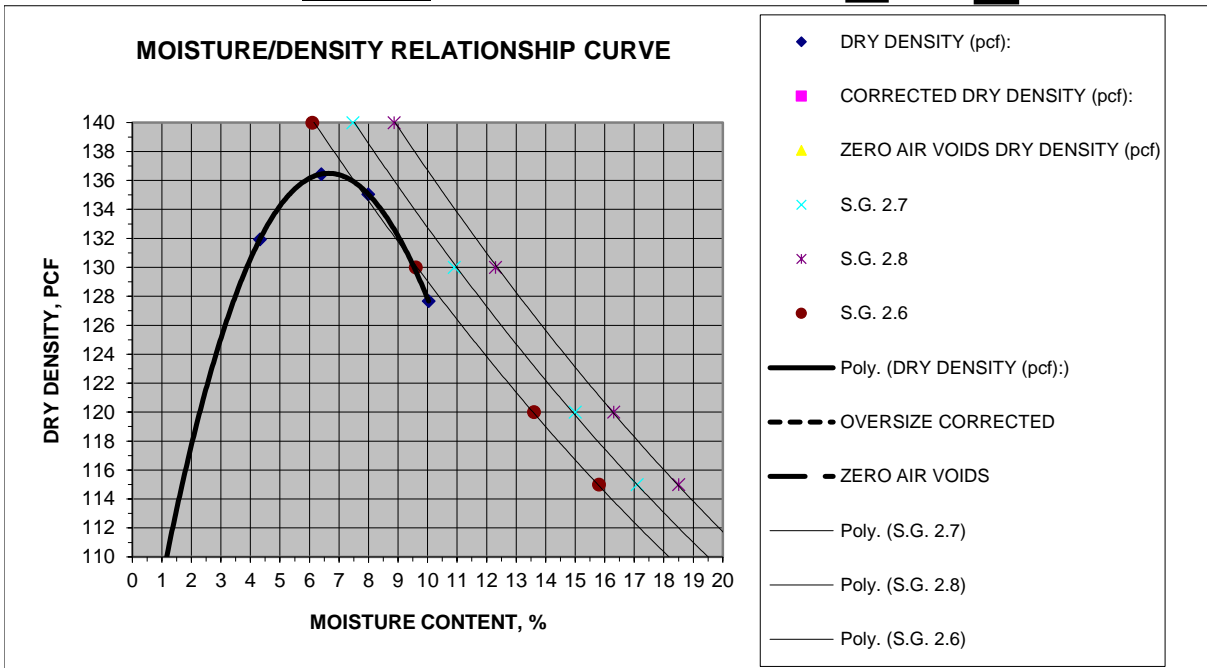
**EXPANSION INDEX = 2**



## MOISTURE/DENSITY RELATIONSHIP

<b>Client:</b> DR Horton <b>Project:</b> Whitney 162 & Valley Church <b>Location:</b> Moreno Valley <b>Material Type:</b> Brown Silty Sand <b>Material Supplier:</b> - <b>Material Source:</b> - <b>Sample Location:</b> B2 @ 0-5'  <b>Sampled By:</b> GP/CD/JB <b>Received By:</b> RJ <b>Tested By:</b> AD <b>Reviewed By:</b> RJ	<b>Job No.:</b> 2868-CR <b>Lab No.:</b> Corona  <b>Date Sampled:</b> 9/7/2021 <b>Date Received:</b> 9/7/2021 <b>Date Tested:</b> 9/16/2021 <b>Date Reviewed:</b> 9/17/2021
---	--

**Test Procedure:** ASTM D1557      **Method:** A  
**Oversized Material (%):** 1.7      **Correction Required:**  yes  no



### MOISTURE DENSITY RELATIONSHIP VALUES

<b>Maximum Dry Density, pcf</b> <input style="width: 80px;" type="text" value="136.5"/>	<b>@ Optimum Moisture, %</b> <input style="width: 80px;" type="text" value="6.5"/>
<b>Corrected Maximum Dry Density, pcf</b> <input style="width: 80px;" type="text"/>	<b>@ Optimum Moisture, %</b> <input style="width: 80px;" type="text"/>

### 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:

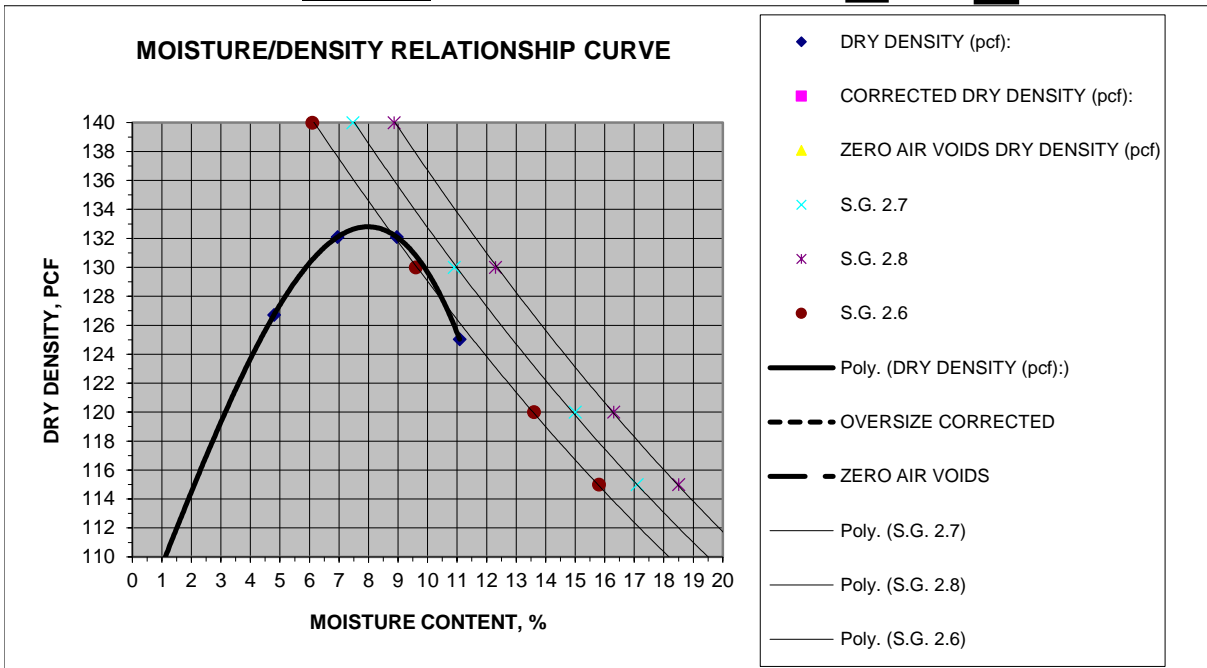
Unified Soils Classification: \_\_\_\_\_  
 AASHTO Soils Classification: \_\_\_\_\_



## MOISTURE/DENSITY RELATIONSHIP

<b>Client:</b> DR Horton <b>Project:</b> Whitney 162 & Valley Church <b>Location:</b> Moreno Valley <b>Material Type:</b> Brown Silty Sand <b>Material Supplier:</b> - <b>Material Source:</b> - <b>Sample Location:</b> B12 @ 0-5'  <b>Sampled By:</b> GP/CD/JB <b>Received By:</b> RJ <b>Tested By:</b> AD <b>Reviewed By:</b> RJ	<b>Job No.:</b> 2868-CR <b>Lab No.:</b> Corona  <b>Date Sampled:</b> 9/7/2021 <b>Date Received:</b> 9/7/2021 <b>Date Tested:</b> 9/16/2021 <b>Date Reviewed:</b> 9/17/2021
--	--

**Test Procedure:** ASTM D1557      **Method:** A  
**Oversized Material (%):** 0.8      **Correction Required:**  yes  no



### MATERIAL DESCRIPTION

**Grain Size Distribution:**

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

**Classification:**

Unified Soils Classification: \_\_\_\_\_  
 AASHTO Soils Classification: \_\_\_\_\_

**Atterberg Limits:**

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

**APPENDIX C**

**PERCOLATION DATA SHEETS & PORCHET CALCULATIONS**

**Single-Family Residential Tract Development  
Alessandro Boulevard and Oliver Street  
Moreno Valley, Riverside County, California  
Project No. 2868-CR**



**Client:** D. R. Horton  
**Project:** Whitney I 62/Valley Church  
**Project No:** 2868-CR  
**Date:** 9/3/2021

**Boring No.** I-I

**Infiltration Rate (Porchet Method)**

Time Interval,  $\Delta t =$  10  
 Final Depth to Water,  $D_F =$  43  
 Test Hole Radius,  $r =$  4  
 Initial Depth to Water,  $D_O =$  40  
 Total Test Hole Depth,  $D_T =$  60

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

$H_O = D_T - D_O =$  20  
 $H_F = D_T - D_F =$  17  
 $\Delta H = \Delta D = H_O - H_F =$  3  
 $H_{avg} = (H_O + H_F) / 2 =$  18.5

$I_t =$  1.76 Inches pe

**Client:** D. R. Horton  
**Project:** Whitney I 62/Valley Church  
**Project No:** 2868-CR  
**Date:** 9/3/2021

**Boring No.** I-2

**Infiltration Rate (Porchet Method)**

Time Interval,  $\Delta t =$  30  
Final Depth to Water,  $D_F =$  46.5  
Test Hole Radius,  $r =$  4  
Initial Depth to Water,  $D_O =$  40  
Total Test Hole Depth,  $D_T =$  60

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

$H_O = D_T - D_O =$  20  
 $H_F = D_T - D_F =$  13.5  
 $\Delta H = \Delta D = H_O - H_F =$  6.5  
 $H_{avg} = (H_O + H_F) / 2 =$  16.75

$I_t =$  1.39 Inches per

**Client:** D. R. Horton  
**Project:** Whitney I 62/Valley Church  
**Project No:** 2868-CR  
**Date:** 9/3/2021

**Boring No.** I-3

**Infiltration Rate (Porchet Method)**

Time Interval,  $\Delta t =$  30  
Final Depth to Water,  $D_F =$  45  
Test Hole Radius,  $r =$  4  
Initial Depth to Water,  $D_O =$  40  
Total Test Hole Depth,  $D_T =$  60

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

$H_O = D_T - D_O =$  20  
 $H_F = D_T - D_F =$  15  
 $\Delta H = \Delta D = H_O - H_F =$  5  
 $H_{avg} = (H_O + H_F) / 2 =$  17.5

$I_t =$  1.03 Inches per

**Client:** D. R. Horton  
**Project:** Whitney I 62/Valley Church  
**Project No:** 2868-CR  
**Date:** 9/3/2021

**Boring No.** I-4

**Infiltration Rate (Porchet Method)**

Time Interval,  $\Delta t =$  10  
 Final Depth to Water,  $D_F =$  42  
 Test Hole Radius,  $r =$  4  
 Initial Depth to Water,  $D_O =$  40  
 Total Test Hole Depth,  $D_T =$  60

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

$H_O = D_T - D_O =$  20  
 $H_F = D_T - D_F =$  18  
 $\Delta H = \Delta D = H_O - H_F =$  2  
 $H_{avg} = (H_O + H_F) / 2 =$  19

$I_t =$  1.14 Inches pe



**Client:** D. R. Horton  
**Project:** Whitney I 62/Valley Church  
**Project No:** 2868-CR  
**Date:** 9/3/2021

**Boring No.** I-5

**Infiltration Rate (Porchet Method)**

Time Interval,  $\Delta t =$  10  
Final Depth to Water,  $D_F =$  42.5  
Test Hole Radius,  $r =$  4  
Initial Depth to Water,  $D_O =$  40  
Total Test Hole Depth,  $D_T =$  60

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

$H_O = D_T - D_O =$  20  
 $H_F = D_T - D_F =$  17.5  
 $\Delta H = \Delta D = H_O - H_F =$  2.5  
 $H_{avg} = (H_O + H_F) / 2 =$  18.75

$I_t =$  1.45 Inches per

**Client:** D. R. Horton  
**Project:** Whitney I 62/Valley Church  
**Project No:** 2868-CR  
**Date:** 9/3/2021

**Boring No.** I-6

**Infiltration Rate (Porchet Method)**

Time Interval,  $\Delta t =$  10  
 Final Depth to Water,  $D_F =$  42.5  
 Test Hole Radius,  $r =$  4  
 Initial Depth to Water,  $D_O =$  40  
 Total Test Hole Depth,  $D_T =$  60

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

$H_O = D_T - D_O =$  20  
 $H_F = D_T - D_F =$  17.5  
 $\Delta H = \Delta D = H_O - H_F =$  2.5  
 $H_{avg} = (H_O + H_F) / 2 =$  18.75

$I_t =$  1.45 Inches pe

**Client:** D. R. Horton  
**Project:** Whitney I 62/Valley Church  
**Project No:** 2868-CR  
**Date:** 9/3/2021

**Boring No.** I-7

**Infiltration Rate (Porchet Method)**

Time Interval,  $\Delta t =$  30  
 Final Depth to Water,  $D_F =$  44.5  
 Test Hole Radius,  $r =$  4  
 Initial Depth to Water,  $D_O =$  40  
 Total Test Hole Depth,  $D_T =$  60

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

$H_O = D_T - D_O =$  20  
 $H_F = D_T - D_F =$  15.5  
 $\Delta H = \Delta D = H_O - H_F =$  4.5  
 $H_{avg} = (H_O + H_F) / 2 =$  17.75

$I_t =$  0.91 Inches pe



**Client:** D. R. Horton  
**Project:** Whitney I 62/Valley Church  
**Project No:** 2868-CR  
**Date:** 9/3/2021

**Boring No.** I-8

**Infiltration Rate (Porchet Method)**

Time Interval,  $\Delta t =$  30  
Final Depth to Water,  $D_F =$  45  
Test Hole Radius,  $r =$  4  
Initial Depth to Water,  $D_O =$  40  
Total Test Hole Depth,  $D_T =$  60

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

$H_O = D_T - D_O =$  20  
 $H_F = D_T - D_F =$  15  
 $\Delta H = \Delta D = H_O - H_F =$  5  
 $H_{avg} = (H_O + H_F) / 2 =$  17.5

$I_t =$  1.03 Inches per

# Percolation Test Data Sheet

Project: DR Horton Whitney 162 E Valley Church <sup>Valley</sup> <sup>Murray</sup> <sup>Valley</sup>  
 Test Hole No.: I-1 Job No.: 2868 CR  
 Depth of Test Hole (ft): 5 ft Soil Description: SM  
 Date Excavated: 9/3/21  
 Percolation Test By: KRM/ω Date: 9/9/21 Presoak: Yes

## Percolation Test Data

Time	Time Interval (min)	Total Elapsed Time (min)	Water Level			Percolation Rate (min/inch)
			Initial (inch)	Final (inch)	Δ (inch)	
7:38						
8:03	25	25	20	12.5	7.5	
8:09						
8:29	25	51	20	14	6	
8:30						
8:40	10	62	20	17.5	2.5	
8:42						
8:52	10	74	20	17.5	2.5	
8:54						
9:04	10	86	20	17.5	2.5	
9:05						
9:15	10	97	20	17	3	
9:16						
9:26	10	108	20	17	3	
9:27						
9:37	10	119	20	17	3	4.3

**Design Percolation Rate:** \_\_\_\_\_ **min/inch**

# Percolation Test Data Sheet

Project: DR Horton Whitney 162 i Valley Church Moreno Job No.: 2868-UR  
 Test Hole No.: E-2 Date Excavated: 7/3/21  
 Depth of Test Hole (ft): 5 ft Soil Description: SM  
 Percolation Test By: KRM/CD Date: 9/9/21 Presoak: Yes

## Percolation Test Data

Time	Time Interval (min)	Total Elapsed Time (min)	Water Level			Percolation Rate (min/inch)
			Initial (inch)	Final (inch)	Δ (inch)	
7:32						
7:57	25	25	20	14.5	5.5	
7:59						
8:24	25	52	20	14.25	5.75	
8:25						
8:55	30	83	20	14	6	
8:56						
9:26	30	114	20	14.5	5.5	
9:27						
9:57	30	145	20	13.5	6.5	
10:00						
10:30	30	178	20	14	6	
10:32						
11:02	30	210	20	14	6	
11:03						
11:33	30	241	20	13.5	6.5	
11:34						
12:04	30	272	20	13.5	6.5	
12:05						
12:35	30	303	20	13.5	6.5	
12:36						
1:06	30	334	20	13.5	6.5	
1:07						
1:37	30	365	20	13.5	6.5	
1:38						
2:07	30	396	20	13.5	6.5	
2:09						
2:39	30	427	20	13.5	6.5	4.6

Design Percolation Rate:

min/inch

# Percolation Test Data Sheet

Project: DR. Horton Wilkey & Valley Church Job No.: 2868-CR  
 Test Hole No.: I-3 Date Excavated: 9/3/21  
 Depth of Test Hole (ft): 5 Soil Description: ML  
 Percolation Test By: KRM Date: 9/7/21 Presoak: Yes

## Percolation Test Data

Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (inches)	Final Water Level (inches)	Δ in Water Level (inches)	Percolation Rate (min/inch)
7:10						
7:35	25	25	20	14.5	5.5	
7:36						
8:01	25	51	20	14.5	5.5	
8:02						
8:32	30	82	20	14.5	5.5	
8:33						
9:03	30	113	20	15	5.0	
9:04						
9:34	30	144	20	14.75	5.25	
9:35						
10:05	30	175	20	15	5	
10:06						
10:36	30	206	20	15	5	
10:37						
11:07	30	237	20	15	5	
11:08						
11:38	30	268	20	15	5	
11:39						
12:09	30	299	20	15	5	
12:10						
12:40	30	330	20	15	5	
12:41						
13:11	30	361	20	15	5	
13:12						
13:42	30	392	20	15	5	
13:43						
2:13	30	423	20	15	5	6

Design Percolation Rate: \_\_\_\_\_ min/inch

# Percolation Test Data Sheet

Project: O.R. Horton Whitney & Valley Church Job No.: 2868-6A  
Test Hole No.: I-4 Date Excavated: 9/3/21  
Depth of Test Hole (ft): 5 Soil Description: ML  
Percolation Test By: KRM Date: 9/7/21 Presoak: Yes

Percolation Test Data

Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (inches)	Final Water Level (inches)	$\Delta$ in Water Level (inches)	Percolation Rate (min/inch)
7:15	15	25	20	13.75	6.25	
7:40						
7:41	25	51	20	14	6	
8:06						
8:07						
8:11	10	62	20	17.5	2.5	
8:12	10	73	20	17.5	2.5	
8:23						
8:25	10	84	20	17.5	2.5	
8:39						
8:40						
8:50	10	95	20	18	2.0	
8:51	10	106	20	18	2.0	
9:01						
9:02	10	117	20	18	2.0	5
9:12						

Design Percolation Rate: \_\_\_\_\_ min/inch



# Percolation Test Data Sheet

Project: D. B. Horton Whitney Valley Church Job No.: 2810 CR  
 Test Hole No.: I-5 Date Excavated: 9/3/21  
 Depth of Test Hole (ft): 5 Soil Description: ML  
 Percolation Test By: KRM Date: 9/8/21 Presoak: Yes

## Percolation Test Data

Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (inches)	Final Water Level (inches)	Δ in Water Level (inches)	Percolation Rate (min/inch)
7:10						
7:35	25	25	20	13	7	
7:36						
8:01	25	51	20	13	7	
8:02						
8:12	10	62	20	17.25	2.75	
8:13						
8:23	10	73	20	17.25	2.75	
8:24						
8:34	10	84	20	17.5	2.5	
8:36						
8:46	10	96	20	17.5	2.5	
8:47						
8:57	10	107	20	17.5	2.5	
8:58						
9:08	10	118	20	17.5	2.5	4

Design Percolation Rate: \_\_\_\_\_ min/inch

# Percolation Test Data Sheet

Project: Or. Hutton Withness & Valley Church Project      Job No.: 2812-02  
 Test Hole No.: I-6      Date Excavated: 9/3/21  
 Depth of Test Hole (ft): 5      Soil Description: ML  
 Percolation Test By: KRM      Date: 9/8/21      Presoak: Yes

**Percolation Test Data**

Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (inches)	Final Water Level (inches)	Δ in Water Level (inches)	Percolation Rate (min/inch)
7:12						
7:37	25	25	20	13	6.5	
7:38						
8:03	25	51	20	13.25	6.75	
8:04						
8:14	10	62	20	17.25	2.75	
8:15						
8:25	10	73	20	17.5	2.5	
8:26						
8:36	10	84	20	17.5	2.5	
8:37						
8:47	10	95	20	17.5	2.5	
8:48						
8:59	10	106	20	17.5	2.5	
8:59						
9:09	10	117	20	17.5	2.5	4

Design Percolation Rate: \_\_\_\_\_ min/inch

# Percolation Test Data Sheet

Project: O.R. Horton Wilkesville & Valley Church Project Job No.: 2868 CR  
 Test Hole No.: I-7 Date Excavated: 9/3/21  
 Depth of Test Hole (ft): 5 Soil Description: ml  
 Percolation Test By: KRM Date: 9/7/21 Presoak: Yes

## Percolation Test Data

Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (inches)	Final Water Level (inches)	Δ in Water Level (inches)	Percolation Rate (min/inch)
7:23	25	25	20	14.25	5.75	
7:48						
7:49	25	51	20	14.5	5.5	
8:14						
8:15	20	82	20	14.75	5.25	
8:45						
8:46	30	113	20	15	5	
9:16						
9:18	30	146	20	14.75	5.25	
9:48						
9:49	30	177	20	15	5	
10:19						
10:20	30	208	20	15.25	4.75	
10:50						
10:51	30	239	20	15.5	4.5	
11:21						
11:24	30	272	20	15.5	4.5	
11:54						
11:55	30	303	20	15.75	4.25	
12:25						
12:26	30	334	20	15.5	4.5	
12:56						
12:57	30	365	20	15.5	4.5	
1:27						
1:28	30	396	20	15.5	4.5	
1:58						
1:59	30	427	20	15.5	4.5	6.7
2:29						

Design Percolation Rate: min/inch

# Percolation Test Data Sheet

Project: D.R. HARTON WAYNE & Volney Church Job No.: 2868-CR  
 Test Hole No.: I-8 Date Excavated: 9/3/21  
 Depth of Test Hole (ft): 5 Soil Description: ML  
 Percolation Test By: KRM Date: 9/7/21 Presoak: Yes

### Percolation Test Data

Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (inches)	Final Water Level (inches)	Δ in Water Level (inches)	Percolation Rate (min/inch)
7:20						
7:45	25	25	20	15	5	
7:46						
8:11	25	51	20	14.75	5.25	
8:12						
8:42	30	82	20	14.75	5.25	
8:44						
9:14	30	114	20	15	5	
9:15						
9:45	30	145	20	15	5	
9:46						
10:16	30	176	20	14.75	5.25	
10:17						
10:47	30	207	20	15	5	
10:48						
11:18	30	238	20	15	5	
11:22						
11:52	30	272	20	15	5	
11:53						
12:23	30	303	20	15	5	
12:24						
12:54	30	334	20	15	5	
12:55						
1:25	30	365	20	15	5	
1:26						
1:56	30	396	20	15	5	
1:57						
2:27	30	427	20	15	5	6

Design Percolation Rate: \_\_\_\_\_ min/inch

**APPENDIX D**

**SEISMIC SETTLEMENT ANALYSIS**

**Single-Family Residential Tract Development  
Alessandro Boulevard and Oliver Street  
Moreno Valley, Riverside County, California  
Project No. 2868-CR**



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## LIQUEFACTION ANALYSIS CALCULATION DETAILS

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Input File Name: UNTITLED  
Title: Whitney 162/Valley Church 80  
Subtitle: Seismic Settlement

### Input Data:

Surface Elev.=1555  
Hole No.=B-12  
Depth of Hole=50.00 ft  
Water Table during Earthquake= 100.00 ft  
Water Table during In-Situ Testing= 2100.00 ft  
Max. Acceleration=0.88 g  
Earthquake Magnitude=7.30  
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 ft	SPT	Gamma pcf	Fines %
2.00	21.00	114.00	60.00
4.00	22.00	115.00	60.00
6.00	26.00	115.00	60.00
10.00	37.00	124.00	25.00
15.00	80.00	120.00	60.00
20.00	80.00	120.00	60.00
25.00	80.00	131.00	25.00

30.00	28.00	131.00	25.00
35.00	28.00	120.00	60.00
40.00	25.00	120.00	60.00
45.00	42.00	120.00	60.00
50.00	41.00	131.00	25.00

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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:

fs1	Depth =CSRfs ft	gamma pcf	sigma atm	gamma' pcf	sigma' atm	rd	mZ g	a(z) g	CSR	x
0.57	2.00	114.00	0.108	114.00	0.108	1.00	0.000	0.877	0.57	1.00
0.57	3.00	114.50	0.162	114.50	0.162	0.99	0.000	0.877	0.57	1.00
0.56	4.00	115.00	0.216	115.00	0.216	0.99	0.000	0.877	0.56	1.00
0.56	5.00	115.00	0.270	115.00	0.270	0.99	0.000	0.877	0.56	1.00
0.56	6.00	115.00	0.325	115.00	0.325	0.99	0.000	0.877	0.56	1.00
0.56	7.00	117.25	0.379	117.25	0.379	0.98	0.000	0.877	0.56	1.00
0.56	8.00	119.50	0.435	119.50	0.435	0.98	0.000	0.877	0.56	1.00
0.56	9.00	121.75	0.492	121.75	0.492	0.98	0.000	0.877	0.56	1.00
0.56	10.00	124.00	0.550	124.00	0.550	0.98	0.000	0.877	0.56	1.00
0.56	11.00	123.20	0.609	123.20	0.609	0.97	0.000	0.877	0.56	1.00
0.55	12.00	122.40	0.667	122.40	0.667	0.97	0.000	0.877	0.55	1.00
0.55	13.00	121.60	0.725	121.60	0.725	0.97	0.000	0.877	0.55	1.00
0.55	14.00	120.80	0.782	120.80	0.782	0.97	0.000	0.877	0.55	1.00
0.55	15.00	120.00	0.839	120.00	0.839	0.97	0.000	0.877	0.55	1.00
0.55	16.00	120.00	0.895	120.00	0.895	0.96	0.000	0.877	0.55	1.00

0.55	17.00	120.00	0.952	120.00	0.952	0.96	0.000	0.877	0.55	1.00
0.55	18.00	120.00	1.009	120.00	1.009	0.96	0.000	0.877	0.55	1.00
0.55	19.00	120.00	1.066	120.00	1.066	0.96	0.000	0.877	0.54	1.00
0.54	20.00	120.00	1.122	120.00	1.122	0.95	0.000	0.877	0.54	1.00
0.54	21.00	122.20	1.179	122.20	1.179	0.95	0.000	0.877	0.54	1.00
0.54	22.00	124.40	1.238	124.40	1.238	0.95	0.000	0.877	0.54	1.00
0.54	23.00	126.60	1.297	126.60	1.297	0.95	0.000	0.877	0.54	1.00
0.54	24.00	128.80	1.357	128.80	1.357	0.94	0.000	0.877	0.54	1.00
0.54	25.00	131.00	1.419	131.00	1.419	0.94	0.000	0.877	0.54	1.00
0.54	26.00	131.00	1.481	131.00	1.481	0.94	0.000	0.877	0.54	1.00
0.54	27.00	131.00	1.542	131.00	1.542	0.94	0.000	0.877	0.53	1.00
0.53	28.00	131.00	1.604	131.00	1.604	0.93	0.000	0.877	0.53	1.00
0.53	29.00	131.00	1.666	131.00	1.666	0.93	0.000	0.877	0.53	1.00
0.53	30.00	131.00	1.728	131.00	1.728	0.93	0.000	0.877	0.53	1.00
0.53	31.00	128.80	1.790	128.80	1.790	0.92	0.000	0.877	0.53	1.00
0.53	32.00	126.60	1.850	126.60	1.850	0.91	0.000	0.877	0.52	1.00
0.52	33.00	124.40	1.909	124.40	1.909	0.91	0.000	0.877	0.52	1.00
0.52	34.00	122.20	1.968	122.20	1.968	0.90	0.000	0.877	0.51	1.00
0.51	35.00	120.00	2.025	120.00	2.025	0.89	0.000	0.877	0.51	1.00
0.51	36.00	120.00	2.081	120.00	2.081	0.88	0.000	0.877	0.50	1.00
0.50	37.00	120.00	2.138	120.00	2.138	0.87	0.000	0.877	0.50	1.00
0.50	38.00	120.00	2.195	120.00	2.195	0.86	0.000	0.877	0.49	1.00
0.49	39.00	120.00	2.252	120.00	2.252	0.86	0.000	0.877	0.49	1.00
0.49	40.00	120.00	2.308	120.00	2.308	0.85	0.000	0.877	0.48	1.00
0.48	41.00	120.00	2.365	120.00	2.365	0.84	0.000	0.877	0.48	1.00



0.48										
0.47	42.00	120.00	2.422	120.00	2.422	0.83	0.000	0.877	0.47	1.00
0.47	43.00	120.00	2.478	120.00	2.478	0.82	0.000	0.877	0.47	1.00
0.47	44.00	120.00	2.535	120.00	2.535	0.82	0.000	0.877	0.47	1.00
0.46	45.00	120.00	2.592	120.00	2.592	0.81	0.000	0.877	0.46	1.00
0.46	46.00	122.20	2.649	122.20	2.649	0.80	0.000	0.877	0.46	1.00
0.45	47.00	124.40	2.707	124.40	2.707	0.79	0.000	0.877	0.45	1.00
0.45	48.00	126.60	2.767	126.60	2.767	0.78	0.000	0.877	0.45	1.00
0.44	49.00	128.80	2.827	128.80	2.827	0.78	0.000	0.877	0.44	1.00
0.44	50.00	131.00	2.888	131.00	2.888	0.77	0.000	0.877	0.44	1.00

CSR is based on water table at 100.00 during earthquake

CRR Calculation from SPT or BPT data:

(N1)60f	Depth CRR7.5 ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60
45.16	2.00 0.50	21.00	1.25	0.75	0.108	1.70	33.47	60.00	11.69
46.12	3.00 0.50	21.50	1.25	0.75	0.162	1.70	34.27	60.00	11.85
47.08	4.00 0.50	22.00	1.25	0.75	0.216	1.70	35.06	60.00	12.01
50.90	5.00 0.50	24.00	1.25	0.75	0.270	1.70	38.25	60.00	12.65
54.73	6.00 0.50	26.00	1.25	0.75	0.325	1.70	41.44	60.00	13.29
57.50	7.00 0.50	28.75	1.25	0.75	0.379	1.62	43.75	51.25	13.75
58.71	8.00 0.50	31.50	1.25	0.75	0.435	1.52	44.76	42.50	13.95
66.43	9.00 0.50	34.25	1.25	0.85	0.492	1.43	51.86	33.75	14.57
63.37	10.00 0.50	37.00	1.25	0.85	0.550	1.35	52.99	25.00	10.38
	11.00	45.60	1.25	0.85	0.609	1.28	62.09	32.00	15.45

77.54	0.50								
	12.00	54.20	1.25	0.85	0.667	1.22	70.52	39.00	19.10
89.62	0.50								
	13.00	62.80	1.25	0.85	0.725	1.17	78.39	46.00	20.68
99.07	0.50								
	14.00	71.40	1.25	0.85	0.782	1.13	85.80	53.00	22.16
107.96	0.50								
	15.00	80.00	1.25	0.95	0.839	1.09	103.73	60.00	25.75
129.48	0.50								
	16.00	80.00	1.25	0.95	0.895	1.06	100.40	60.00	25.08
125.47	0.50								
	17.00	80.00	1.25	0.95	0.952	1.02	97.36	60.00	24.47
121.83	0.50								
	18.00	80.00	1.25	0.95	1.009	1.00	94.58	60.00	23.92
118.50	0.50								
	19.00	80.00	1.25	0.95	1.066	0.97	92.03	60.00	23.41
115.44	0.50								
	20.00	80.00	1.25	0.95	1.122	0.94	89.68	60.00	22.94
112.61	0.50								
	21.00	80.00	1.25	0.95	1.179	0.92	87.48	53.00	22.50
109.97	0.50								
	22.00	80.00	1.25	0.95	1.238	0.90	85.39	46.00	22.08
107.47	0.50								
	23.00	80.00	1.25	0.95	1.297	0.88	83.42	39.00	21.68
105.10	0.50								
	24.00	80.00	1.25	0.95	1.357	0.86	81.54	32.00	18.77
100.32	0.50								
	25.00	80.00	1.25	0.95	1.419	0.84	79.76	25.00	13.46
93.22	0.50								
	26.00	69.60	1.25	0.95	1.481	0.82	67.93	25.00	12.10
80.03	0.50								
	27.00	59.20	1.25	0.95	1.542	0.81	56.61	25.00	10.80
67.40	0.50								
	28.00	48.80	1.25	1.00	1.604	0.79	48.16	25.00	9.83
57.99	0.50								
	29.00	38.40	1.25	1.00	1.666	0.77	37.19	25.00	8.57
45.75	0.50								
	30.00	28.00	1.25	1.00	1.728	0.76	26.63	25.00	7.35
33.98	0.50								
	31.00	28.00	1.25	1.00	1.790	0.75	26.16	32.00	9.30
35.47	0.50								
	32.00	28.00	1.25	1.00	1.850	0.74	25.73	39.00	10.15
35.88	0.50								
	33.00	28.00	1.25	1.00	1.909	0.72	25.33	46.00	10.07
35.40	0.50								
	34.00	28.00	1.25	1.00	1.968	0.71	24.95	53.00	9.99
34.94	0.50								
	35.00	28.00	1.25	1.00	2.025	0.70	24.60	60.00	9.92
34.52	0.50								
	36.00	27.40	1.25	1.00	2.081	0.69	23.74	60.00	9.75

33.49	0.50								
	37.00	26.80	1.25	1.00	2.138	0.68	22.91	60.00	9.58
32.49	0.50								
	38.00	26.20	1.25	1.00	2.195	0.67	22.11	60.00	9.42
31.53	0.50								
	39.00	25.60	1.25	1.00	2.252	0.67	21.33	60.00	9.27
30.59	0.50								
	40.00	25.00	1.25	1.00	2.308	0.66	20.57	60.00	9.11
29.68	0.41								
	41.00	28.40	1.25	1.00	2.365	0.65	23.08	60.00	9.62
32.70	0.50								
	42.00	31.80	1.25	1.00	2.422	0.64	25.54	60.00	10.11
35.65	0.50								
	43.00	35.20	1.25	1.00	2.478	0.64	27.95	60.00	10.59
38.54	0.50								
	44.00	38.60	1.25	1.00	2.535	0.63	30.30	60.00	11.06
41.36	0.50								
	45.00	42.00	1.25	1.00	2.592	0.62	32.61	60.00	11.52
44.13	0.50								
	46.00	41.80	1.25	1.00	2.649	0.61	32.10	53.00	11.42
43.52	0.50								
	47.00	41.60	1.25	1.00	2.707	0.61	31.60	46.00	11.32
42.92	0.50								
	48.00	41.40	1.25	1.00	2.767	0.60	31.11	39.00	11.22
42.34	0.50								
	49.00	41.20	1.25	1.00	2.827	0.59	30.63	32.00	10.07
40.70	0.50								
	50.00	41.00	1.25	1.00	2.888	0.59	30.16	25.00	7.76
37.91	0.50								

CRR is based on water table at 2100.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 7.30:

F.S.=CRRm/CSRfs	Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	
	2.00	0.07	0.50	1.00	0.50	1.07	0.54	0.57	5.00
	3.00	0.11	0.50	1.00	0.50	1.07	0.54	0.57	5.00
	4.00	0.14	0.50	1.00	0.50	1.07	0.54	0.56	5.00
	5.00	0.18	0.50	1.00	0.50	1.07	0.54	0.56	5.00
	6.00	0.21	0.50	1.00	0.50	1.07	0.54	0.56	5.00
	7.00	0.25	0.50	1.00	0.50	1.07	0.54	0.56	5.00
	8.00	0.28	0.50	1.00	0.50	1.07	0.54	0.56	5.00
	9.00	0.32	0.50	1.00	0.50	1.07	0.54	0.56	5.00
	10.00	0.36	0.50	1.00	0.50	1.07	0.54	0.56	5.00
	11.00	0.40	0.50	1.00	0.50	1.07	0.54	0.56	5.00
	12.00	0.43	0.50	1.00	0.50	1.07	0.54	0.55	5.00

13.00	0.47	0.50	1.00	0.50	1.07	0.54	0.55	5.00
14.00	0.51	0.50	1.00	0.50	1.07	0.54	0.55	5.00
15.00	0.55	0.50	1.00	0.50	1.07	0.54	0.55	5.00
16.00	0.58	0.50	1.00	0.50	1.07	0.54	0.55	5.00
17.00	0.62	0.50	1.00	0.50	1.07	0.54	0.55	5.00
18.00	0.66	0.50	1.00	0.50	1.07	0.54	0.55	5.00
19.00	0.69	0.50	1.00	0.50	1.07	0.54	0.54	5.00
20.00	0.73	0.50	1.00	0.50	1.07	0.54	0.54	5.00
21.00	0.77	0.50	1.00	0.50	1.07	0.54	0.54	5.00
22.00	0.80	0.50	1.00	0.50	1.07	0.54	0.54	5.00
23.00	0.84	0.50	1.00	0.50	1.07	0.54	0.54	5.00
24.00	0.88	0.50	1.00	0.50	1.07	0.54	0.54	5.00
25.00	0.92	0.50	1.00	0.50	1.07	0.54	0.54	5.00
26.00	0.96	0.50	1.00	0.50	1.07	0.54	0.54	5.00
27.00	1.00	0.50	1.01	0.50	1.07	0.54	0.53	5.00
28.00	1.04	0.50	1.00	0.50	1.07	0.54	0.53	5.00
29.00	1.08	0.50	0.99	0.50	1.07	0.53	0.53	5.00
30.00	1.12	0.50	0.99	0.49	1.07	0.53	0.53	5.00
31.00	1.16	0.50	0.98	0.49	1.07	0.53	0.53	5.00
32.00	1.20	0.50	0.97	0.49	1.07	0.52	0.52	5.00
33.00	1.24	0.50	0.97	0.48	1.07	0.52	0.52	5.00
34.00	1.28	0.50	0.96	0.48	1.07	0.52	0.51	5.00
35.00	1.32	0.50	0.96	0.48	1.07	0.51	0.51	5.00
36.00	1.35	0.50	0.95	0.48	1.07	0.51	0.50	5.00
37.00	1.39	0.50	0.95	0.47	1.07	0.51	0.50	5.00
38.00	1.43	0.50	0.94	0.47	1.07	0.50	0.49	5.00
39.00	1.46	0.50	0.94	0.47	1.07	0.50	0.49	5.00
40.00	1.50	0.41	0.93	0.39	1.07	0.41	0.48	5.00
41.00	1.54	0.50	0.93	0.46	1.07	0.50	0.48	5.00
42.00	1.57	0.50	0.92	0.46	1.07	0.49	0.47	5.00
43.00	1.61	0.50	0.92	0.46	1.07	0.49	0.47	5.00
44.00	1.65	0.50	0.91	0.46	1.07	0.49	0.47	5.00
45.00	1.68	0.50	0.91	0.45	1.07	0.49	0.46	5.00
46.00	1.72	0.50	0.90	0.45	1.07	0.48	0.46	5.00
47.00	1.76	0.50	0.90	0.45	1.07	0.48	0.45	5.00
48.00	1.80	0.50	0.90	0.45	1.07	0.48	0.45	5.00
49.00	1.84	0.50	0.89	0.45	1.07	0.48	0.44	5.00
50.00	1.88	0.50	0.89	0.44	1.07	0.47	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
2.00	-	-	-	45.16	60.00	0.00	45.16

3.00	-	-	-	46.12	60.00	0.00	46.12
4.00	-	-	-	47.08	60.00	0.00	47.08
5.00	-	-	-	50.90	60.00	0.00	50.90
6.00	-	-	-	54.73	60.00	0.00	54.73
7.00	-	-	-	57.50	51.25	0.00	57.50
8.00	-	-	-	58.71	42.50	0.00	58.71
9.00	-	-	-	66.43	33.75	0.00	66.43
10.00	-	-	-	63.37	25.00	0.00	63.37
11.00	-	-	-	77.54	32.00	0.00	77.54
12.00	-	-	-	89.62	39.00	0.00	89.62
13.00	-	-	-	99.07	46.00	0.00	99.07
14.00	-	-	-	100.00	53.00	0.00	100.00
15.00	-	-	-	100.00	60.00	0.00	100.00
16.00	-	-	-	100.00	60.00	0.00	100.00
17.00	-	-	-	100.00	60.00	0.00	100.00
18.00	-	-	-	100.00	60.00	0.00	100.00
19.00	-	-	-	100.00	60.00	0.00	100.00
20.00	-	-	-	100.00	60.00	0.00	100.00
21.00	-	-	-	100.00	53.00	0.00	100.00
22.00	-	-	-	100.00	46.00	0.00	100.00
23.00	-	-	-	100.00	39.00	0.00	100.00
24.00	-	-	-	100.00	32.00	0.00	100.00
25.00	-	-	-	93.22	25.00	0.00	93.22
26.00	-	-	-	80.03	25.00	0.00	80.03
27.00	-	-	-	67.40	25.00	0.00	67.40
28.00	-	-	-	57.99	25.00	0.00	57.99
29.00	-	-	-	45.75	25.00	0.00	45.75
30.00	-	-	-	33.98	25.00	0.00	33.98
31.00	-	-	-	35.47	32.00	0.00	35.47
32.00	-	-	-	35.88	39.00	0.00	35.88
33.00	-	-	-	35.40	46.00	0.00	35.40
34.00	-	-	-	34.94	53.00	0.00	34.94
35.00	-	-	-	34.52	60.00	0.00	34.52
36.00	-	-	-	33.49	60.00	0.00	33.49
37.00	-	-	-	32.49	60.00	0.00	32.49
38.00	-	-	-	31.53	60.00	0.00	31.53
39.00	-	-	-	30.59	60.00	0.00	30.59
40.00	-	-	-	29.68	60.00	0.00	29.68
41.00	-	-	-	32.70	60.00	0.00	32.70
42.00	-	-	-	35.65	60.00	0.00	35.65
43.00	-	-	-	38.54	60.00	0.00	38.54
44.00	-	-	-	41.36	60.00	0.00	41.36
45.00	-	-	-	44.13	60.00	0.00	44.13
46.00	-	-	-	43.52	53.00	0.00	43.52
47.00	-	-	-	42.92	46.00	0.00	42.92
48.00	-	-	-	42.34	39.00	0.00	42.34
49.00	-	-	-	40.70	32.00	0.00	40.70
50.00	-	-	-	37.91	25.00	0.00	37.91

(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

dsp	Depth	CSRs <sub>f</sub>	/ MSF*	=CSR <sub>m</sub>	F.S.	Fines	(N1)60s	Dr	ec	dsz
	S									
in.	ft					%		%	%	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:

ec	Depth	sigma'	sigC'	(N1)60s	CSRs <sub>f</sub>	Gmax	g*Ge/Gm	g_eff	ec7.5	Cec
	dsz	dsp	S							
%	ft	atm	atm			atm			%	
	in.	in.	in.							
	49.95	2.89	1.88	38.06	0.44	2057.48	6.1E-4	0.2197	0.0793	1.01
0.0800	9.60E-4	0.001	0.001							
	49.00	2.83	1.84	40.70	0.44	2082.63	6.0E-4	0.2062	0.0652	1.01
0.0657	7.89E-4	0.016	0.017							
	48.00	2.77	1.80	42.34	0.45	2087.54	5.9E-4	0.1986	0.0628	1.01
0.0633	7.60E-4	0.015	0.033							
	47.00	2.71	1.76	42.92	0.45	2074.57	5.9E-4	0.1958	0.0619	1.01
0.0624	7.49E-4	0.015	0.048							
	46.00	2.65	1.72	43.52	0.46	2061.63	5.9E-4	0.1930	0.0610	1.01
0.0615	7.39E-4	0.015	0.062							
	45.00	2.59	1.68	44.13	0.46	2048.69	5.8E-4	0.1901	0.0601	1.01
0.0606	7.28E-4	0.015	0.077							
	44.00	2.54	1.65	41.36	0.47	1982.92	5.9E-4	0.2012	0.0636	1.01
0.0642	7.70E-4	0.015	0.092							
	43.00	2.48	1.61	38.54	0.47	1914.96	6.1E-4	0.2141	0.0749	1.01
0.0755	9.06E-4	0.016	0.108							
	42.00	2.42	1.57	35.65	0.47	1844.47	6.2E-4	0.2295	0.0963	1.01
0.0971	1.16E-3	0.021	0.129							
	41.00	2.37	1.54	32.70	0.48	1771.05	6.4E-4	0.2480	0.1227	1.01

0.1238	1.49E-3	0.027	0.156								
	40.00	2.31	1.50	29.68	0.48	1694.19	6.6E-4	0.2711	0.1568	1.01	
0.1581	1.90E-3	0.034	0.190								
	39.00	2.25	1.46	30.59	0.49	1690.13	6.5E-4	0.6766	0.3736	1.01	
0.3768	4.52E-3	0.096	0.285								
	38.00	2.19	1.43	31.53	0.49	1685.55	6.4E-4	0.6305	0.3318	1.01	
0.3346	4.02E-3	0.085	0.370								
	37.00	2.14	1.39	32.49	0.50	1680.42	6.3E-4	0.5874	0.2938	1.01	
0.2963	3.56E-3	0.075	0.446								
	36.00	2.08	1.35	33.49	0.50	1674.73	6.2E-4	0.5469	0.2594	1.01	
0.2616	3.14E-3	0.067	0.512								
	35.00	2.02	1.32	34.52	0.51	1668.49	6.2E-4	0.5091	0.2280	1.01	
0.2299	2.76E-3	0.059	0.571								
	34.00	1.97	1.28	34.94	0.51	1651.47	6.1E-4	0.4869	0.2128	1.01	
0.2147	2.58E-3	0.053	0.624								
	33.00	1.91	1.24	35.40	0.52	1633.82	6.0E-4	0.4639	0.1975	1.01	
0.1992	2.39E-3	0.050	0.674								
	32.00	1.85	1.20	35.88	0.52	1615.52	6.0E-4	0.4402	0.1822	1.01	
0.1837	2.20E-3	0.046	0.720								
	31.00	1.79	1.16	35.47	0.53	1582.81	5.9E-4	0.4326	0.1835	1.01	
0.1850	2.22E-3	0.043	0.763								
	30.00	1.73	1.12	33.98	0.53	1533.36	6.0E-4	0.4442	0.2051	1.01	
0.2068	2.48E-3	0.047	0.810								
	29.00	1.67	1.08	45.75	0.53	1662.48	5.3E-4	0.2757	0.0872	1.01	
0.0879	1.06E-3	0.030	0.840								
	28.00	1.60	1.04	57.99	0.53	1765.27	4.8E-4	0.1970	0.0623	1.01	
0.0628	7.54E-4	0.018	0.857								
	27.00	1.54	1.00	67.40	0.53	1819.81	4.5E-4	0.1594	0.0504	1.01	
0.0508	6.10E-4	0.014	0.871								
	26.00	1.48	0.96	80.03	0.54	1887.81	4.2E-4	0.1283	0.0406	1.01	
0.0409	4.91E-4	0.011	0.882								
	25.00	1.42	0.92	93.22	0.54	1944.26	3.9E-4	0.1066	0.0337	1.01	
0.0340	4.08E-4	0.009	0.891								
	24.00	1.36	0.88	100.00	0.54	1946.72	3.8E-4	0.0956	0.0302	1.01	
0.0305	3.66E-4	0.008	0.898								
	23.00	1.30	0.84	100.00	0.54	1902.97	3.7E-4	0.0910	0.0288	1.01	
0.0290	3.48E-4	0.007	0.906								
	22.00	1.24	0.80	100.00	0.54	1858.97	3.6E-4	0.0866	0.0274	1.01	
0.0276	3.31E-4	0.007	0.912								
	21.00	1.18	0.77	100.00	0.54	1814.71	3.5E-4	0.0823	0.0260	1.01	
0.0262	3.15E-4	0.006	0.919								
	20.00	1.12	0.73	100.00	0.54	1770.16	3.4E-4	0.1445	0.0457	1.01	
0.0461	5.53E-4	0.009	0.928								
	19.00	1.07	0.69	100.00	0.54	1724.85	3.4E-4	0.1318	0.0417	1.01	
0.0420	5.04E-4	0.011	0.939								
	18.00	1.01	0.66	100.00	0.55	1678.33	3.3E-4	0.1199	0.0379	1.01	
0.0383	4.59E-4	0.010	0.948								
	17.00	0.95	0.62	100.00	0.55	1630.48	3.2E-4	0.1089	0.0344	1.01	
0.0347	4.17E-4	0.009	0.957								
	16.00	0.90	0.58	100.00	0.55	1581.18	3.1E-4	0.0987	0.0312	1.01	

0.0315	3.78E-4	0.008	0.965							
	15.00	0.84	0.55	100.00	0.55	1530.29	3.0E-4	0.0892	0.0282	1.01
0.0285	3.41E-4	0.007	0.972							
	14.00	0.78	0.51	100.00	0.55	1477.47	2.9E-4	0.0804	0.0254	1.01
0.0256	3.08E-4	0.006	0.979							
	13.00	0.72	0.47	99.07	0.55	1417.89	2.8E-4	0.0729	0.0230	1.01
0.0232	2.79E-4	0.006	0.984							
	12.00	0.67	0.43	89.62	0.55	1315.66	2.8E-4	0.0717	0.0227	1.01
0.0229	2.74E-4	0.006	0.990							
	11.00	0.61	0.40	77.54	0.56	1197.93	2.8E-4	0.0727	0.0230	1.01
0.0232	2.78E-4	0.005	0.995							
	10.00	0.55	0.36	63.37	0.56	1064.98	2.9E-4	0.0770	0.0244	1.01
0.0246	2.95E-4	0.006	1.001							
	9.00	0.49	0.32	66.43	0.56	1023.21	2.7E-4	0.1183	0.0374	1.01
0.0377	4.53E-4	0.010	1.011							
	8.00	0.44	0.28	58.71	0.56	923.38	2.6E-4	0.1039	0.0328	1.01
0.0331	3.97E-4	0.008	1.019							
	7.00	0.38	0.25	57.50	0.56	856.14	2.5E-4	0.0729	0.0230	1.01
0.0232	2.79E-4	0.007	1.026							
	6.00	0.32	0.21	54.73	0.56	778.90	2.3E-4	0.0564	0.0178	1.01
0.0180	2.16E-4	0.005	1.031							
	5.00	0.27	0.18	50.90	0.56	693.78	2.2E-4	0.0462	0.0146	1.01
0.0147	1.77E-4	0.004	1.034							
	4.00	0.22	0.14	47.08	0.56	604.20	2.0E-4	0.0865	0.0273	1.01
0.0276	3.31E-4	0.005	1.039							
	3.00	0.16	0.11	46.12	0.57	519.31	1.8E-4	0.0406	0.0128	1.01
0.0129	1.55E-4	0.004	1.043							
	2.00	0.11	0.07	45.16	0.57	420.92	1.5E-4	0.0278	0.0088	1.01
0.0089	1.07E-4	0.003	1.046							

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Settlement of Unsaturated Sands=1.046 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=1.046 in.  
 Differential Settlement=0.523 to 0.690 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/ft<sup>2</sup> = 2 kip/ft<sup>2</sup>)  
 1 atm (atmosphere) = 101.325 kPa (1 kPa = 1 kN/m<sup>2</sup> = 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, $CRRv = CRR_{7.5} * K_{sig}$
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	$CSRfs = CSR * fs_1$ (Default $fs_1 = 1$ )
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	$(N1)_{60f} = (N1)_{60} + d(N1)_{60}$
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 CORROSION REPORT**

**Single-Family Residential Tract Development  
Alessandro Boulevard and Oliver Street  
Moreno Valley, Riverside County, California  
Project No. 2868-CR**





September 23, 2021

via email: kmchargue@geotekusa.com

Geotek, Inc.  
1548 N. Maple St.  
Corona, CA, 92878

Attention: Mr. Kyle McHargue

Re: Soil Corrosivity Study  
Whitney & Valley Church Project  
Moreno Valley, CA  
HDR #21-0793SCS, Geotek #2868-CR

## Introduction

Laboratory tests have been completed on 15 soil samples provided for the referenced 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 samples are representative of the most corrosive soils at the site.

The proposed construction consists of over 260 single family residence with one to two stories. The site is located at Alessandro Blvd. and Oliver St. 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 21-0845SCS. 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.<sup>1</sup>

**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 mildly to moderately corrosive categories. The resistivities dropped considerably with added moisture because the samples were dry as-received.

Soil pH values varied from 5.7 to 7.2. This range is moderately acidic to neutral.<sup>2</sup> These values do not particularly increase soil corrosivity.

The soluble salt content of the samples ranged from low to moderate.

The ammonium and nitrate concentration were high enough to be aggressive to copper.

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

The Caltrans Corrosion Guidelines were used as a basis for mechanically stabilized embankment (MSE) material backfill evaluation.<sup>3</sup> Section 8.1, Corrosion Requirements for MSE Structure Backfill, references Caltrans Standard Specification Section 47-2.02 for properties of structure backfill. The minimum requirements for soil in contact with MSE are shown in Table 2.

<sup>1</sup> Romanoff, Melvin. *Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166-167.*

<sup>2</sup> Romanoff, Melvin. *Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*

<sup>3</sup> *Corrosion Guidelines, Version 3.0. California Department of Transportation (Caltrans), Sacramento, CA, 2018.*

**Table 2: MSE Backfill Minimum Soil Requirements.**

Property	Requirement	ASTM Test
Resistivity	2000 ohm-cm minimum	G187
Chloride	Less than 250 ppm	D4327 and D6919
Sulfates	Less than 500 ppm	D4327 and D6919
pH	5.5 to 10.0	G51

Caltrans considers MSE backfill material that meets the above criteria to be “non-corrosive” to the metallic soil reinforcement. Comparison of the attached Table 1 soil sample analysis results with the above criteria indicates that the material satisfies all the above requirements and is therefore suitable for use as MSE backfill.

Variation in soil resistivity of an order of magnitude or more can create differential-aeration corrosion cells that would affect all metals.

In conclusion, this soil is classified as moderately corrosive to ferrous metals, aggressive to copper, and suitable for use as MSE backfill.

## 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 3/4-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.

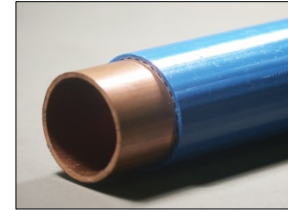
## Copper Tubing

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.<sup>4</sup>
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

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<sup>4</sup> 2016 California Plumbing Code (CPC), July 1, 2018 Supplement, Section 604.3.

- 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.

## Post-Tensioned Slabs: Unbonded Single-Stranded Tendons and Anchors

Although chloride levels were relatively low, soil is considered an aggressive environment for post-tensioning strands and anchors. Protect post-tensioning strands and anchors against corrosion by implementing all the following measures:<sup>5,6,7</sup>

1. Limit the water-soluble chloride ion content in the concrete mix design to less than 0.06 percent by weight of cement.
2. Design all tendons to prevent ingress of moisture. A corrosion-inhibiting coating should be incorporated into the tendon sheaths.
3. Use non-shrink grout mixes for all post-tensioning pockets.
4. Prior to grouting the pocket, apply a protective grease cap filled with corrosion protection material that provides a watertight seal for the strand end and wedge cavity. Ensure the cap fully seats against the face of the standard anchor at the live end.
5. Protect all components from moisture prior to installation and within one working day after installation.
6. Ensure the minimum concrete cover over the tendon tail is 1 inch, or greater if required by the applicable building code.
7. Install caps within one working day after the cutting of the tendon tails and acceptance of the elongation records by the engineer.

<sup>5</sup> *Post-Tensioning Manual, sixth edition. Post-Tensioning Institute (PTI), Phoenix, AZ, 2006.*

<sup>6</sup> *PTI M10.2-00: Specification for Unbonded Single Strand Tendons. Post-Tensioning Institute (PTI), Phoenix, AZ, 2000.*

<sup>7</sup> *ACI 423.6-01: Specification for Unbonded Single Strand Tendons. American Concrete Institute (ACI), 2001*

8. Install pre-cast concrete plug over the grease cap to ensure the live end is sealed from further moisture intrusion.
9. Limit the access of direct runoff onto the anchorage area by designing proper drainage. Do not allow water to pond against anchors.
10. Provide at least 2 inches of space between finish grade and the anchorage area, or more if required by applicable building codes.

## 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.



Kristy McCann, EIT  
*Corrosion EIT*



Marc E N Wegner, PE  
*Sr. Corrosion Project Manage*

Enc: Table 1 – Laboratory Tests on Soil Samples

21-0845SCS SCS KM\_BS\_MW.docx



**Table 1 - Laboratory Tests on Soil Samples**

**Geotek, Inc.**  
**Whitney & Valley Church Project**  
**Your #2868-CR, HDR Lab #21-0845SCS**  
**20-Sep-21**

Sample ID			Cor-1	Cor-2	Cor-3	Cor-4	Cor-5
			@ 0-0.5'	@ 0-0.5'	@ 0-0.5'	@ 0-0.5'	@ 0-0.5'
<b>Resistivity</b>	<b>Units</b>						
as-received	ohm-cm		>4,400,000	>4,400,000	>4,400,000	>4,400,000	>4,400,000
saturated	ohm-cm		3,800	12,400	22,400	3,960	10,000
<b>pH</b>			6.9	6.8	7.2	7.1	7.1
<b>Electrical</b>							
<b>Conductivity</b>	mS/cm		0.17	0.20	0.07	0.16	0.13
<b>Chemical Analyses</b>							
<b>Cations</b>							
calcium	Ca <sup>2+</sup>	mg/kg	72	85	43	56	68
magnesium	Mg <sup>2+</sup>	mg/kg	15	23	8.5	15	12
sodium	Na <sup>1+</sup>	mg/kg	13	10	4.5	4.3	3.3
potassium	K <sup>1+</sup>	mg/kg	106	127	42	186	134
ammonium	NH <sub>4</sub> <sup>1+</sup>	mg/kg	52	47	ND	ND	ND
<b>Anions</b>							
carbonate	CO <sub>3</sub> <sup>2-</sup>	mg/kg	ND	ND	ND	ND	ND
bicarbonate	HCO <sub>3</sub> <sup>1-</sup>	mg/kg	223	189	162	232	299
fluoride	F <sup>1-</sup>	mg/kg	40	19	13	13	18
chloride	Cl <sup>1-</sup>	mg/kg	28	18	14	33	27
sulfate	SO <sub>4</sub> <sup>2-</sup>	mg/kg	26	25	13	19	14
nitrate	NO <sub>3</sub> <sup>1-</sup>	mg/kg	130	523	65	198	84
phosphate	PO <sub>4</sub> <sup>3-</sup>	mg/kg	1.9	3.3	1.4	2.5	54
<b>Other Tests</b>							
sulfide	S <sup>2-</sup>	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



**Table 1 - Laboratory Tests on Soil Samples**

**Geotek, Inc.**  
**Whitney & Valley Church Project**  
**Your #2868-CR, HDR Lab #21-0845SCS**  
**20-Sep-21**

Sample ID			Cor-6	Cor-7	Cor-8	Cor-9	Cor-10
			@ 0-0.5'	@ 0-0.5'	@ 0-0.5'	@ 0-0.5'	@ 0-0.5'
<b>Resistivity</b>							
	<b>Units</b>						
as-received	ohm-cm		>4,400,000	>4,400,000	1,800,000	>4,400,000	3,000,000
saturated	ohm-cm		2,080	2,360	6,800	2,960	11,200
<b>pH</b>			6.2	6.5	5.7	6.0	6.1
<b>Electrical</b>							
<b>Conductivity</b>	mS/cm		0.24	0.21	0.14	0.19	0.12
<b>Chemical Analyses</b>							
<b>Cations</b>							
calcium	Ca <sup>2+</sup>	mg/kg	87	86	46	55	45
magnesium	Mg <sup>2+</sup>	mg/kg	21	22	14	19	13
sodium	Na <sup>1+</sup>	mg/kg	14	34	16	25	8.2
potassium	K <sup>1+</sup>	mg/kg	165	109	139	182	119
ammonium	NH <sub>4</sub> <sup>1+</sup>	mg/kg	54	ND	0.8	ND	ND
<b>Anions</b>							
carbonate	CO <sub>3</sub> <sup>2-</sup>	mg/kg	ND	ND	ND	ND	ND
bicarbonate	HCO <sub>3</sub> <sup>1-</sup>	mg/kg	210	210	180	235	180
fluoride	F <sup>1-</sup>	mg/kg	16	17	17	19	34
chloride	Cl <sup>1-</sup>	mg/kg	26	48	24	38	23
sulfate	SO <sub>4</sub> <sup>2-</sup>	mg/kg	28	22	22	29	19
nitrate	NO <sub>3</sub> <sup>1-</sup>	mg/kg	683	533	260	357	88
phosphate	PO <sub>4</sub> <sup>3-</sup>	mg/kg	8.4	8.3	1.3	2.1	3.8
<b>Other Tests</b>							
sulfide	S <sup>2-</sup>	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



**Table 1 - Laboratory Tests on Soil Samples**

**Geotek, Inc.**  
**Whitney & Valley Church Project**  
**Your #2868-CR, HDR Lab #21-0845SCS**  
**20-Sep-21**

Sample ID			Cor-11	Cor-12	Cor-13	Cor-14	Cor-15
			@ 0-0.5'	@ 0-0.5'	@ 0-0.5'	@ 0-0.5'	@ 0-0.5'
<b>Resistivity</b>	<b>Units</b>						
as-received	ohm-cm		>4,400,000	>4,400,000	>4,400,000	>4,400,000	>4,400,000
saturated	ohm-cm		5,600	4,800	7,200	11,200	7,600
<b>pH</b>			6.1	6.2	6.7	6.7	6.4
<b>Electrical</b>							
<b>Conductivity</b>	mS/cm		0.10	0.12	0.11	0.10	0.18
<b>Chemical Analyses</b>							
<b>Cations</b>							
calcium	Ca <sup>2+</sup>	mg/kg	44	44	47	41	59
magnesium	Mg <sup>2+</sup>	mg/kg	11	14	13	11	18
sodium	Na <sup>1+</sup>	mg/kg	5.7	8.3	3.2	2.5	2.9
potassium	K <sup>1+</sup>	mg/kg	90	123	129	121	214
ammonium	NH <sub>4</sub> <sup>1+</sup>	mg/kg	ND	45	ND	ND	46
<b>Anions</b>							
carbonate	CO <sub>3</sub> <sup>2-</sup>	mg/kg	ND	ND	ND	ND	ND
bicarbonate	HCO <sub>3</sub> <sup>1-</sup>	mg/kg	207	226	256	229	332
fluoride	F <sup>1-</sup>	mg/kg	18	19	14	14	19
chloride	Cl <sup>1-</sup>	mg/kg	18	28	21	14	40
sulfate	SO <sub>4</sub> <sup>2-</sup>	mg/kg	16	17	12	14	20
nitrate	NO <sub>3</sub> <sup>1-</sup>	mg/kg	78	98	87	82	144
phosphate	PO <sub>4</sub> <sup>3-</sup>	mg/kg	1.5	3.7	8.7	2.4	12
<b>Other Tests</b>							
sulfide	S <sup>2-</sup>	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  
Alessandro Boulevard and Oliver Street  
Moreno Valley, Riverside County, California  
Project No. 2868-CR**

## **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 California Building Code, CBC (2019) and the guidelines presented below.

### **Preconstruction Meeting**

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

### **Grading Observation and Testing**

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



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

### **Site Clearing**

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

### **Treatment of Existing Ground**

1. Following site clearing, all surficial deposits of topsoil, alluvium and colluvium as well as weathered or creep effected bedrock, should be removed 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.

### **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. 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.

## **UTILITY TRENCH CONSTRUCTION AND BACKFILL**

Utility trench excavation and backfill is the contractors 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 contractors 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 contractors 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.

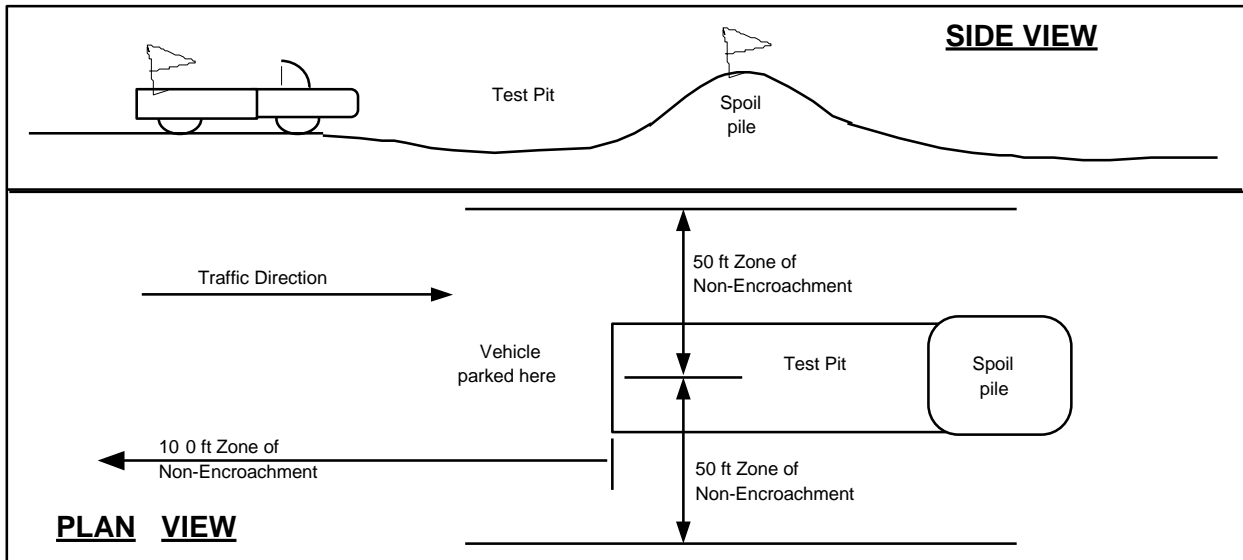
### **Test Pits Location, Orientation and Clearance**

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 effect 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 effect 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 technicians attention and notify our project manager or office. Effective communication and coordination between the contractors' representative and the field technician(s) is strongly encouraged in order to implement the above safety program and safety in general.

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

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

# ALTERNATES

Finish Grade

Original Ground

Loose Surface Materials

Suitable Material

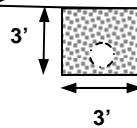
4 feet typical

Construct Benches where slope exceeds 5:1

Slope to Drain

Suitable Material

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



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

Finish Grade

Original Ground

Loose Surface Materials

Construct Benches where slope exceeds 5:1

Slope to Drain

Suitable Material

4 feet typical

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

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



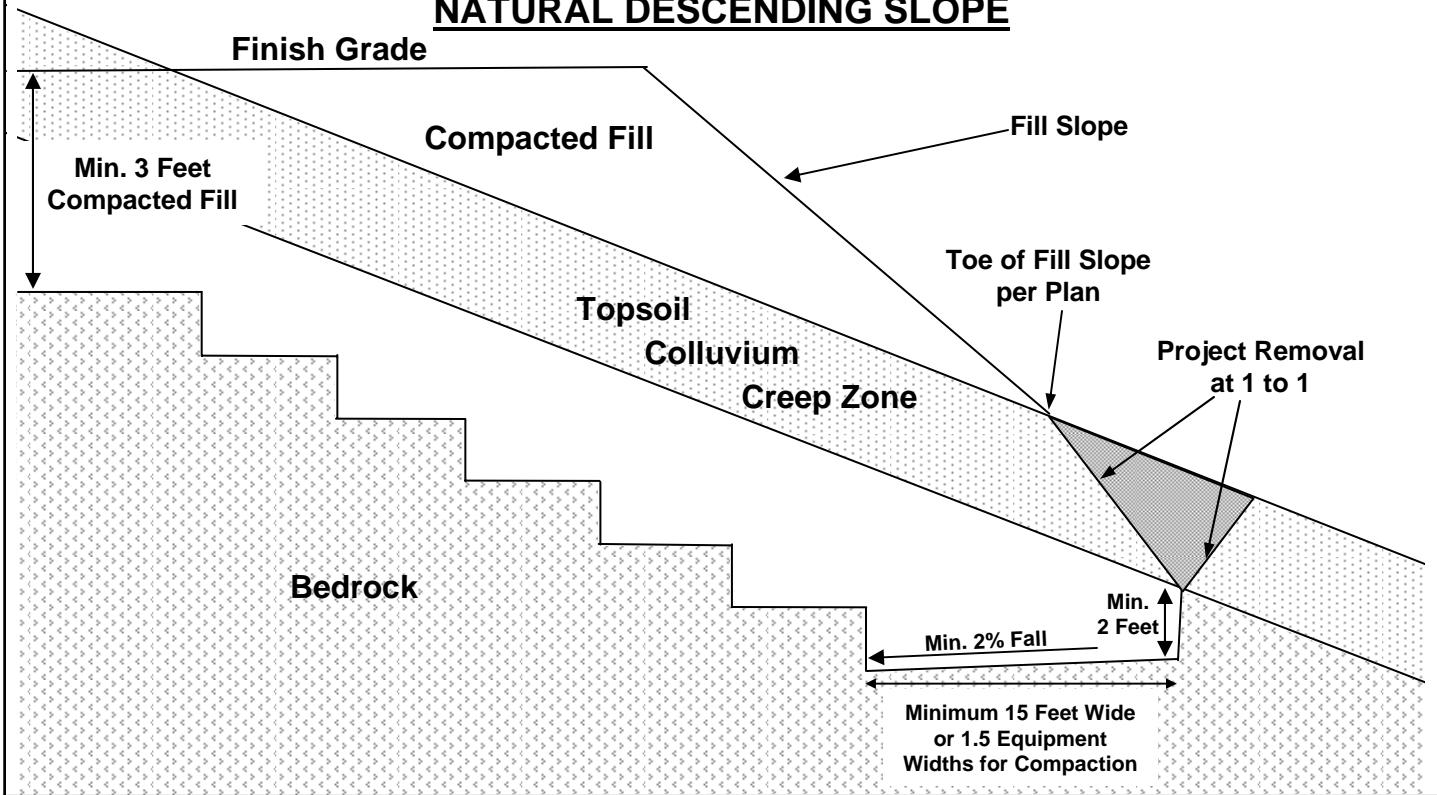
1548 North Maple Street  
Corona, California 92880

TYPICAL CANYON  
CLEANOUT

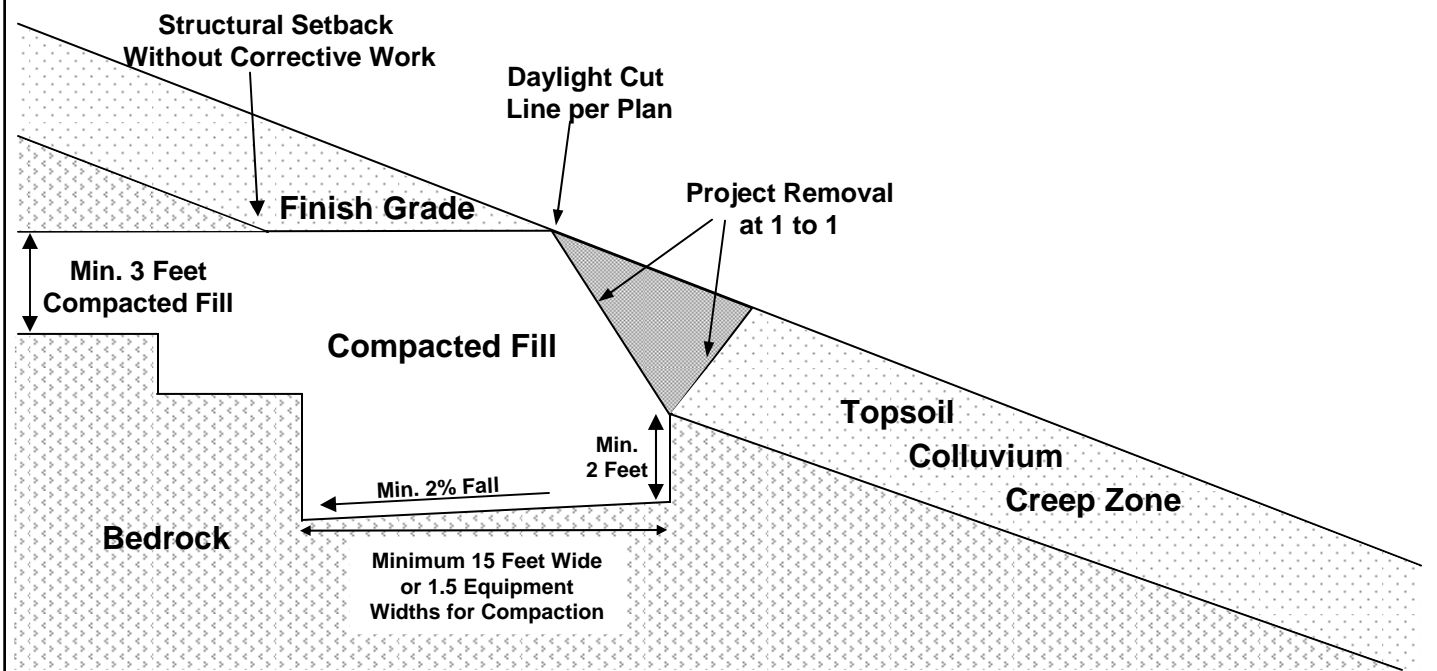
STANDARD GRADING  
GUIDELINES

PLATE E-1

## TYPICAL FILL SLOPE OVER NATURAL DESCENDING SLOPE



## DAYLIGHT CUT AREA OVER NATURAL DESCENDING SLOPE



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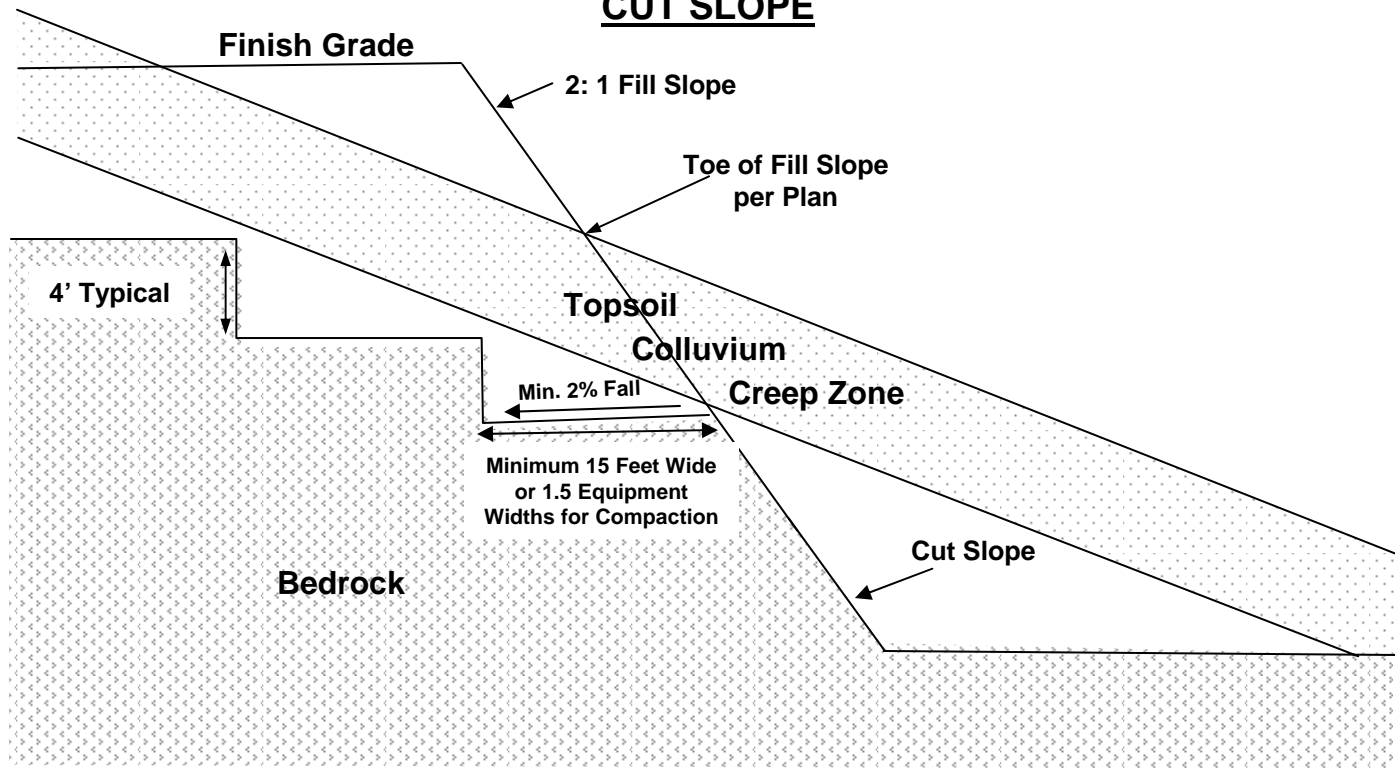
TREATMENT ABOVE  
NATURAL SLOPES

STANDARD GRADING  
GUIDELINES

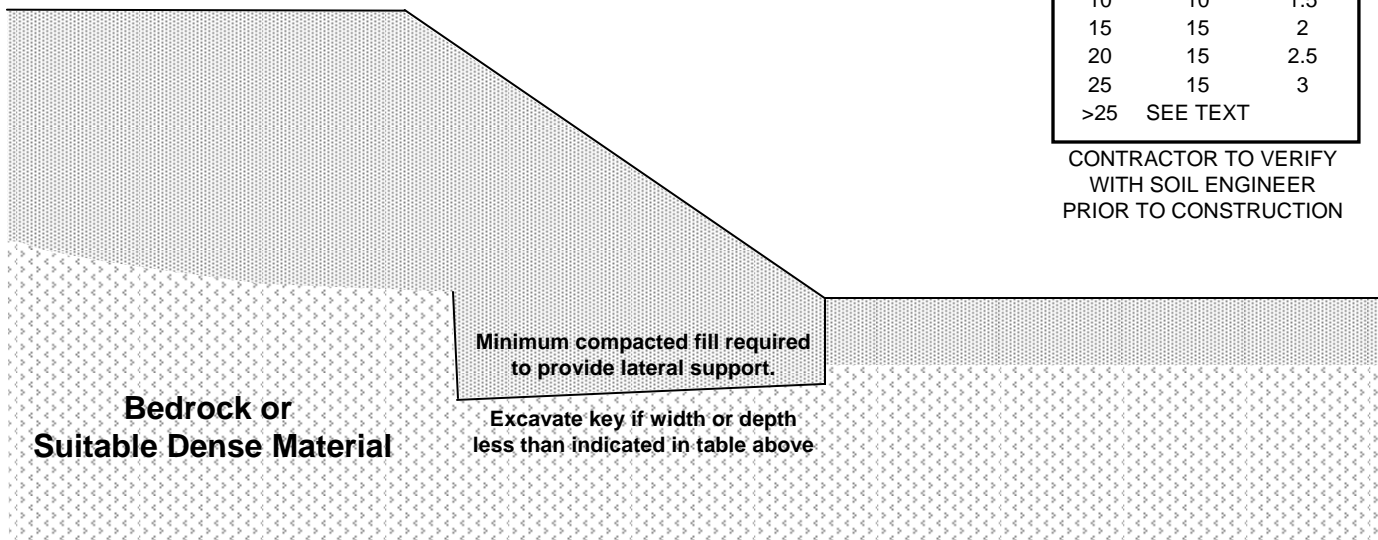
PLATE E-2



## 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



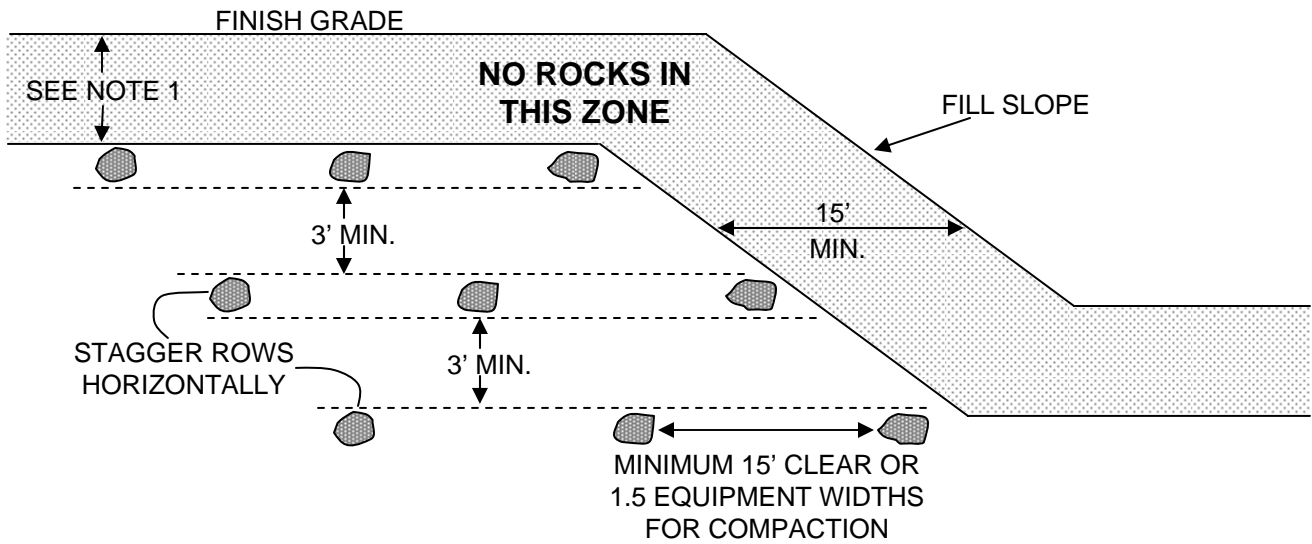
1548 North Maple Street  
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COMMON FILL  
SLOPE KEYS

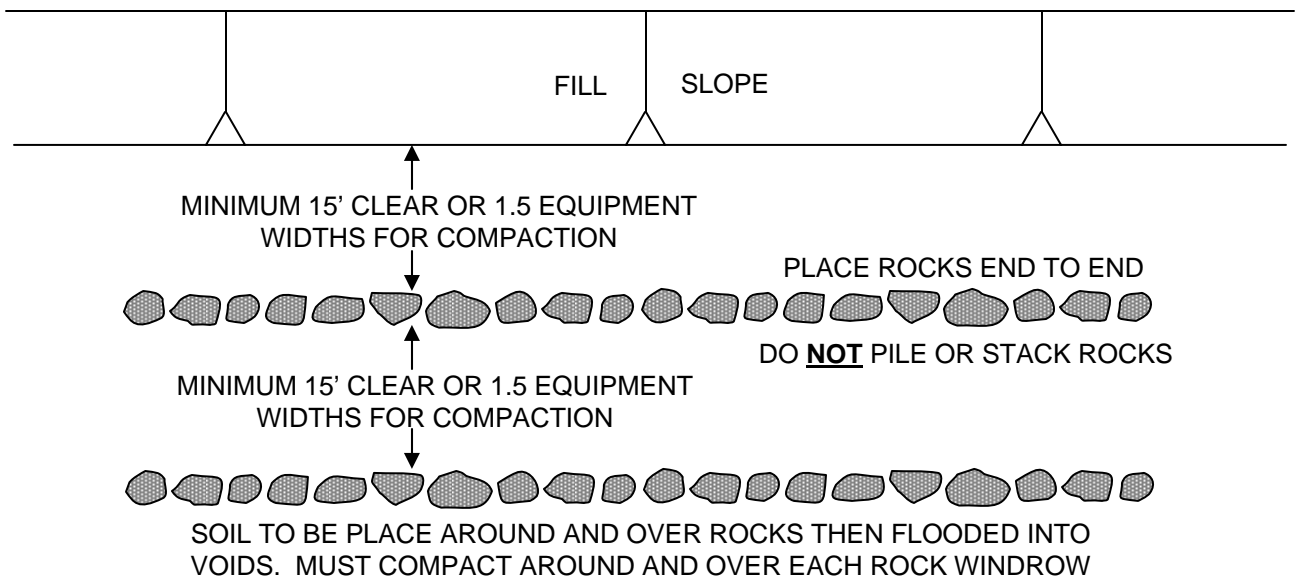
STANDARD GRADING  
GUIDELINES

PLATE E-3

# CROSS SECTIONAL VIEW



# PLAN VIEW



**NOTES:**

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

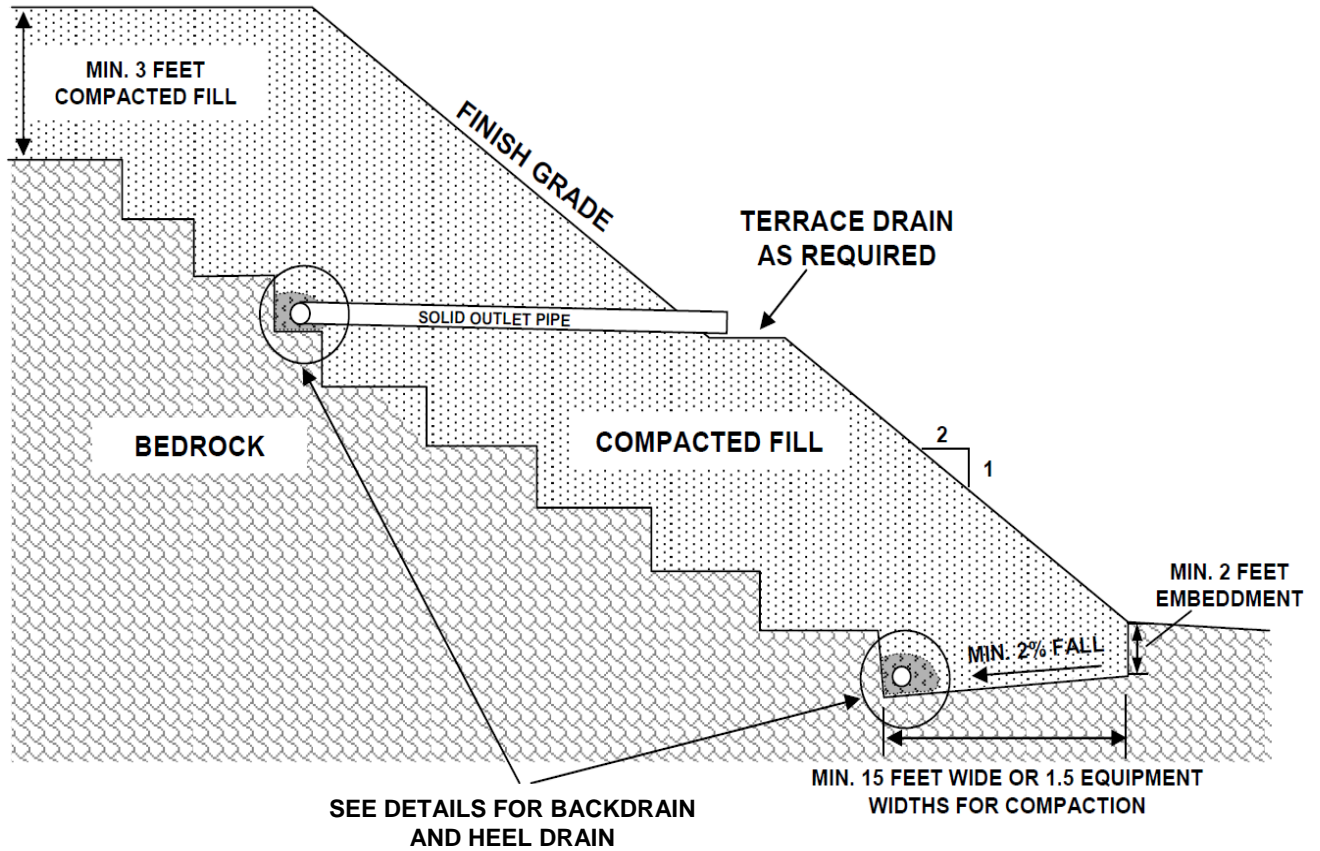


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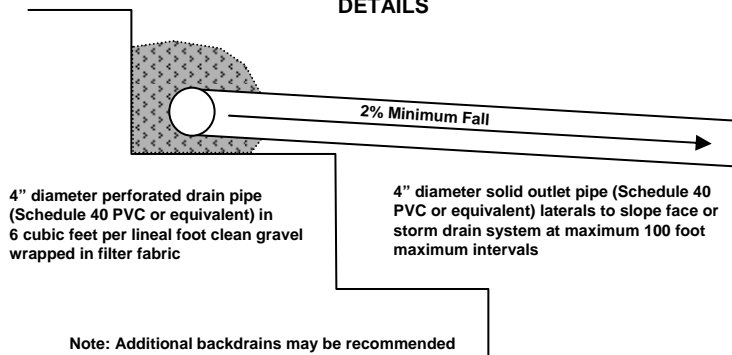
ROCK BURIAL DETAILS

STANDARD GRADING  
GUIDELINES

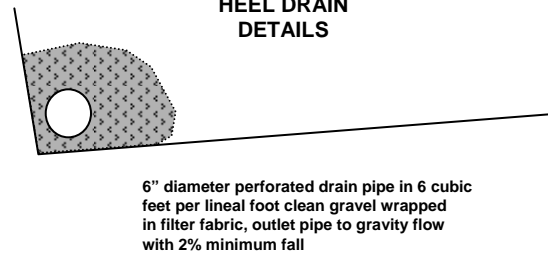
PLATE E-4



**BACKDRAIN DETAILS**



**HEEL DRAIN DETAILS**



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TYPICAL BUTTRESS AND  
STABILIZATION FILL

STANDARD GRADING  
GUIDELINES

PLATE E-5