UPDATED GEOTECHNICAL EVALUATION
PROPOSED SINGLE-FAMILY RESIDENTIAL DEVELOPMENT
TRES CERRITOS WEST PROJECT – TRACT NO. 31513
HEMET, RIVERSIDE COUNTY, CALIFORNIA

PREPARED FOR

D•R•HORTON LOS ANGELES HOLDING COMPANY, INC.
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March 21, 2024
Project No. 2593-CR

D•R•Horton Los Angeles Holding Company, Inc.
2280 Wardlow Circle, Suite 100
Corona, California 92880

Attention: Mr. Parker Chorich

Subject: **Updated Geotechnical Evaluation**
Proposed Single-Family Residential Development
Tres Cerritos West Project -Tract No. 31513
Hemet, Riverside County, California

Dear Mr. Chorich:

GeoTek, Inc. (GeoTek) is pleased to provide the results of this Updated Geotechnical Evaluation for Tract No. 31513, a proposed single-family residential development to be located in the City of Hemet, Riverside County, California. This report presents the results of GeoTek's evaluation and discussion of findings.

Based on the results of this evaluation, the proposed site development appears feasible from a geotechnical viewpoint provided that the recommendations included herein are incorporated into the design and construction phases of site development.

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

Respectfully submitted,
GeoTek, Inc.

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1. PURPOSE AND SCOPE OF SERVICES

The purpose of this study was to evaluate the geotechnical conditions of the project site as related to the currently proposed development. Services provided for this study included the following:

- Research and review of available geologic and geotechnical data, and general information pertinent to the site,
- Review of the Updated Geotechnical and Infiltration Evaluation report previously prepared for the site by GeoTek (GeoTek, 2021),
- Review of the Soil Corrosivity Evaluation report previously prepared for the site by Project X Corrosion Engineering, Inc. (Project X, 2021),
- Perform a site reconnaissance, including evaluation of current geologic and geotechnical conditions at the site, including assessment of current rockfall hazards present,
- Review and evaluation of site seismicity, and
- Compilation of this Updated Geotechnical Evaluation which presents GeoTek's findings and a general summary of pertinent geotechnical conditions relevant for site development.

The intent of this report is to aid in the evaluation of the site for future development from a geotechnical perspective. The professional opinions and geotechnical information contained in this report will likely need to be updated based on review of final site development plans. These should be provided to GeoTek for review when available.

2. SITE DESCRIPTION AND PROPOSED DEVELOPMENT

2.1 SITE DESCRIPTION

The approximate 180-acre irregular-shaped project site is generally located north and adjacent to Rose Road and east of the intersection of Old Warren Road, in the City of Hemet, Riverside County, California (See Figure 1). Based on review of the previous site reporting by GeoSoils, Inc., (2021) and GeoTek (2021), as well as a recent site reconnaissance, the site has previously been partially rough graded.

Based upon the referenced report by GeoSoils, Inc., (GeoSoils, 2021), site grading previously occurred in or about late 2006. Previous site development included the planned construction of 176 single-family residences on 176 lots. Grading for the previously planned development was not completed. Except for an access road to the reservoirs located north of the site, the site streets have not been paved and it appears that the street utilities have not yet been installed. The majority of the previously proposed building pads also do not appear to have been brought up to finish grade. The central portion of the site appears to have been a borrow site and is estimated to be about 10 to 15 feet below adjacent grades, with some stockpiled boulder-sized materials present. An existing Eastern Municipal Water District (EMWD) 36-inch water main utility line is understood to exist near the alignment of the existing paved access road to the reservoir.

The site is bordered by vacant land to the north, east and west with a residential development to the southwest and water storage tanks further to the north, and by an agricultural field to the south. Site access is generally available from Rose Road, an unpaved roadway located adjacent to the southern project site boundary.

A site reconnaissance was performed on March 13, 2024 by a GeoTek engineer for the preparation of this Updated Report, and to document current geologic and geotechnical engineering conditions. The site conditions encountered were generally consistent with the findings as reported by GeoSoils (2021) and GeoTek (2021). A concrete brow ditch was observed adjacent to the eastern site boundary, but rockfall hazard mitigation walls have not yet been constructed. Rough grading was seen to have been previously completed on some lots, initiated but not completed on others, or not performed on the remaining proposed lots. Stormwater disposal basins were seen to be constructed near the southeastern and southwestern portions of the project site.

2.2 PROPOSED DEVELOPMENT

Based upon review of current *Conceptual Development Plan for Tract No. 31513*, prepared by SP2 & Co., Inc., dated December 16, 2023, GeoTek understands the subject property is to be developed with 279 single-family residential lots, several parks, open space conservation, several water quality basins, along with streets, utilities, hardscaping, and associated tract infrastructure improvements. Stormwater disposal is to be by means of multiple stormwater detention basins. Cuts and fills up to approximately ±20 feet in height are anticipated, not including remedial grading. Overlay of the proposed lot layout on the existing topography indicates that modifications of existing building pads and street alignments are proposed.

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. GeoTek understands that 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 40 kips and 3.0 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

Additional subsurface explorations were not performed by GeoTek for this Updated report.

The field exploration for the referenced site geotechnical report by GeoTek (2021) was conducted on January 12, 2021 and consisted of excavating five (5) geotechnical exploratory borings (B-2 through B-6; B-1 was eliminated) with a truck mounted drill rig to depths ranging from about 9 to 51-½ feet below ground surface. Several of the test borings were terminated at depths shallower than initially planned due to refusal on granitic bedrock. Additionally, four (4) test pits were also excavated with a Hyundai HX480 (108,000 pound) track mounted excavator to assess the hardness of the near-surface earth materials. The test pits were generally about 6 to 8 feet in width, 16 to 22 feet in length and about 7-½ to 20 feet in depth. The approximate locations of the GeoTek excavations are shown on the Site Exploration Map (Figure 2). Logs of the GeoTek borings and test pits are included in Appendix A.

Near-surface soil samples were also collected for laboratory testing and those sample locations are also presented on Figure 2.

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, 2.4-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. Standard Penetration Tests (SPT) were performed within Boring B-4 per ASTM D-1586m to assess the relative density of the encountered soils. 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 ring and SPT sampler data is presented on the boring logs in Appendix A.

Percolation Testing

In addition to the geotechnical exploratory borings, GeoTek performed six (6) percolation test borings (I-1 through I-6) in the three (3) areas of the proposed storm water management basins to depths of about 5 feet each. Infiltration/percolation testing was conducted in these borings in general accordance with the requirements of the County of Riverside.

The percolation tests consisted of drilling an eight-inch diameter test hole to the desired depth and installing approximately two inches of gravel in the bottom of the hole. 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 boring. Water was then placed in the borings to presoak the holes and percolation testing was performed the following the pre-soak period. Following presoaking, the percolation tests were then performed which consisted of adding water to each test hole and measuring the water drop over a 10-minute period. The water drop was recorded for at least six (6) 10-minute or twelve (12) 30-minute test intervals, depending on the results of the "sandy soil" initial trial measurements. 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.

PRELIMINARY INFILTRATION RATES		
Boring No.	Depth (ft.)	Infiltration Rate* (inch/hour)
I-1	5.0	2.40
I-2	5.0	1.88
I-3	5.0	0.00
I-4	5.0	0.00
I-5	5.0	1.91
I-6	5.0	0.09

*Converted infiltration rate using the Porchet Method

The results of the conversions indicate infiltration rate range from about 0.00 to 2.40 inches 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

No additional laboratory testing was conducted by GeoTek for this Updated report.

Laboratory testing was performed on selected relatively undisturbed ring and bulk samples collected during the previous field exploration performed by GeoTek (2021). 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.
The County of Riverside (https://countyofriverside.us/Residents/PropertyInformation.aspx) has designated the site area as "not in fault zone" and "not in a fault line". The site is designated as being within a "low" to "moderate" liquefaction potential area. The site is not situated within a State of California liquefaction hazard area and is not within a designated *Alquist-Priolo* Earthquake Fault hazard zone.

More specific to the subject property, the site is located in an area geologically mapped to be underlain by older alluvium and granitic bedrock (Dibble, 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 previously performed (GeoTek, 2021) and review of published geologic maps, the area of anticipated improvements are locally underlain by undocumented and engineered fill, older alluvium and granitic bedrock.

4.2.1 Existing Fill

Existing fill was encountered within four (4) of the test borings and three (3) of the test pits excavated at the site for the previous geotechnical investigation (GeoTek, 2021). At the exploration locations performed by GeoTek (2021), the depths of fill (where encountered) extended to depths ranging from about 4.5 to 12 feet below existing grade. The existing fill generally consisted of silty sands (SM soil type based upon the Unified Soil Classification System) which were observed to be in a relatively dense to very dense condition. The documented fill, reported by GeoSoils (2021), as encountered during the previous investigation (GeoTek, 2021) appears to have been properly compacted in accordance with industry standard practices.

Undocumented fills are present across the project site, mostly in the central and northern portions and are most likely to be present in areas where either grading had been initiated but not completed, where grading operations had not been performed, or where documentation of prior grading is not available.

According to the results of the laboratory testing performed on three (3) samples of the near surface soils, the near surface soils have a "Very Low" (0≤EI≤20) to "Medium" (51≤EI≤90) Expansion Index (EI) when tested and classified in accordance with ASTM D 4829. Additionally, 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 laboratory test results are provided in Appendix B.

4.2.2 Older Alluvium

Older alluvium was encountered within one (1) test boring, and one (1) test pit excavated for the previous site geotechnical investigation (GeoTek, 2021). The older alluvium encountered generally consisted of dense to very dense silty sands and clayey sands (SM and SC soil types) and extended to the maximum depth explored of 51-½ feet.

4.2.3 Granitic Bedrock

Granitic bedrock was locally encountered beneath the existing fill and/or at the ground surface, predominantly toward the western and eastern edges of the site, and extended to the maximum depths explored for the previous site geotechnical investigation performed (GeoTek, 2021). As encountered, the weathered bedrock that was sampled generally excavated as a silty sand (SM soil type) material. Relatively unweathered crystalline bedrock was also observed and encountered, as noted in the Logs of Exploratory Excavations in Appendix A.

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 borings performed by GeoTek for the referenced report (GeoTek, 2021) which were extended to a maximum depth of about 51-½ feet below existing grade. A review of groundwater depths noted on the State Department of Water Resources Water Data Library website indicates a depth to groundwater within nearby wells to be in excess of 100 feet below existing grade at the site. However, previous reporting by GeoSoils (2006a) noted that previous work at the site by EnGEN (report not provided) in 2003 indicated that perched water was observed at depths as shallow as 37 feet below grade.

It is possible that seasonal variations (temperature, rainfall, etc.) will cause fluctuations in the groundwater level. Additionally, perched water may be encountered at shallow depths following extensive rain events. If shallow perched water is encountered, it is anticipated that it can be managed with conventional sump pumps.

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 southern California area is dominated mainly by northwest-trending faults associated with the San Andreas system. The site is in a seismically active region. No active or potentially active fault is known to exist at this site nor is the site situated within an *"Alquist-Priolo"* Earthquake Fault Zone (CGS, 1998). The subject property is not located within a State of California Seismic Hazard Zone for earthquake induced landslide potential. The nearest zoned fault is the San Jacinto Fault zone, located approximately 2-¾ miles to the northeast.

4.4.2 Seismic Design Parameters

The site is located at approximately 33.7554 degrees Latitude and -117.0287 degrees Longitude. Due to the presence of relatively shallow bedrock, a Site Class "C" is considered appropriate for the site. Site spectral accelerations (S_s and S_1), for 0.2 and 1.0 second periods for a Class "C" site, were determined from the USGS Website, Earthquake Hazards Program, U.S. Seismic Design Maps for Risk-Targeted Maximum Considered Earthquake (MCE_R) Ground Motion Response Accelerations for the Conterminous 48 States by Latitude/Longitude. Due to the presence of very dense older alluvium and granitic bedrock, a Site Class C is deemed appropriate for the site. The results, based upon the 2015 NEHRP and CBC 2022 are presented in the following table:

SITE SEISMIC PARAMETERS	
Mapped 0.2 sec Period Spectral Acceleration, S_S	1.696g
Mapped 1.0 sec Period Spectral Acceleration, S_1	0.662g
Site Coefficient for Site Class "C", F_a	1.2
Site Coefficient for Site Class "C", F_v	1.4
Maximum Considered Earthquake Spectral Response Acceleration for 0.2 Second, S_{MS}	2.036g
Maximum Considered Earthquake Spectral Response Acceleration for 1.0 Second, S_{M1}	0.927g
5% Damped Design Spectral Response Acceleration Parameter at 0.2 Second, S_{DS}	1.357g
5% Damped Design Spectral Response Acceleration Parameter at 1 second, S_{D1}	0.618g
Peak Ground Acceleration (PGA_M)	0.862g
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 County of Riverside has indicated the site to be located within a "low" to "moderate" liquefaction potential area. A liquefaction assessment was previously conducted for this site (GeoTek, 2021). A historic high groundwater depth of 37 feet was used in the GeoTek (2021) analysis. The soil profile identified within B-5 was used for the assessment. A mean magnitude weighted (Mw) seismic event of 7.49 was incorporated into the analysis. A PGA_M value of 0.87g was obtained from the USGS website and incorporated the ASCE 7-16 provisions.

GeoTek evaluated the liquefaction potential of the on-site soils using the computer program LiquefyPro Version 5.

The results of the analyses indicated that the soils within Boring B-5 are not susceptible to significant soil liquefaction during the design-level earthquake. The seismic-settlement assessment performed for the referenced report (GeoTek, 2021) estimated total seismic settlements on the order of about 1/3 inch is possible. Additionally, GeoTek estimated the differential seismic-induced settlement to be less than ¼ inch over a 30-foot span. The results of the liquefaction analysis are presented within Appendix D.

4.6 OTHER SEISMIC HAZARDS

Due to the general flat terrain and low site liquefaction potential, 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. Evidence of ancient landslides or slope instabilities at the site were not observed during the previous investigation nor during a recent site reconnaissance.

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.

Based upon the laboratory test results reported by GeoTek (2021), the older alluvium generally exhibits a “very low” potential for hydrocollapse (settlement upon wetting with or without additional loading) and exhibit a “Very Low” (0≤EI≤20) to “Medium” (51≤EI≤90) Expansion Index (EI) when tested in accordance with ASTM D 4829.

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 Hemet and the 2022 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. The horizontal limits of the clearing should extend at least 10 feet beyond the new building and beneath any new improvements.

5.2.3 Site Preparation

As shown on the attached Figure 3, the site generally exists in three (3) phases of grading – areas where rough grading has previously been completed, areas where rough grading had not been performed, and areas where rough grading had been initiated but not completed. In addition, due to changes in project design since the date of the previous report (GeoTek, 2021), modification of existing graded pads and street alignments will be required.

The documented fill encountered on the existing vacant graded pads was previously evaluated through exploratory hollow-stem auger borings and exploratory test pits performed for the referenced report (GeoTek, 2021). Based upon the field investigation and laboratory data previously performed by GeoTek (2021), the documented fill appears to have been compacted in general accordance with industry standard practices, and appears to be suitable for its intended function, from a geotechnical standpoint. However, due to the age of the fill and weathering effects, the upper approximate 18 inches of surficial documented fill soils will require removal, scarification of at least 12 inches, followed by moisture conditioning, and recompaction to provide a suitable bearing surface for the proposed structures. This recommendation pertains to currently proposed Lots 23 through 71, 77 through 108, and 138-139. The soils exposed at the base of the recommended reprocessing depth should be locally observed and tested to confirm that a minimum relative compaction of 90% (ASTM D1557) percent has been attained. Also, any soil that ruts or excessively deflects during proof rolling should be removed as recommended by the GeoTek representative.

In areas where remedial grading was not previously performed (currently proposed Lots 14 through 22, 72-76, 135-137, and 255-273), the existing soils should be over-excavated to remove all loose or compressible alluvium and any highly weathered granitic bedrock. The over-excavation should extend laterally at least 10 feet beyond the limits of the planned buildings, where obtainable. An over-excavation depth ranging from about 2 to 10 feet was estimated by GeoSoils and this estimate is considered to be reasonable. However, as a minimum, the over-excavation should be extended in depth, where needed, so that all foundations and floor slabs will be underlain by at least 2 feet of engineered fill. The actual removal depths should be based on in-field inspections by a GeoTek representative during site grading operations.

For areas where remedial grading was initiated but not completed (Lots 1 through 13, 109-134, 140-254, and 274-279) removals of all loose or compressible alluvium should be performed prior to engineered fill placement. Necessary removal depths for these lots will vary based on the degree of grading completion prior to work stoppage in or around the year 2006, and the change in lot configuration. Over-excavation should be extended to a depth so that all foundations and floor slabs be underlain by at least two (2) feet of engineered fill, and extend a minimum of 10 feet beyond the limits of the planned buildings, where obtainable. Removal depths on the order of about 1 to 6 feet are considered reasonable estimates for these lots, however, actual removal depths shall be determined by a GeoTek representative during site grading. These grading procedures are also considered applicable to existing graded pads that will be modified.

In areas where granitic bedrock is exposed at finished grade or within the depth of the deepest utility, GeoTek recommends that the bedrock be over-excavated to a depth of at least 3 feet below finish grade or beneath the deepest utility excavation and be backfilled with a properly compacted engineered fill. Any building pads underlain by granitic bedrock that require over-excavation, the over-excavation bottom should be tilted to drain toward the street to help prevent any future ponding of water at the fill/bedrock interface.

Once the soils exposed at the bottom of the over-excavations are approved by GeoTek, the exposed soils should be scarified to a depth of about 12 inches, be moisture treated to slightly above the soil's maximum dry density, per ASTM D1557, and then be compacted to at least 90% of the soil's maximum dry density (ASTM D1557).

The test pits excavated at the site by GeoTek for the referenced report (GeoTek, 2021) were loosely backfilled upon completion. GeoTek recommends that the loose backfill be removed and be replaced as a properly compacted fill. The depth of the backfill ranges from about 7-½ to 20-½ feet. The approximate locations of the test pits are presented on Figure 2.

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.

The test pits excavated by GeoTek should be identified (see Figure 2) prior to site grading, along with the removal and replacement as properly compacted engineered fill should occur for the test pits backfill soil. The depth of the backfill ranges from about 7-½ to 20-½ feet.

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. Rough grading will create cut, cut/fill transition and shallow fill lots. The cut portion of the cut/fill transition should be undercut to a depth of at least three (3) feet below existing grades or two (2) foot below the deepest proposed footing, whichever is deeper, and be backfilled with a properly compacted engineered fill and overexcavation bottoms should slope a minimum of 1 percent to drain to the adjacent street of suitable direction so ponding of water is not likely. The lateral extent of this recommendation should include the area extending at least five (5) feet beyond the building limits.

Transition (i.e., cut/fill) lots should be overexcavated a minimum of at least three (3) feet below proposed grades or to a depth of 1/3 the maximum fill thickness.

All cut lots exposing granitic bedrock should be overexcavated to a depth of three (3) feet below proposed grade and replaced with engineered fill. All removals bottoms should be sloped a minimum of 1 percent to drain towards the adjacent street.

5.2.6 Oversized Rock Disposal

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

5.2.7 Slope Construction

Grading details for slope construction are presented as details E-2 and E-3 in Appendix E. Cut slopes in bedrock and competent alluvial soils constructed at maximum gradients of 2:1 (horizontal: vertical), in accordance with industry standards, are anticipated to be grossly stable. Surficial stability should be assessed during grading by an engineering geologist. An engineering geologist should observe all cut slopes. Cut slopes should expose competent granitic bedrock or competent alluvial soils. If adverse structure or incompetent materials are exposed and identified in the cut slopes, stabilization fills may be recommended. Where alluvial soils are present over bedrock in the cut slope, the alluvial portion of the slope should be reconstructed as a surficial stability fill.

Swales should be constructed at the top of all slopes to collect and divert drainage away from the slope face. Drainage should be directed to an approved drainage discharge location. Swales should be constructed with concrete, shotcrete or approved non-erosive material. Swales should be cleaned of loose soil and debris on an on-going basis.

Fill slopes constructed at maximum gradients of 2:1 (horizontal: vertical), in accordance with industry standards, are anticipated to be both grossly and surficially stable. Where fill is to be placed against sloping terrain with gradients of 5:1 (horizontal: vertical) or steeper, the sloping ground surface should be benched to remove loose and disturbed surface soil to assure that the new fill is placed in direct contact with competent bedrock and to provide horizontal surfaces for fill placement. A 10- to 15-foot-wide keyway should be constructed at the toe of the fill slope areas extending at least 2 to 3 feet vertically into competent natural material.

The base of the keyways and benches should be sloped back into the hillside at a gradient of at least two percent. The base of the benches should be evaluated by a representative of GeoTek prior to processing. Upon approval, the exposed materials should be moistened to at least the optimum moisture content and densified to a relative compaction of at least 90 percent of maximum dry density as determined by ASTM D 1557 test procedures.

Fill slopes should also be overbuilt and cut-back to their design grade or the slopes should be back-rolled with a sheepsfoot roller or similar compaction equipment at vertical heights not exceeding four (4) feet. The finished graded slope surface should be compacted to at least 90 percent of the soil's maximum dry density per ASTM D1557. A suitable alternative would be to compact the slopes during construction and then roll the final slope to provide a dense, erosion resistant surface.

Berms should be constructed and maintained at the top of all fill slopes to divert drainage away from the slope faces. An abatement program to control ground-burrowing rodents should be implemented and maintained. Burrowing rodents can decrease the long-term performance of slopes.

5.2.8 Excavation Characteristics

Excavations in the on-site alluvium and highly weathered bedrock 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.

Five test pits were excavated at the site utilizing a 108,000-pound Hyundai HX480 track-mounted excavator to assess the rock harness and excavation characteristics of the encountered earth materials. The locations of the test pits is presented on Figure 2 and logs of the test pits are presented in Appendix A. The table below summarizes the excavation characteristics at the test pit locations.

Test Pit No.	Max. Depth (ft)	Comments
TP-1	20.5	No Refusal-full depth
TP-2	20.5	No Refusal-full depth
TP-3	7.5	Refusal on Bedrock
TP-4	16.0	Refusal on Bedrock
TP-5	10.0	Refusal on Bedrock

Based on the test pit observations and dependent upon the depths of the planned excavations, excavation difficulties and localized blasting may be required for site development and installation of deep utilities. In areas where shallow bedrock is not present, excavations are expected to be feasible utilizing conventional heavy-duty earth moving equipment in good working condition. All excavations should be formed in accordance with current Cal-OSHA requirements.

5.2.9 Trench Excavations and Backfill

Temporary trench excavations within the on-site materials should be stable at a 1.5:1 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 may be locally difficult at the project site due to the presence of relatively shallow granitic bedrock. Special excavation techniques may be required to reach design grades for excavations within hard bedrock. Proper equipment should be employed by the contractor to accomplish trench excavations. All 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.

5.2.10 Shrinkage and Bulking

For planning purposes, a shrinkage factor of less than about 5 percent may be considered for excavations with the existing fill and/or older alluvium. A bulking factor of about 5 to 15% is estimated for excavations extending into the granitic bedrock where excavatable/rippable and a bulking factor of about 15 to 25% is estimated for bedrock that requires blasting. A negligible subsidence loss is also anticipated within older alluvium and bedrock areas.

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

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.

5.2.11 Rockfall Mitigation

Based upon review of the referenced report (GeoSoils, 2006) prepared for the site, GeoSoils recommended that debris/impact walls, gabion baskets, catchment/deflection devises and/or rockfall netting be constructed behind formerly proposed Lots 66-73, 132-145, 147-166 and 170-177. These lot numbers are based upon the *Rough Grading Plans* prepared by Myers and

Associates, Inc., dated January 20, 2020. Based on the recent reconnaissance of the site, these recommended walls have not yet been constructed. Based upon review of the *PAR Exhibit for Tract No. 31513 Conceptual Development Plan*, prepared by SP2 & Co., Inc., dated February 16, 2024, GeoTek recommends rockfall mitigation debris walls or similar devices be installed at Lots 1 through 26, 32 through 42, 72 through 74, and 255 through 279 (See Figure 3).

Various methods are available for rockfall hazard mitigation, including: entrapment and removal; rock bolts; wire mesh slope control, rock barrier fencing, and removal of rockfall hazard by removal of loose rocks or boulders by scaling, mechanical removal, and/or by blasting. Any rockfall barrier fencing (GeoBrugg GBE or equivalent) should be at least 6- to 8-feet in height depending on location, and designed for an equivalent fluid pressure of at least 125 pounds per cubic foot, or impact loads as determined by a formalized analysis. GeoTek suggests a formal rockfall hazard mitigation study be performed to identify and mitigate potential rockfall hazards.

5.2.12 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 2022 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≤EI≤20) and “Medium” (51≤EI≤90) Expansion Index (EI) when tested in accordance with ASTM D4829. Therefore, it is GeoTek’s opinion that conventional foundations supported by engineered fill may be used for this site.

The conventional foundation elements for the proposed buildings should bear entirely in engineered fill soils. Foundations should be designed in accordance with the 2022 California Building Code. Expansion index and soluble sulfate evaluation of the soils should be performed during construction to evaluate the as-graded conditions. Final recommendations should be based upon the as-graded soils conditions. A summary of GeoTek’s preliminary foundation design recommendations is presented in the table below:

Design Parameter	"Very Low" (0≤EI≤20) and "Low" (21≤EI≤50) Expansion Index	"Medium" Expansion Index (51≤EI≤90)
Foundation Depth or Minimum Perimeter Beam Depth (inches below lowest adjacent grade)	12 - One- and two- Stories	18 – One- and two- Stories
Minimum Foundation Width (Inches)*	12 – One- and two- Stories	15 – One- and two- Stories
Minimum Slab Thickness (actual)	4 inches – Actual	4 inches – Actual
Minimum Slab Reinforcing	6" x 6" – W2.9/W2.9 welded wire fabric placed in middle of slab or No. 3 bars at 18-inch centers.	No. 3 bars at 18-inch on-center, each way, placed in middle of slab
Minimum Footing Reinforcement	Two No. 4 Reinforcing Bars, one top and one bottom	Four No. 4 Reinforcing Bars, two near top and two near bottom
Effective Plasticity Index**	< 15	< 20
Presaturation of Subgrade Soil (Percent of Optimum)	Minimum 100% (Very Low) and 110% (Low) to a depth of 12 inches prior to placement of concrete	Minimum 120% to a depth of at least 18 inches prior to placement of concrete.

*Code minimums per Table 1809.7 of the 2022 CBC.
**Plasticity Index testing should be performed following site grading

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 of 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 450 psf for each additional foot of footing depth and 150 psf for each additional foot of footing width to a maximum value of 3,000 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 2022 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. Seismic-induced total and differential settlements over a 40-foot horizontal distance of up to 1/3 inch and ¼ inch, respectively, should also be considered as part of the structural design.

5.3.1.3 The passive earth pressure may be computed as an equivalent fluid having a density of 230 psf per foot of depth, to a maximum earth pressure of 3,000 psf for footings cast adjacent to competent engineered fill, competent alluvial soil, or bedrock. 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 2022 California Green Building Standards Code (CALGreen) Section 4.505.2, the 2022 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.

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 2022 CBC or City of Hemet 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

Based on the chemical test results performed by GeoTek's subconsultant (Project X Corrosion Engineering, Inc., 2021) on fifteen samples collected from the site as presented in Appendix B, the corrosivity test results indicate that the on-site soils are "*highly corrosive*" to "*essentially non-corrosive*" to buried ferrous metal (2,144 to 24,120 ohm-cm). This corrosion classification is obtained from "Handbook of Corrosion Engineering," by Pierre R. Roberge, 2nd Edition, 2000. A full site Soil Corrosivity Evaluation, prepared by Project X Corrosion Engineering, Inc., (Project X, 2021) is presented in Appendix E of this report. The Soil Corrosivity Evaluation includes recommendations to mitigate soil corrosivity.

5.3.5 Soil Sulfate Content

The soil sulfate content was determined in the laboratory on previous samples collected during the previous field investigation (GeoTek, 2021). The results indicate that the water-soluble sulfate result is less than 0.1 percent by weight, which is considered "negligible" (S0 exposure category) as per Table 19.3.1.1 of ACI 318-19. Based on the test results and Table 19.3.1.1 of ACI 318-19, no special recommendations for concrete are required for this project due to soil sulfate exposure.

5.4 RETAINING DESIGN AND CONSTRUCTION

5.4.1.1 General Design Criteria

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

Retaining wall foundations should be embedded a minimum of 18 inches into engineered fill.

Retaining wall foundations should be designed in accordance with Section 5.3 of this report. Structural needs may govern and should be evaluated by the project structural engineer. All earth retention structure plans, as applicable, should be reviewed by this office prior to finalization.

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

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

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

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

5.4.1.2 Cantilevered Walls

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

ACTIVE EARTH PRESSURES	
Surface Slope of Retained Materials (horizontal:vertical)	Equivalent Fluid Pressure (pcf) Select Backfill* and Native Soils
Level	40
2:1	65

*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 six (6) feet, an incremental seismic pressure should be included into the wall design. Where needed, it is recommended that an incremental seismic pressure of $18H^2$ should be included into the wall design to account for seismic loading conditions, where H is the retained height of the wall. This pressure may be applied as a conventional triangular distribution.

5.4.1.3 Retaining Wall Backfill and Drainage

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

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

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

5.4.1.4 Restrained Retaining Walls

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

5.4.1.5 Other Design Considerations

- Wall design should consider the additional surcharge loads from adjacent 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

Although planned final grades beneath the street improvements within the site are not yet known, the following preliminary pavement design recommendations are based on assumed Traffic Indexes of 5.0 and 6.0. Preliminary pavement thickness design is based on the Caltrans Highway Design Manual (2018). An R-value of 40 has been assumed for the preliminary design of the project pavement sections. Once the traffic loading information becomes more defined, revision to the pavement design recommendations may be warranted. It is recommended that the final pavement design be based on R-value testing of the as-graded subgrade soils within the pavement areas.

Based on the assumptions noted above the following preliminary pavement recommendations are provided for the site:

PRELIMINARY MINIMUM PAVEMENT SECTION		
Traffic Index	Thickness of Asphalt Concrete (inches)	Thickness of Aggregate Base (inches)
5.0	4.0*	4.0
6.0	4.0*	6.0

*D.R. Horton minimum

Traffic Indices (TIs) used in the pavement design 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.

All base material and the upper 12 inches of subgrade should be compacted to at least 95 percent of the material's maximum dry density as determined by ASTM D 1557 test procedures. All materials and methods of construction should conform to the requirements of the City of Hemet.

5.6 CONCRETE CONSTRUCTION

5.6.1 General

Concrete construction should follow the 2022 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-0304824-CR) dated March 8, 2024, 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.

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NORTH

0 ft 1000 ft

Legend

Approximate Site Limits

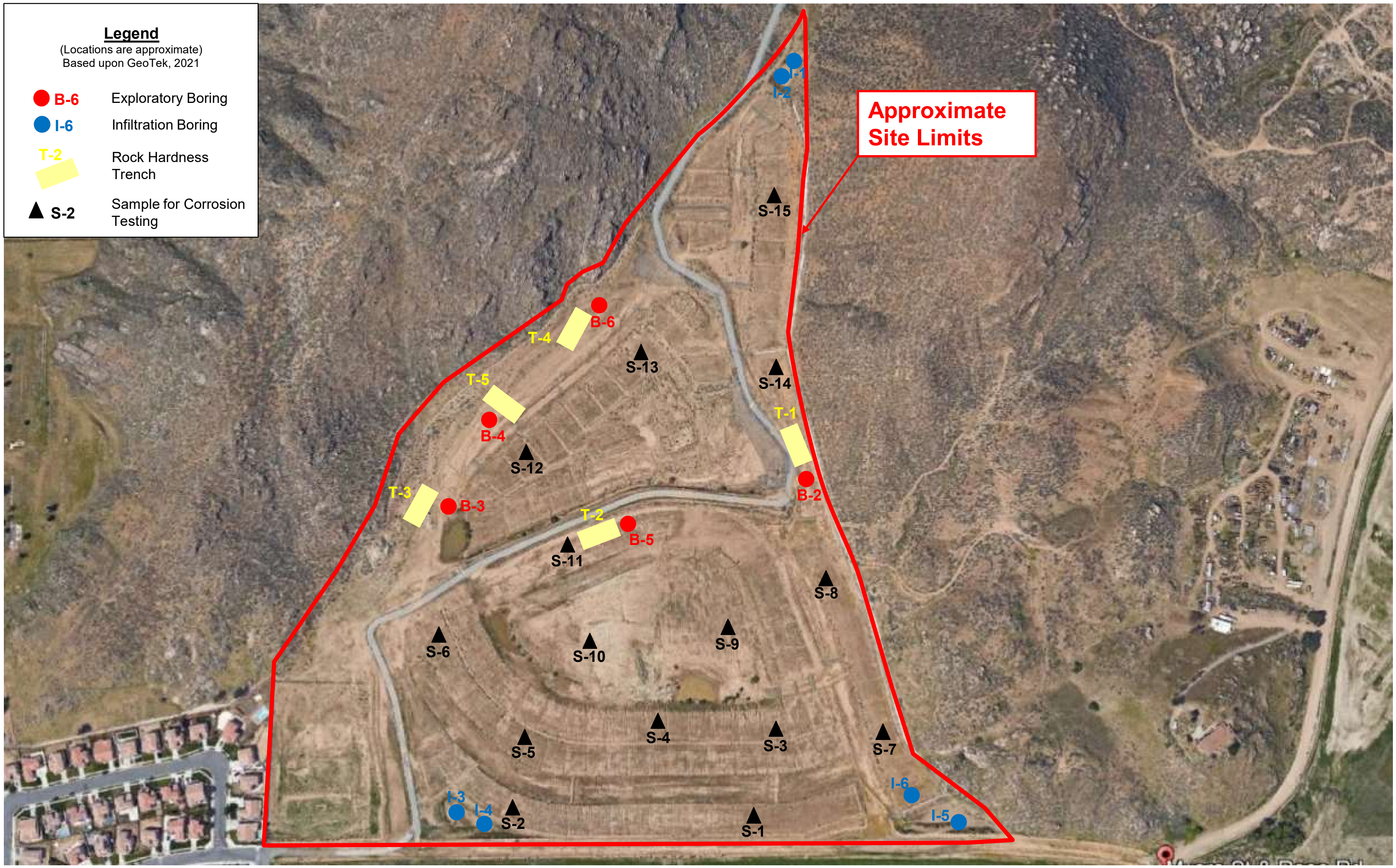
D.R. Horton Los Angeles Holding Company, Inc.
Tract No. 31513
Hemet, Riverside County, California

Project No. 2593-CR

GEOTEK

Figure 1

Site Location Map



1000 FEET

NORTH

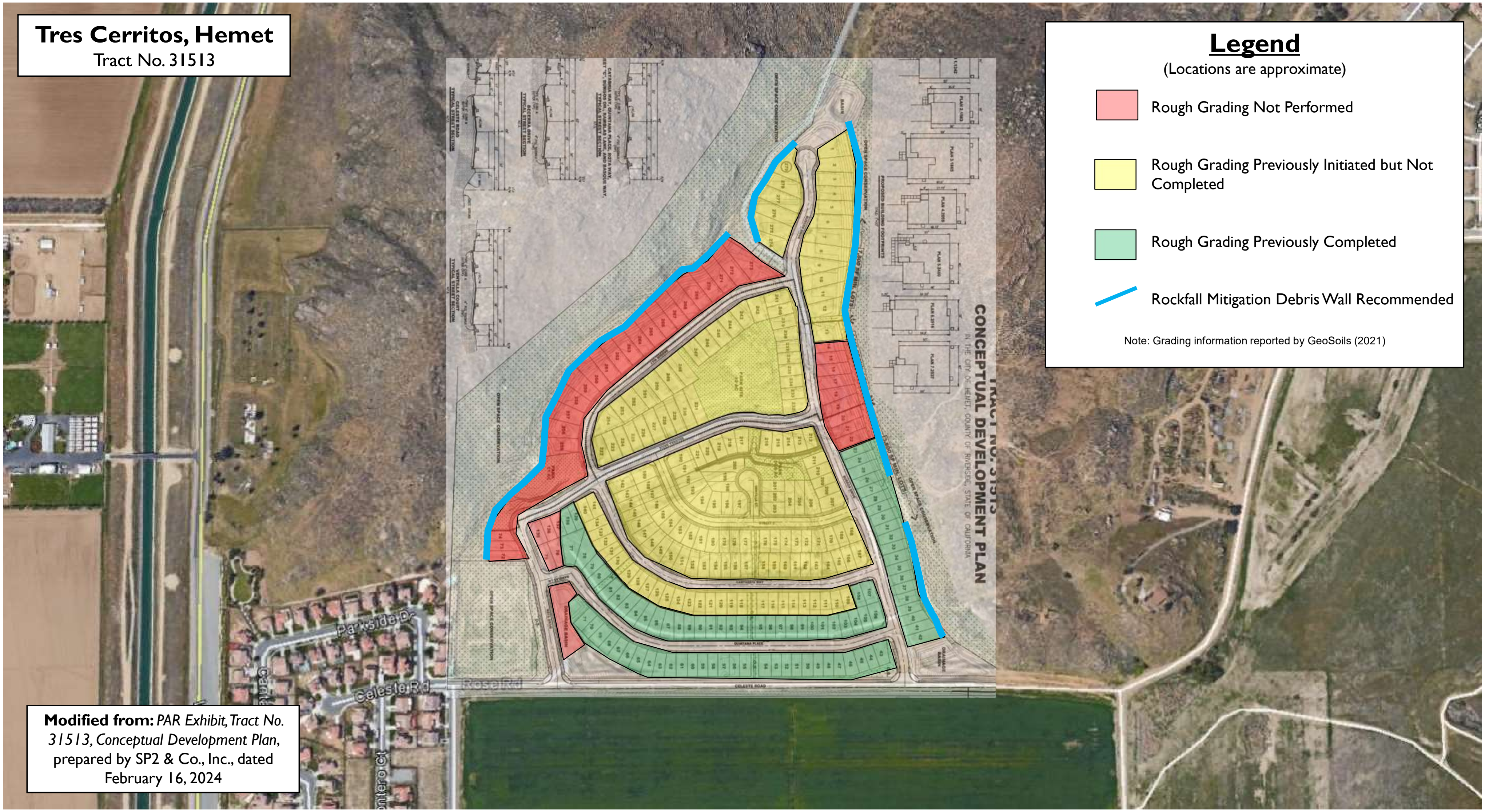
D.R. Horton Los Angeles Holding Company, Inc.
Tract No. 31513
Hemet, Riverside County, California

Project No. 2593-CR

GEOTEK

Figure 2

Site Exploration Map



NORTH

0 ft — 1000 ft

D.R. Horton Los Angeles Holding Company, Inc. Tract No. 31513 Hemet, Riverside County, California Project No. 2593-CR	GEOTEK	**Figure 3** **Geotechnical Conditions Map**

APPENDIX A

LOGS OF EXPLORATORY EXCAVATIONS (GeoTek, 2021)

Updated Geotechnical Evaluation
Proposed Single-Family Residential Development
Tract No. 31513
Hemet, Riverside County, California
Project No. 2593-CR

GEOTEK

A - FIELD TESTING AND SAMPLING PROCEDURES

The Modified Split-Barrel Sampler (Ring)

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

The Standard Penetration Test (SPT) Sampler

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. Blow counts are recorded for every 6 inches of penetration as indicated on the log of boring. Disturbed samples are removed from the sample barrel, sealed in a plastic bag, 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)

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LOG OF EXPLORATORY BORING

GEOTEK

CLIENT:	DR Horton	**DRILLER:**	2R	**LOGGED BY:**	JE
PROJECT NAME:	Tract No. 31513	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Jerry
PROJECT NO.:	2593-CR	**HAMMER:**	140 lbs, 30 in.	**RIG TYPE:**	CME 75
LOCATION:	Hemet, CA			**DATE:**	1/12/2021

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	**BORING NO.: B-2** **MATERIAL DESCRIPTION AND COMMENTS**	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Existing Fill			
	Ring	50/3"		SM	Silty f-m SAND, light brown, slightly moist, , very dense, trace cobble			
	Ring	50/3"		SM	Same as above, some gravel and cobbles			
5								
					Granitic Bedrock			
	Ring	50/1.5"			Excavates as silty f-c SAND, light brown, slightly moist, very dense, some cobble			
					BORING TERMINATED AT 9' (REFUSAL)			
10					No groundwater encountered Boring Backfilled with soil cuttings			
15								
20								
25								
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/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	DR Horton	**DRILLER:**	2R	**LOGGED BY:**	JE
PROJECT NAME:	Tract No. 31513	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Jerry
PROJECT NO.:	2593-CR	**HAMMER:**	140 lbs, 30 in.	**RIG TYPE:**	CME 75
LOCATION:	Hemet, CA			**DATE:**	1/12/2021

BORING NO.: B-3

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Granitic Bedrock			
	SPT	50/2"			Excavates as silty f-c SAND, brown to grayish brown, slightly moist to moist, very dense	0.6		
5	SPT	50/1.5"			Same as above, difficult to excavate	1.1		
	SPT	50/2.5"			Excavates as f-c SAND, dark gray to brown, slightly moist, very dense			
10								
	SPT	50/1.5"			No recovery Same as above			
15					**BORING TERMINATED AT 15' (REFUSAL)**			
					No groundwater encountered Boring Backfilled with soil cuttings			
20								
25								
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/Resisitivity Test	SH = Shear Test	HC= Consolidation	MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

GEOTEK

CLIENT:	DR Horton	**DRILLER:**	2R	**LOGGED BY:**	JE
PROJECT NAME:	Tract No. 31513	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Jerry
PROJECT NO.:	2593-CR	**HAMMER:**	140 lbs, 30 in.	**RIG TYPE:**	CME 75
LOCATION:	Hemet, CA			**DATE:**	1/12/2021

BORING NO.: B-4

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Existing Fill			
	SPT	29 50/5"		SP	F-c SAND, grayish brown, slightly moist to moist, very dense, some cobble	0.6		
5								
	SPT	50/2.5"		SP	Same as above, difficult to excavate	2.2		
					Granitic Bedrock			
	SPT	50/4"			Excavates as gravelly f-c SAND, grayish brown, slightly moist	4.3		
10								
					BORING TERMINATED AT 11.5' (REFUSAL)			
					No groundwater encountered Boring Backfilled with soil cuttings			
15								
20								
25								
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/Resisitivity Test SH = Shear Test HC= Consolidation MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	DR Horton	**DRILLER:**	2R	**LOGGED BY:**	JE
PROJECT NAME:	Tract No. 31513	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Ish
PROJECT NO.:	2593-CR	**HAMMER:**	140 lbs, 30 in.	**RIG TYPE:**	CME 75
LOCATION:	Hemet, CA			**DATE:**	1/12/2021

BORING NO.: B-5

Depth (ft)	Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
0	Large Bulk				**Existing Fill**			EI=0 SH MD
5	Ring	12 25 42		SM	Silty f-m SAND, light brown, slightly moist to moist, dense	6.9	125.4	
					Older Alluvium			
10	Ring	37 50/4.5"		SM	Silty f-c SAND, brown, moist, very dense, difficult to excavate	6.5	132.3	HC
15	Ring	28 42 50/5"		SM	Silty f-c SAND, light brown to brown, moist, very dense	4.5	122.5	HC
20	Ring	28 39 50/5"		SM	Silty f-m SAND, brown, slightly moist, very dense	4.3	122.0	fines=19.3%
25	Ring	20 46 50/5"		SC	Clayey f-c SAND, brown, slightly moist, very dense	7.3	122.6	
30	SPT	24 36 40		SM	Silty f-m SAND, brown to dark brown, slightly moist, dense			fines=34.2%

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/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	DR Horton	**DRILLER:**	2R	**LOGGED BY:**	JE
PROJECT NAME:	Tract No. 31513	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Ish
PROJECT NO.:	2593-CR	**HAMMER:**	140 lbs, 30 in.	**RIG TYPE:**	CME 75
LOCATION:	Hemet, CA			**DATE:**	1/12/2021

BORING NO.: B-5 cont.

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
35	SPT	24 24 26		SM	Silty f-c SAND, grayish brown to brown, slightly moist, some cobble			
40	SPT	18 32 50/5"		SM	Silty f-m SAND, light brown to brown, slightly moist, very dense			fines=34.0%
45	SPT	16 20 21		SM	Silty f-c SAND, light brown, slightly moist, medium dense			
50	SPT	15 22 30		SM-ML	Silty f-m SAND to sandy SILT, brown to dark brown, slightly moist, dense			
					BORING TERMINATED AT 51.5' No groundwater encountered Boring backfilled with soil cuttings			
55								
60								

LEGEND

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

Lab testing:
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SR = Sulfate/Resisitivity Test SH = Shear Test HC= Consolidation MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	DR Horton	**DRILLER:**	2R	**LOGGED BY:**	JE
PROJECT NAME:	Tract No. 31513	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Jerry
PROJECT NO.:	2593-CR	**HAMMER:**	140 lbs, 30 in.	**RIG TYPE:**	CME 75
LOCATION:	Hemet, CA			**DATE:**	1/12/2021

BORING NO.: B-6

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Existing Fill			
	Ring	16 26 29		SM	Silty f-m SAND, brown, slightly moist, dense	2.5	114.8	HC
5	Ring	21 40 50/5.5"		SM	Becomes very dense, some cobbles	1.7	124.4	
	Ring	33 41 50/5.5"		SM	Silty f-c SAND, light brown, slightly moist, very dense	1.9	120.7	
10								
					Granitic Bedrock			
	Ring	50/3"			Excavates as gravelly f-c SAND, grayish brown, slightly moist			
15								
	Ring	50/1"			Excavates as f-c SAND, dark gray, slightly moist, some cobbles			
					BORING TERMINATED AT 19.5'			
20					No groundwater encountered Boring Backfilled with soil cuttings			
25								
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/Resisitivity Test SH = Shear Test HC= Consolidation MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

GEOTEK

CLIENT:	DR Horton	**DRILLER:**	2R	**LOGGED BY:**	JE
PROJECT NAME:	Tract No. 31513	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Jerry
PROJECT NO.:	2593-CR	**HAMMER:**	140 lbs, 30 in.	**RIG TYPE:**	CME 75
LOCATION:	Hemet, CA			**DATE:**	1/12/2021

Depth (ft)	SAMPLES			USCS Symbol	BORING NO.: I-1	Laboratory Testing		
	Sample Type	Blows/ 6 in	Sample Number		**MATERIAL DESCRIPTION AND COMMENTS**	Water Content (%)	Dry Density (pcf)	Others
					Alluvium			
				SM	Silty f-m SAND, grayish brown, moist			
5					**BORING TERMINATED AT 5'**			
					No groundwater encountered			
10								
15								
20								
25								
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/Resisitivity Test	SH = Shear Test	HC= Consolidation	MD = Maximum Density

GeoTek, Inc.
LOG OF EXPLORATORY BORING

CLIENT:	DR Horton	**DRILLER:**	2R	**LOGGED BY:**	JE
PROJECT NAME:	Tract No. 31513	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Jerry
PROJECT NO.:	2593-CR	**HAMMER:**	140 lbs, 30 in.	**RIG TYPE:**	CME 75
LOCATION:	Hemet, CA			**DATE:**	1/12/2021

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	**BORING NO.: I-2** **MATERIAL DESCRIPTION AND COMMENTS**	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Alluvium			
				SM	Silty f-m SAND, grayish brown, moist			
5					**BORING TERMINATED AT 5'**			
					No groundwater encountered			
10								
15								
20								
25								
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/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

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LOG OF EXPLORATORY BORING

GEOTEK

CLIENT:	DR Horton	**DRILLER:**	2R	**LOGGED BY:**	JE
PROJECT NAME:	Tract No. 31513	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Jerry
PROJECT NO.:	2593-CR	**HAMMER:**	140 lbs, 30 in.	**RIG TYPE:**	CME 75
LOCATION:	Hemet, CA			**DATE:**	1/12/2021

BORING NO.: I-3

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Alluvium			
				SM/CL	Silty f-c SAND to sandy CLAY, light brown to brown, slightly moist to moist			
5					**BORING TERMINATED AT 5'**			
					No groundwater encountered			
10								
15								
20								
25								
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/Resisitivity Test
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MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

GEOTEK

CLIENT:	DR Horton	**DRILLER:**	2R	**LOGGED BY:**	JE
PROJECT NAME:	Tract No. 31513	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Jerry
PROJECT NO.:	2593-CR	**HAMMER:**	140 lbs, 30 in.	**RIG TYPE:**	CME 75
LOCATION:	Hemet, CA			**DATE:**	1/12/2021

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	BORING NO.: I-4 — MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Alluvium			
				SM/CL	Silty f-c SAND to sandy CLAY, light brown to brown, slightly moist to moist,			
5					**BORING TERMINATED AT 5'**			
					No groundwater encountered			
10								
15								
20								
25								
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/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	DR Horton	**DRILLER:**	2R	**LOGGED BY:**	JE
PROJECT NAME:	Tract No. 31513	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Jerry
PROJECT NO.:	2593-CR	**HAMMER:**	140 lbs, 30 in.	**RIG TYPE:**	CME 75
LOCATION:	Hemet, CA			**DATE:**	1/12/2021

BORING NO.: I-5

Depth (ft)	SAMPLES: Sample Type	Blows/ 6 in	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
					Alluvium			
				SM	Silty f-c SAND, light brown to brown, moist			
5					**BORING TERMINATED AT 5'**			
					No groundwater encountered			
10								
15								
20								
25								
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/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

GEOTEK

CLIENT:	DR Horton	**DRILLER:**	2R	**LOGGED BY:**	JE
PROJECT NAME:	Tract No. 31513	**DRILL METHOD:**	Hollow Stem Auger	**OPERATOR:**	Jerry
PROJECT NO.:	2593-CR	**HAMMER:**	140 lbs, 30 in.	**RIG TYPE:**	CME 75
LOCATION:	Hemet, CA			**DATE:**	1/12/2021

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/ 6 in	SAMPLES: Sample Number	USCS Symbol	BORING NO.: I-6 — MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Alluvium			
				SM/CL	Silty f-c SAND to sandy CLAY, light brown to brown, slightly moist to moist			
5					**BORING TERMINATED AT 5'**			
					No groundwater encountered			
10								
15								
20								
25								
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/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY TRENCH

CLIENT:	DR Horton	**LOGGED BY:**	DRW
PROJECT NAME:	Tract No. 31513	**EQUIPMENT**	100K Excavator
PROJECT NO.:	2593-CR	**DATE:**	1/12/2021
LOCATION:	See Trench Location Map		

TRENCH NO.: TP-1

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Time for Excavation	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
				Existing Fill:			
				Silty f-c SAND, brown, moist, some cobbles			
			SM				
		0:48					
				Cobble layer approximately 1 foot thick			
		2:09					
5							
		3:37		Silty f-c SAND, brown, moist, some gravel, trace cobbles			
				Granitic Bedrock:			
		5:23		Weathered granitic bedrock, easy to excavate			
10							
		6:52		Becomes indurated			
		8:31		Excavates as silty f-c SAND, brown, moist, trace cobbles, 2-3 scratches for 1/2 to full bucket			
		11:53		Same as above			
15							
		14:52		Becomes hard to excavate, 3-4 scratches for 1/2 bucket, no "floaters"			
		24:53		Becomes very hard, 5+ scratches for 1/2 bucket			
20		30:18		Same as above, excavator was fully extended			
				TRENCH TERMINATED AT 20.5 FEET			
				No groundwater encountered Boring backfilled with excavated soils Trench dimensions: 6'W x 22'L x 20'D			

LEGEND

Sample type: ---Ring ---Large Bulk ---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 TRENCH

CLIENT:	DR Horton	**LOGGED BY:**	DRW
PROJECT NAME:	Tract No. 31513	**EQUIPMENT**	100K Excavator
PROJECT NO.:	2593-CR	**DATE:**	1/12/2021
LOCATION:	See Trench Location Map		

TRENCH NO.: TP-2

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Time for Excavation	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
				Existing Fill:			
		0:41	SM	Silty f-c SAND, brown, moist, trace gravel			
		1:05		Same as above, trace gray mottles			
5							
		1:26		**Older Alluvium:**			
			SM	Silty f-c SAND, brown, moist, easy to excavate			
		2:56					
10		4:12		Becomes slightly harder to excavate			
		5:33					
		7:08		2 scratches for a full bucket			
15							
		8:55	SM	Silty f-c SAND to f-c SAND, brown, moist, very dense			
		10:32					
			SM/SC	Silty clayey f-c SAND, light brown, slightly moist to moist			
20		13:00					

TRENCH TERMINATED AT 20.5 FEET

No groundwater encountered
Boring backfilled with excavated soils
Trench dimensions: 6'W x 20'L x 20'D

LEGEND

Sample type: ---Ring ---Large Bulk ---Water Table

Lab testing:

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

GeoTek, Inc.
LOG OF EXPLORATORY TRENCH

CLIENT:	DR Horton	**LOGGED BY:**	DRW
PROJECT NAME:	Tract No. 31513	**EQUIPMENT**	100K Excavator
PROJECT NO.:	2593-CR	**DATE:**	1/12/2021
LOCATION:	See Trench Location Map		

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Time for Excavation	USCS Symbol	TRENCH NO.: TP-3 — MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
				Granitic Bedrock:			
			SM	Excavates as silty f-c SAND, brown to grayish brown, slightly moist to moist, slightly difficult to excavate			
		0:40		Becomes very hard to excavate, 5+ scratches to get 1/2 bucket, sparking when scratching Hard to move to the south 10 feet due to "unexcavatable floaters"			
		2:18		Hard to excavate, 4-5 scratches for 1/2 to full bucket			
5							
		8:47		Becomes hard to excavate, 5+ scratches for 1/2 to full bucket			
		12:41		Refusal within the trench bottom			
				TRENCH TERMINATED AT 7.5 FEET			
				No groundwater encountered Boring backfilled with excavated soils Trench dimensions: 8'W x 20'L x 7.5'D			
10							
15							
20							

LEGEND

Sample type: ---Ring ---Large Bulk ---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 TRENCH

CLIENT:	DR Horton	**LOGGED BY:**	DRW
PROJECT NAME:	Tract No. 31513	**EQUIPMENT**	100K Excavator
PROJECT NO.:	2593-CR	**DATE:**	1/12/2021
LOCATION:	See Trench Location Map		

TRENCH NO.: TP-4

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Time for Excavation	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Dry Density (pcf)	Others
				Existing Fill:			
			SM	Excavates as silty f-c SAND, brown, slightly moist to moist, some cobbles			
		0:32					
		1:03					
				Granitic Bedrock:			
5				Slightly weathered, easy to excavate			
		2:34		Moderately easy to excavate 2-3 scratches for 1/2 to full bucket			
				Becomes indruated and hard to excavate			
		4:41		Same as above			
10		7:17		becomes hard to excavate, 4-5 scratches for 1/2 to full bucket			
		12:12					
				Becomes very hard to excavate, 5+ scratches for 1/2 to full bucket			
		21:38		Becomes light gray			
15							
		25:16		Refusal within the trench bottom			
				TRENCH TERMINATED AT 16.0 FEET			
				No groundwater encountered			
				Boring backfilled with excavated soils			
				Trench dimensions: 6'W x 18'L x 16'D			
20							

LEGEND

Sample type: ---Ring ---Large Bulk ---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 TRENCH

CLIENT:	DR Horton	**LOGGED BY:**	DRW
PROJECT NAME:	Tract No. 31513	**EQUIPMENT**	100K Excavator
PROJECT NO.:	2593-CR	**DATE:**	1/12/2021
LOCATION:	See Trench Location Map		

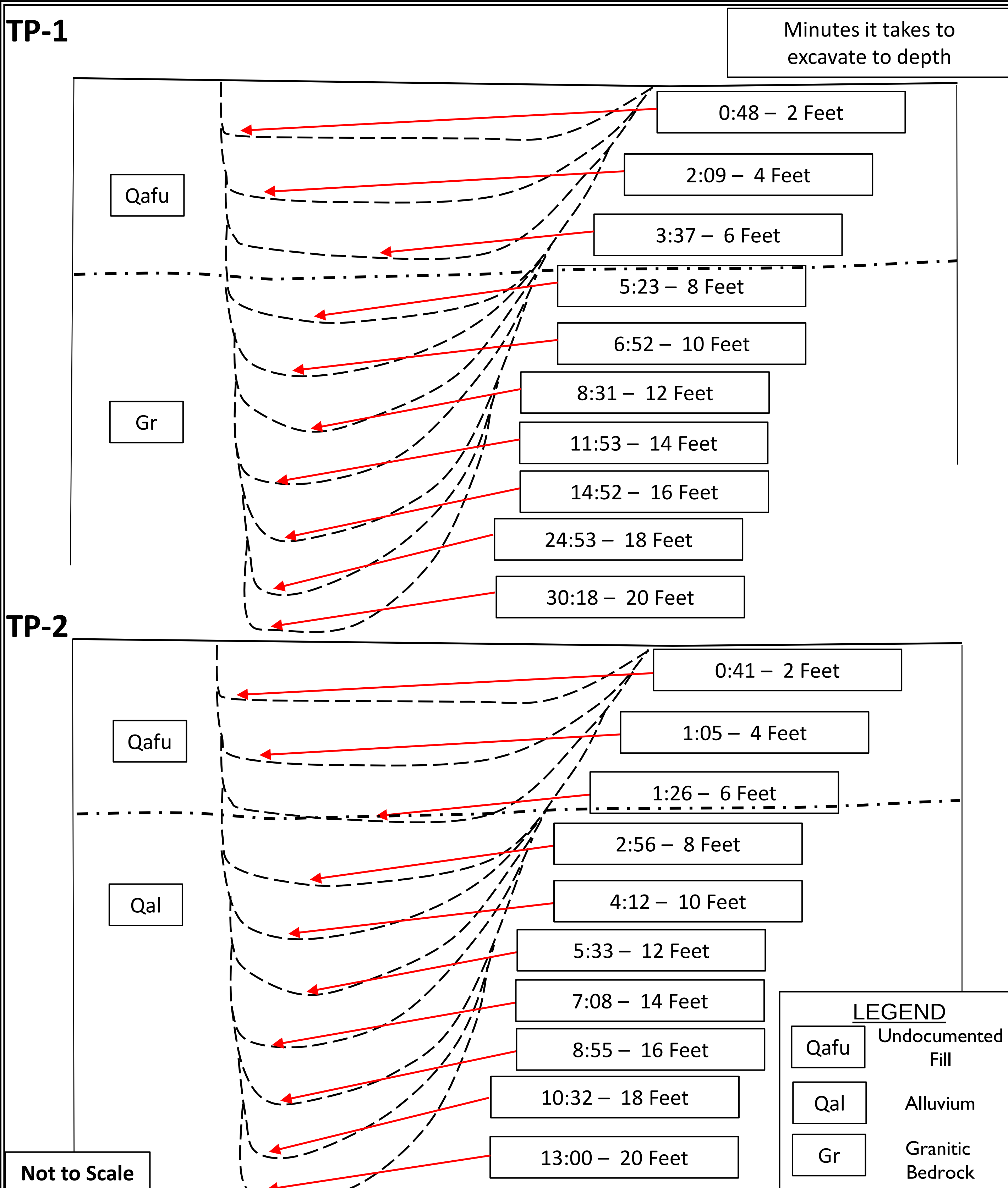
TRENCH NO.: TP-5

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Time for Excavation	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
				Granitic Bedrock:			
				Excavates as large boulders, 4-6 foot in diameter			
		0:43		Same as above, had to widen trench due to large boulders			
		3:15		Starts to excavate as silty f-c SAND, brown, moist, some gravel and cobbles			
5							
		5:42		Same as above, becomes hard, 4-5 scratches for 1/2 to full bucket			
		9:00		Becomes very hard to excavate, 5+ scratches for 1/2 to full bucket			
		14:32		Refusal within the trench bottom			
10				**TRENCH TERMINATED AT 10.0 FEET**			
				No groundwater encountered			
				Boring backfilled with excavated soils			
				Trench dimensions: 8'W x 16'L x 10'D			
15							
20							

LEGEND

Sample type: ---Ring ---Large Bulk ---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



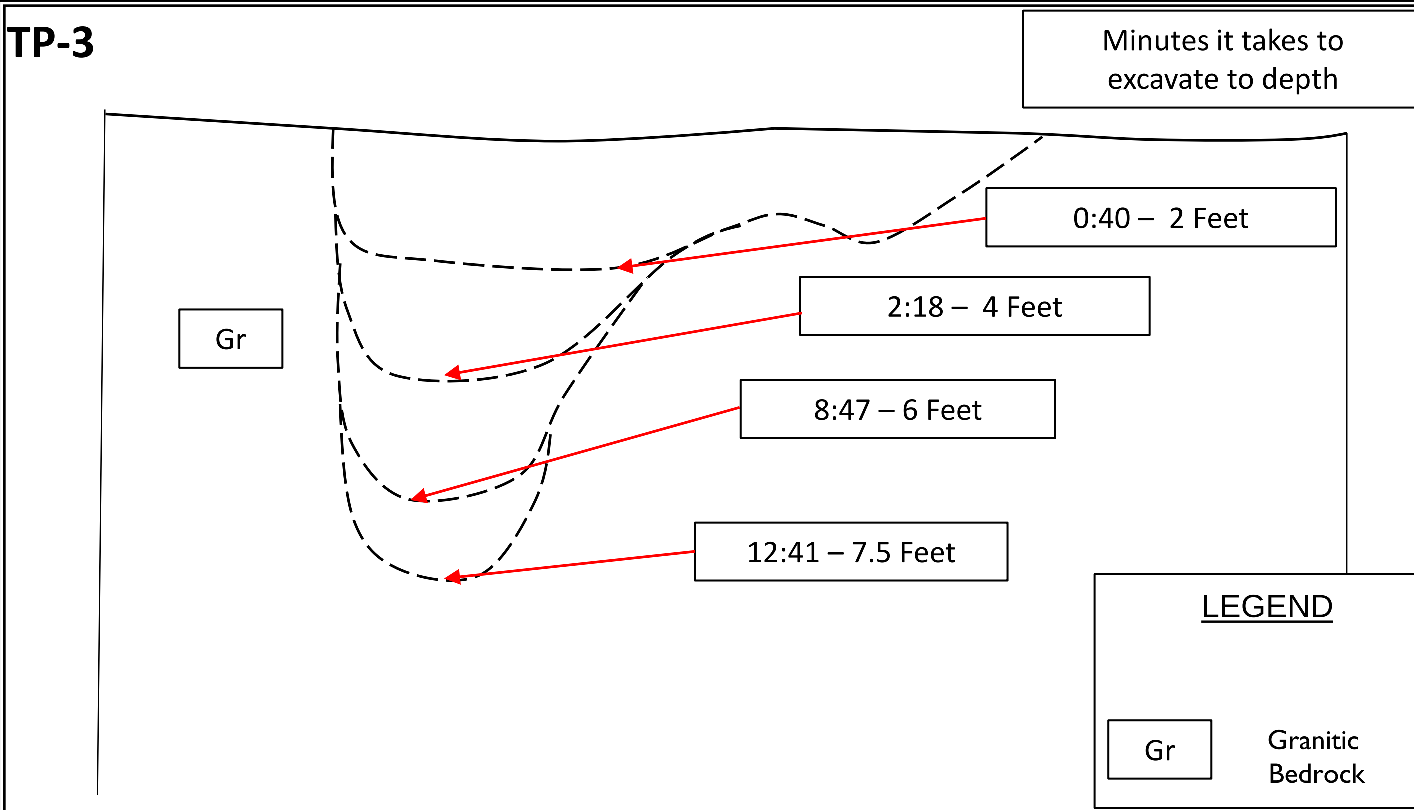
DR Horton Los Angeles Holding Company
Tracts No.. 31513
Hemet, Riverside County, California

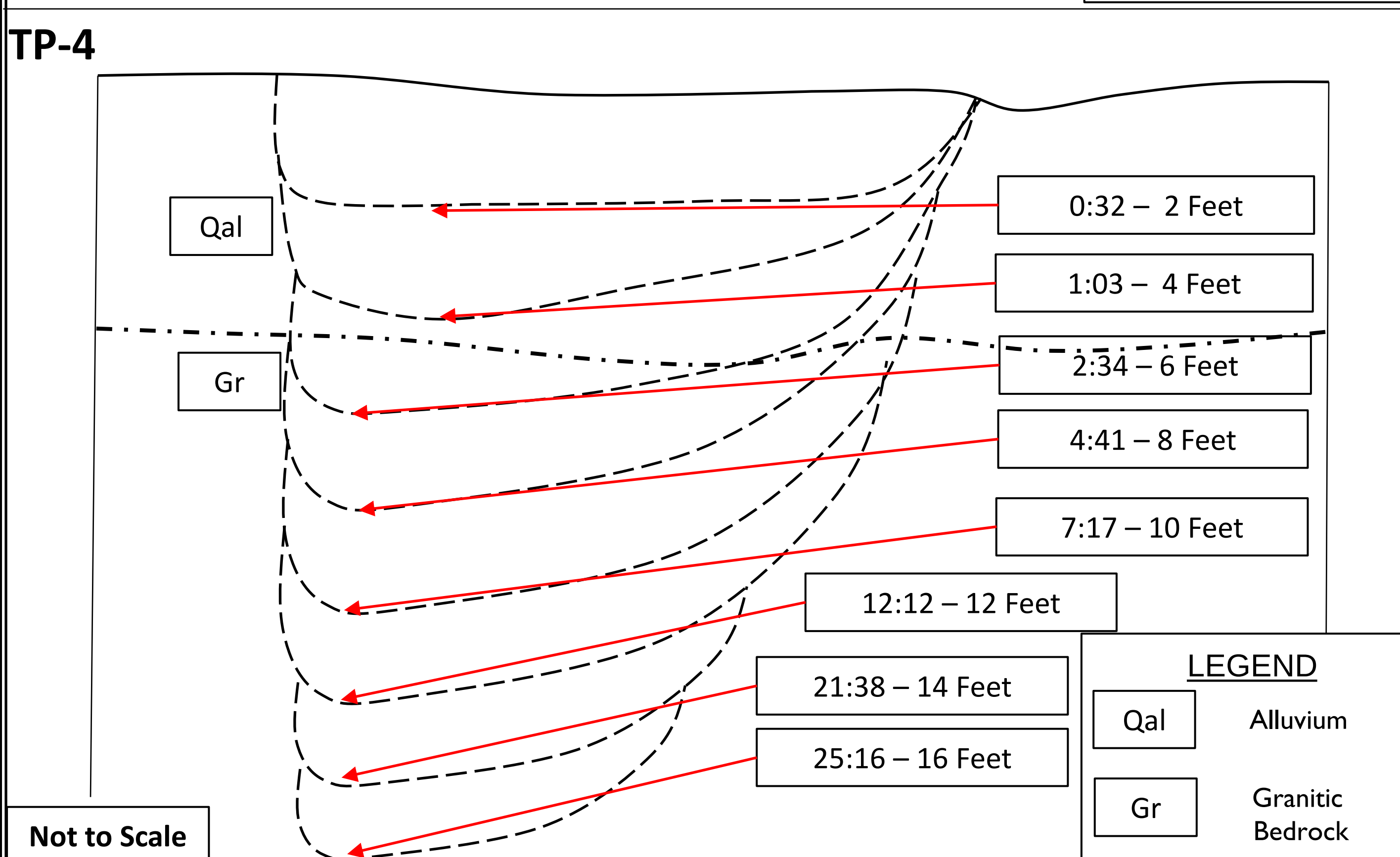
GeoTek Project No. 2593-CR

Figure I

Graphic Trench Logs

GEOTEK





DR Horton Los Angeles Holding Company
Tracts No.. 31513
Hemet, Riverside County, California

GeoTek Project No. 2593-CR

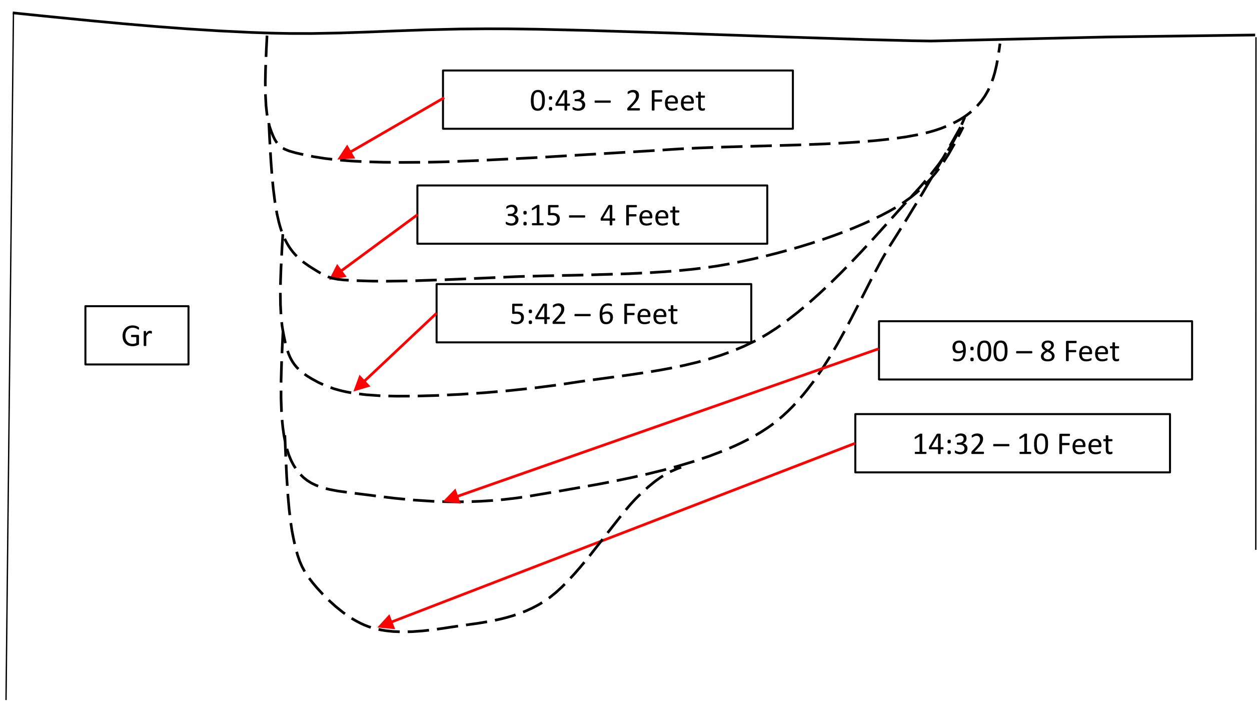
Figure 2

Graphic Trench Logs

GEOTEK

Minutes it takes to excavate to depth

TP-5



LEGEND

Gr Granitic Bedrock

Not to Scale

DR Horton Los Angeles Holding Company
Tracts No.. 31513
Hemet, Riverside County, California

GeoTek Project No. 2593-CR

Figure 3

Graphic Trench Logs

GEOTEK

APPENDIX B

LABORATORY TEST RESULTS (GeoTek, 2021)

Updated Geotechnical Evaluation
Proposed Single-Family Residential Development
Tract No. 31513
Hemet, Riverside County, California
Project No. 2593-CR

GEOTEK

SUMMARY OF LABORATORY TESTING

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

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

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

Expansion Index
Expansion Index testing was performed on one (1) soil sample obtained from the field exploration. Testing was performed in general accordance with ASTM D 4829 test procedures. The results of the testing are provided below.

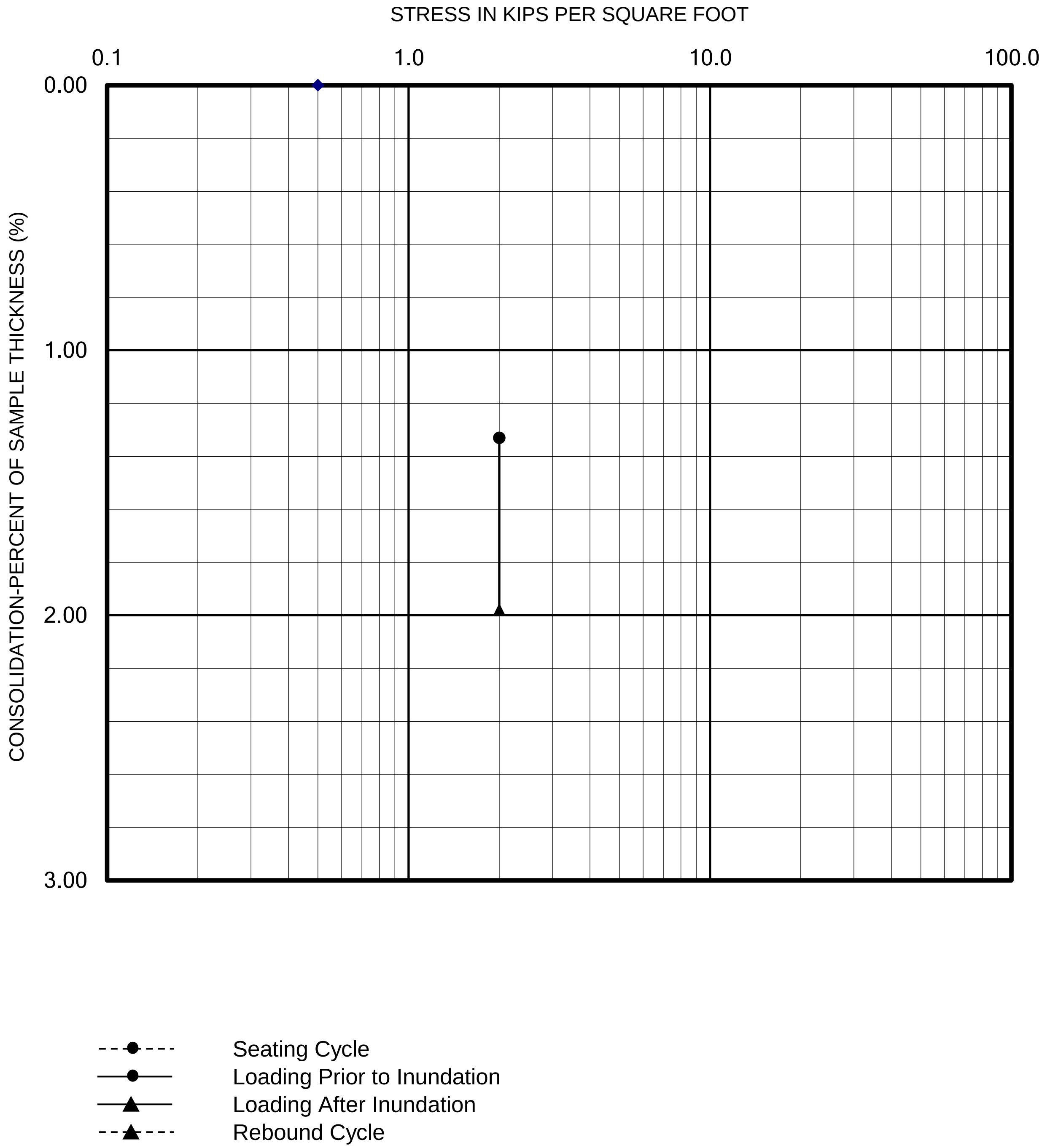
Boring #	Depth (ft.)	Description	Expansion Index	Classification
B-5	0-5	Silty Sand (SM)	0	Very Low
S-2	0-5	Sandy Clay (CL)	82	Medium
S-3	0-3	Silty Sand (SM)	0	Very Low

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 one (1) sample collected during the subsurface exploration. The laboratory maximum dry density and optimum moisture content for the soil was determined in general accordance with ASTM D 1557 test procedures. The results are presented graphically in Appendix B.

Sulfate Content, Resistivity and Chloride Content
Testing to determine the water-soluble sulfate content was performed in general accordance with ASTM D4327 test procedures. Resistivity testing was completed in general accordance with ASTM G187 test procedures. Testing to determine the chloride content was performed in general accordance with ASTM D4327 test procedures. The results of the testing are provided below and in Appendix B.

Boring No.	Depth (ft.)	pH ASTM G51	Chloride ASTM D4327 (ppm)	Sulfate ASTM D4327 (% by weight)	Resistivity ASTM G187 (ohm-cm)
S-1	0-1	7.6	9.5	0.0017	20,770
S-2	0-1	9.0	102.6	0.0272	2,144
S-3	0-1	7.4	6.1	0.0014	11,390
S-4	0-1	8.3	7.1	0.0013	10,720
S-5	0-1	7.4	45.4	0.0042	5,360
S-6	0-1	8.1	8.2	0.0006	10,720
S-7	0-1	7.0	4.1	0.0010	24,120
S-8	0-1	7.0	8.5	0.0017	13,400
S-9	0-1	6.6	7.4	0.0012	12,060
S-10	0-1	7.0	5.9	0.0009	18,760
S-11	0-1	8.3	93.6	0.0072	2,747
S-12	0-1	7.3	3.1	0.0007	20,100
S-13	0-1	7.2	3.6	0.0007	43,550
S-14	0-1	7.8	35.8	0.0038	5,360
S-15	0-1	7.0	3.3	0.0008	23,450



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

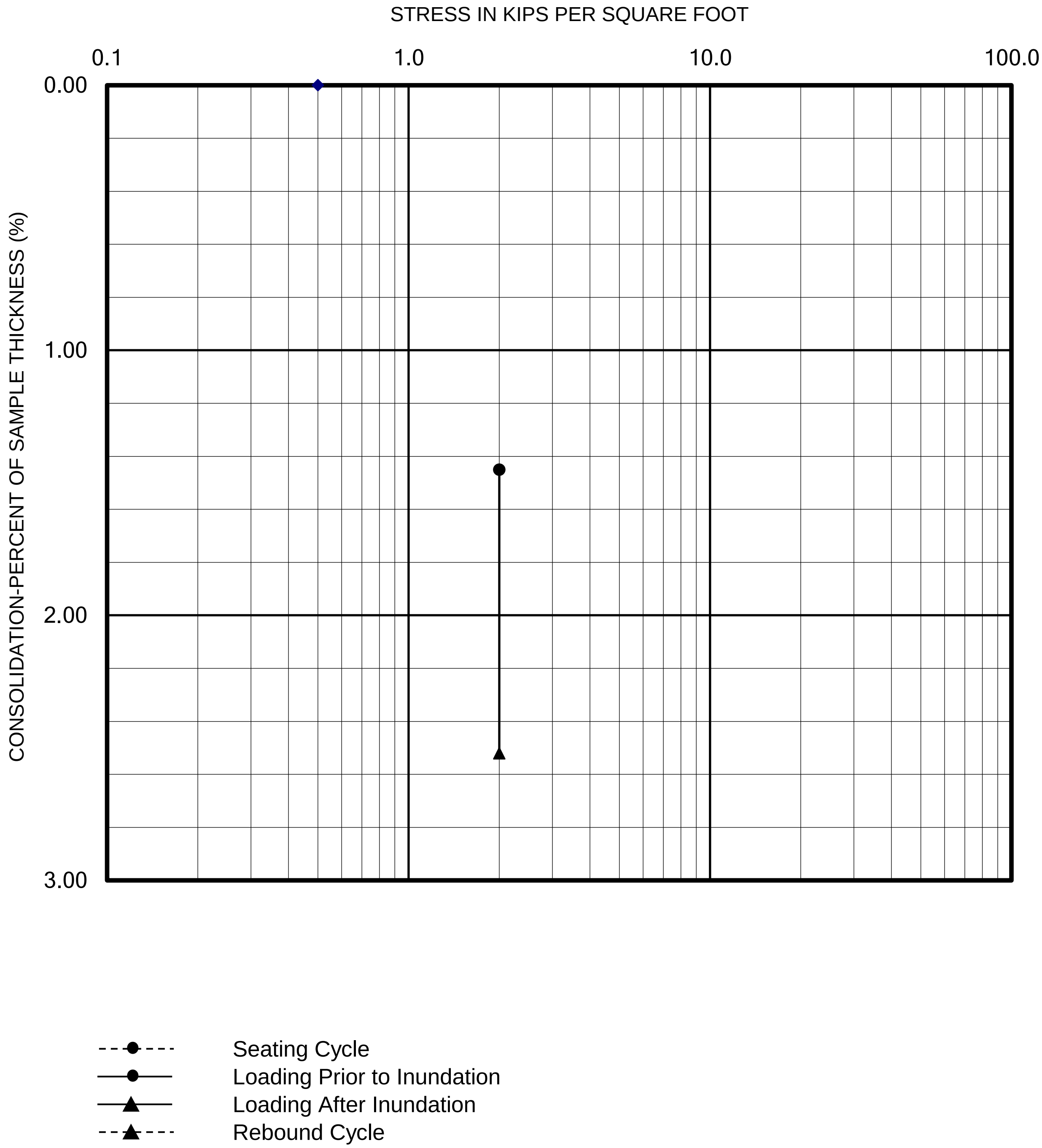
GEOTEK

COLLAPSE REPORT

Sample: B-5 @ 10 feet

Plate B-1

CHECKED BY: RJ	Lab: Corona
PROJECT NO.: 2593CR	Date: 1-27-20



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

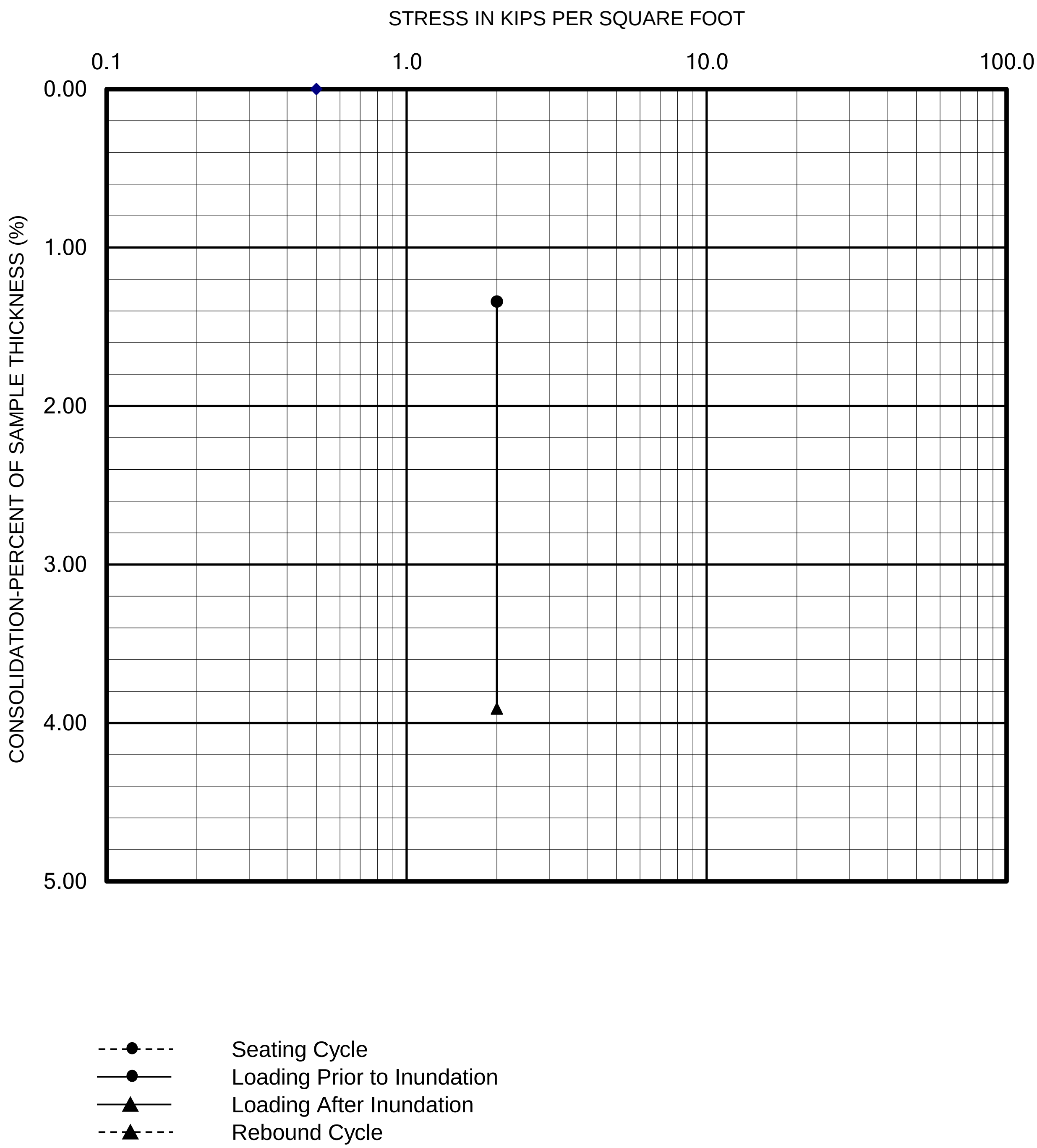
GEOTEK

COLLAPSE REPORT

Sample: B-5 @ 15 feet

Plate B-2

CHECKED BY: RJ	Lab: Corona
PROJECT NO.: 2593CR	Date: 1-27-21



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

COLLAPSE REPORT

Sample: B-6 @ 2 feet

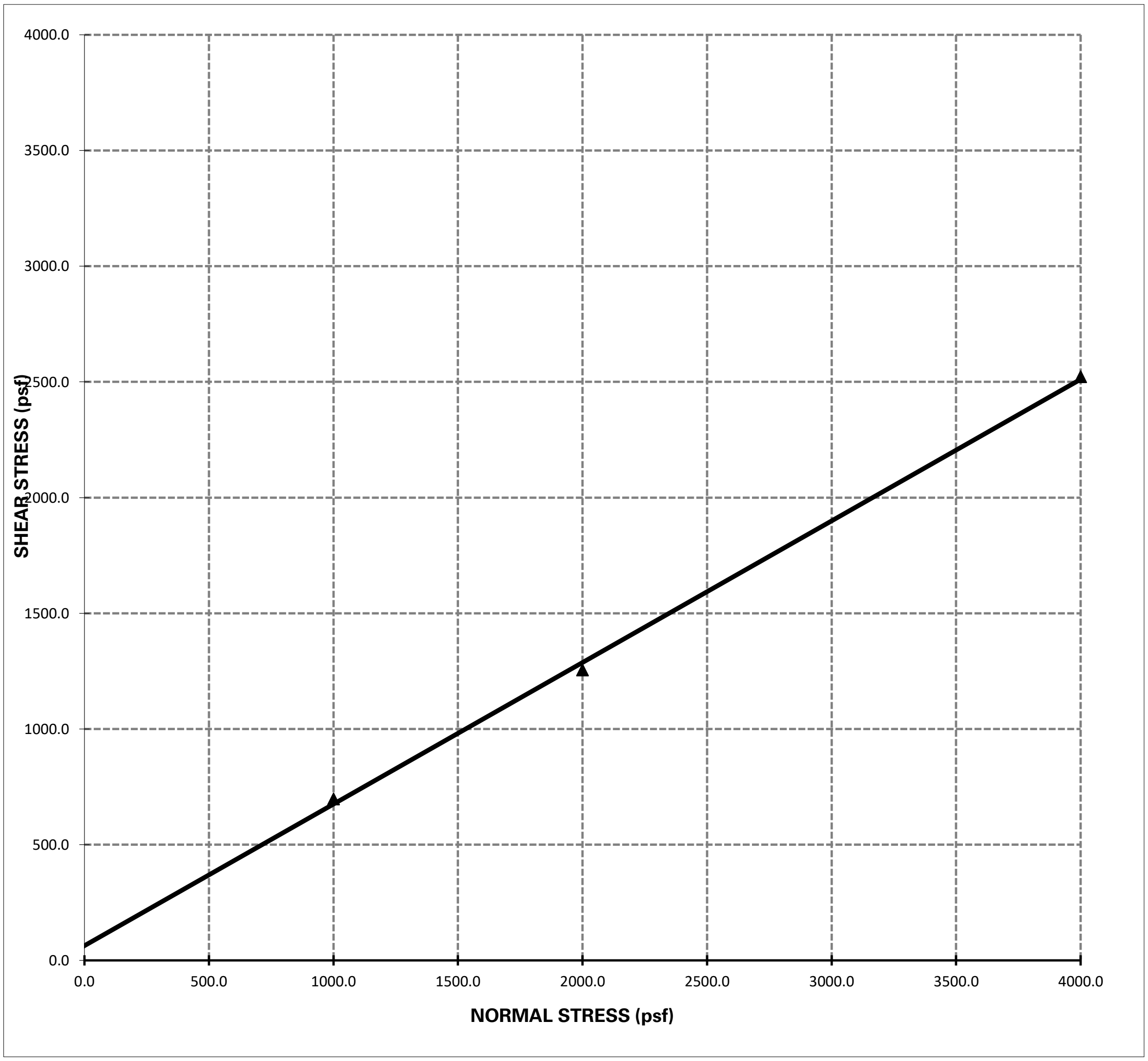
Plate B-3

CHECKED BY: RJ	Lab: Corona
PROJECT NO.: 2593CR	Date: 1-27-21

DIRECT SHEAR TEST

GEOTEK

Project Name:	Dr Horton	***Sample Location:***	B5 @ 0-5 feet
Project Number:	2593-CR	***Date Tested:***	2/2/2021



Shear Strength: **Φ = 31 °** **C = 64 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.

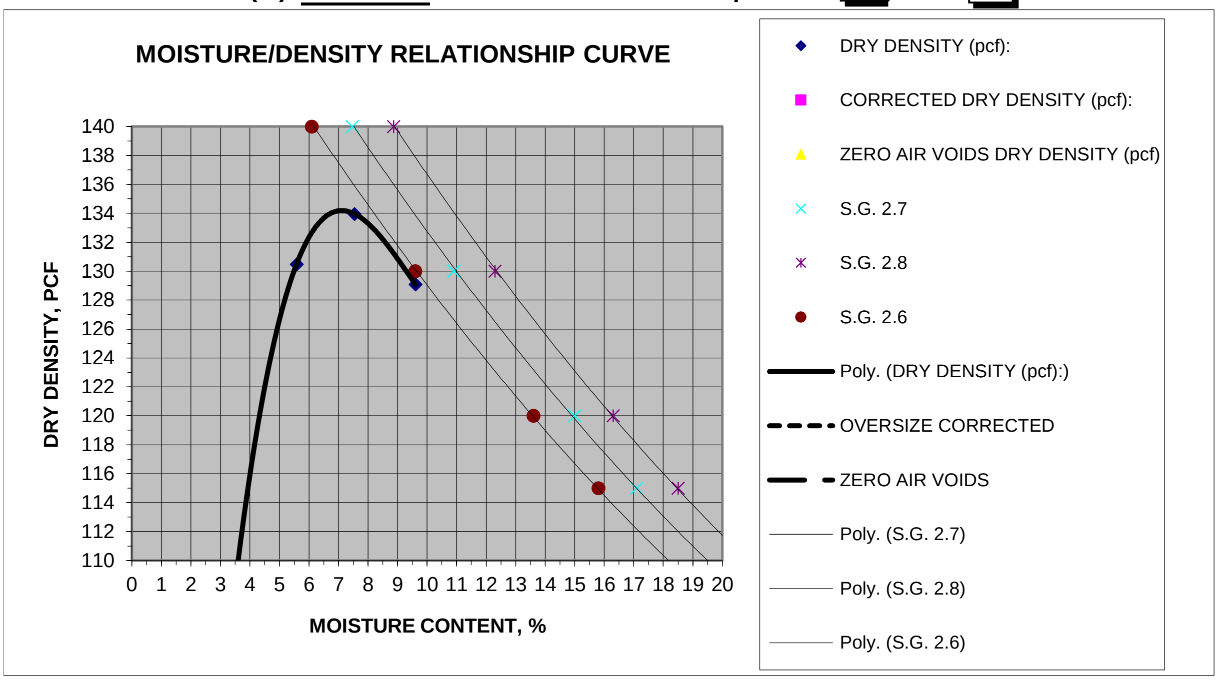
2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.01 in/min.

MOISTURE/DENSITY RELATIONSHIP

Client: Dr. Horton
Job No.: 2593-CR
Project: TR: 31513
Lab No.: Corona
Location: Hemet
Material Type: Brown Silty Sand
Material Supplier: -
Material Source: -
Sample Location: B5 @ 0-5'
-
Sampled By: JE
Date Sampled: 1/20/2021
Received By: RJ
Date Received: 1/20/2021
Tested By: FS
Date Tested: 1/21/2021
Reviewed By: RJ
Date Reviewed: 1/22/2021

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



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	134.0	**@ Optimum Moisture, %**	7.5
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

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

Atterberg Limits:

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

Classification:

Unified Soils Classification: ______

AASHTO Soils Classification: ______

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client: Dr. Horton
Project Number: 2593-CR
Project Location: TR: 31513

Tested/ Checked By: GP Lab No Corona
Date Tested: 1/21/2022
Sample Source: B5 @ 0-5'
Sample Description:

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

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	784.0
B	Weight of ring (gm)	364.1
C	Net weight of sample (gm)	**419.9**
D	Wet Density, lb / ft3 (C*0.3016)	**126.6**
E	Dry Density, lb / ft3 (D/1.F)	**117.0**

SATURATION DETERMINATION

F	Moisture Content, %	8.2
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
I	% Saturation	**50.4**

READINGS

DATE	TIME	READING	
1/21/2022	12:40P	0.4270	Initial
1/21/2022		0.4270	10 min/Dry
1/22/2022		0.4270	Final

FINAL MOISTURE

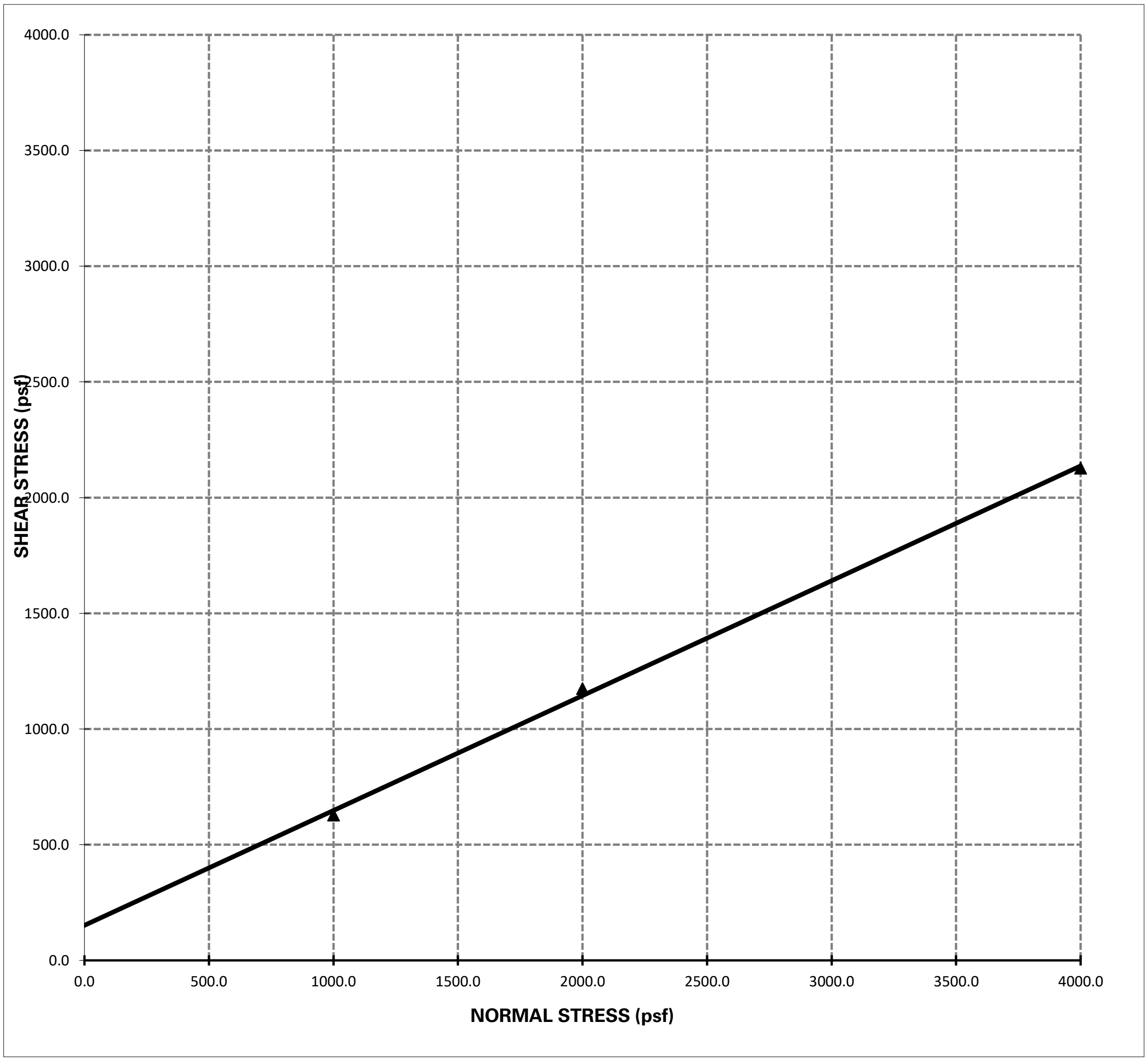
Final Weight of wet sample & tare	% Moisture
799.7	**11.9**

EXPANSION INDEX = 0

GEOTEK

DIRECT SHEAR TEST

Project Name:	Dr Horton	***Sample Location:***	S2 @ 0-5 feet
Project Number:	2593-CR	***Date Tested:***	2/1/2021



Shear Strength: **Φ = 26 °** **C = 152 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 0.01 in/min.

GEOTEK

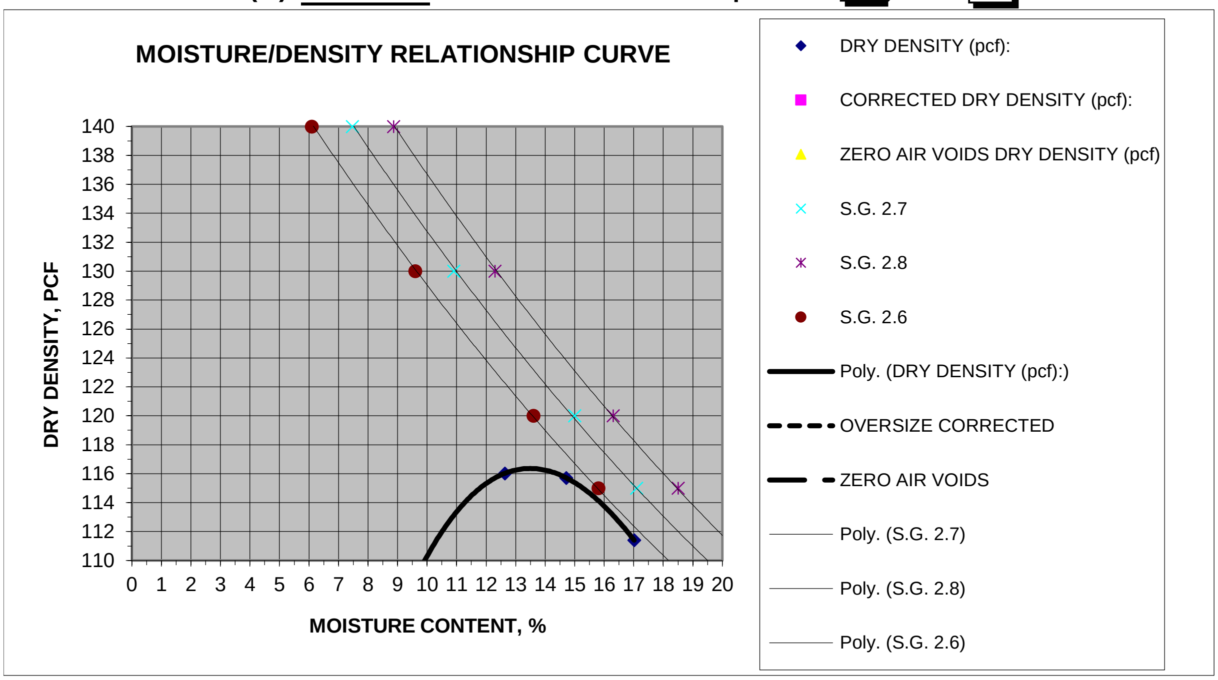
MOISTURE/DENSITY RELATIONSHIP

Client: Dr. Horton
Project: TR: 31513
Location: Hemet
Material Type: Brown Fine Sand/ Sandy Silt
Material Supplier: -
Material Source: -
Sample Location: S2 @ 0-5'
-
Sampled By: JE
Received By: RJ
Tested By: FS
Reviewed By: RJ

Job No.: 2593-CR
Lab No.: Corona

Date Sampled: 1/20/2021
Date Received: 1/20/2021
Date Tested: 1/21/2021
Date Reviewed: 1/22/2021

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



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	116.5	**@ Optimum Moisture, %**	13.5
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

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

Atterberg Limits:

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

Classification:

Unified Soils Classification: __________
AASHTO Soils Classification: __________

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client:	Dr. Horton	***Tested/ Checked By:***	FS Lab No Corona
Project Number:	2593-CR	***Date Tested:***	1/22/2021
Project Location:	TR: 31513	***Sample Source:***	S2 @ 0-5'
		Sample Description:	Clayey Sand

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

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	732.5
B	Weight of ring (gm)	362.2
C	Net weight of sample (gm)	**370.3**
D	Wet Density, lb / ft3 (C*0.3016)	**111.7**
E	Dry Density, lb / ft3 (D/1.F)	**98.7**

SATURATION DETERMINATION

F	Moisture Content, %	13.1
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
I	% Saturation	**50.1**

READINGS

DATE	TIME	READING	
1/22/2021	10am	0.6530	Initial
1/22/2021		0.6590	10 min/Dry
1/23/2021		0.7410	Final

FINAL MOISTURE

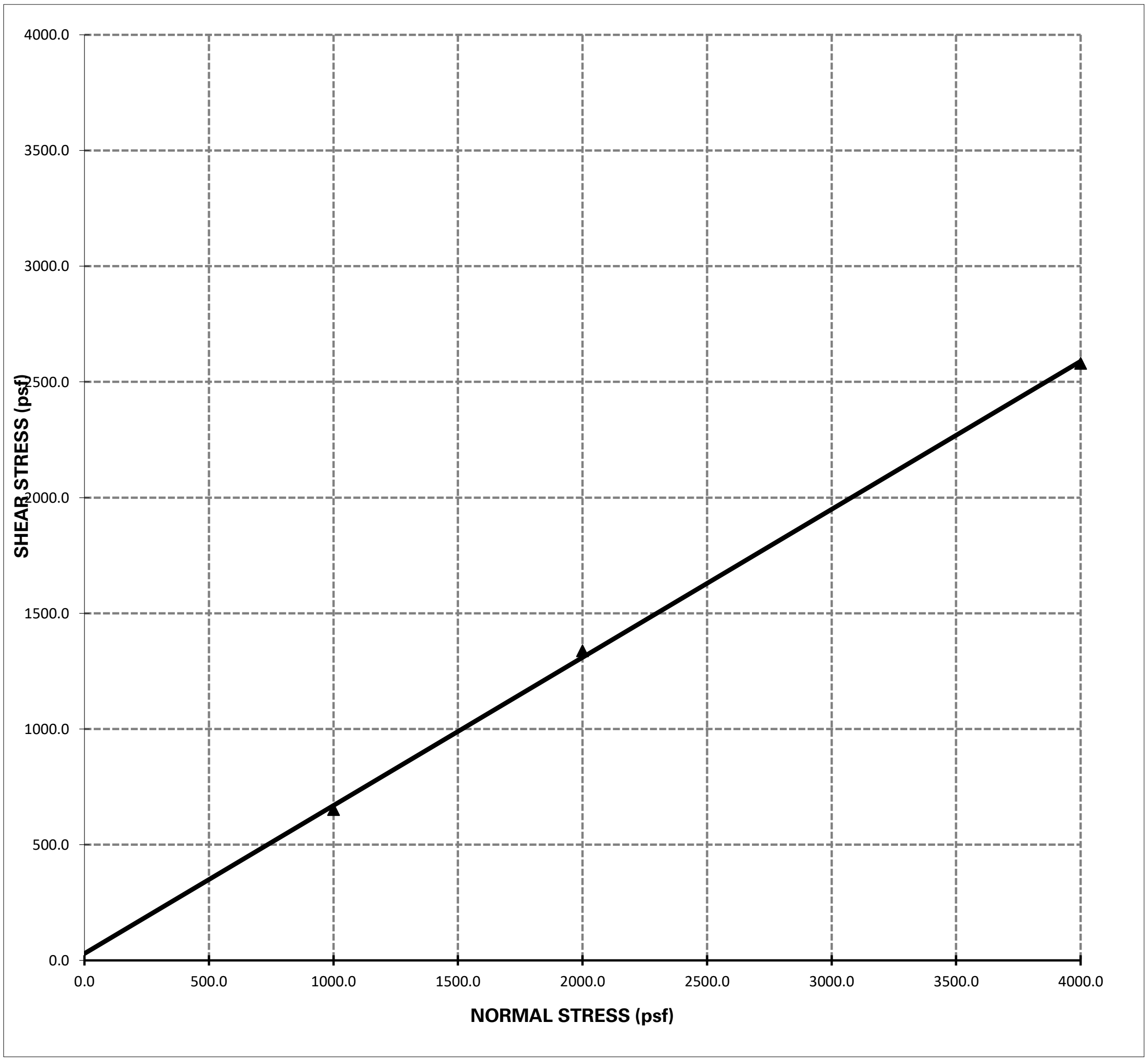
Final Weight of wet sample & tare	% Moisture
782.1	**26.5**

EXPANSION INDEX = 82

GEOTEK

DIRECT SHEAR TEST

Project Name:	Dr Horton	***Sample Location:***	S3 @ 0-3 feet
Project Number:	2593-CR	***Date Tested:***	2/3/2021



Shear Strength: **Φ = 33 °** **C = 30 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.

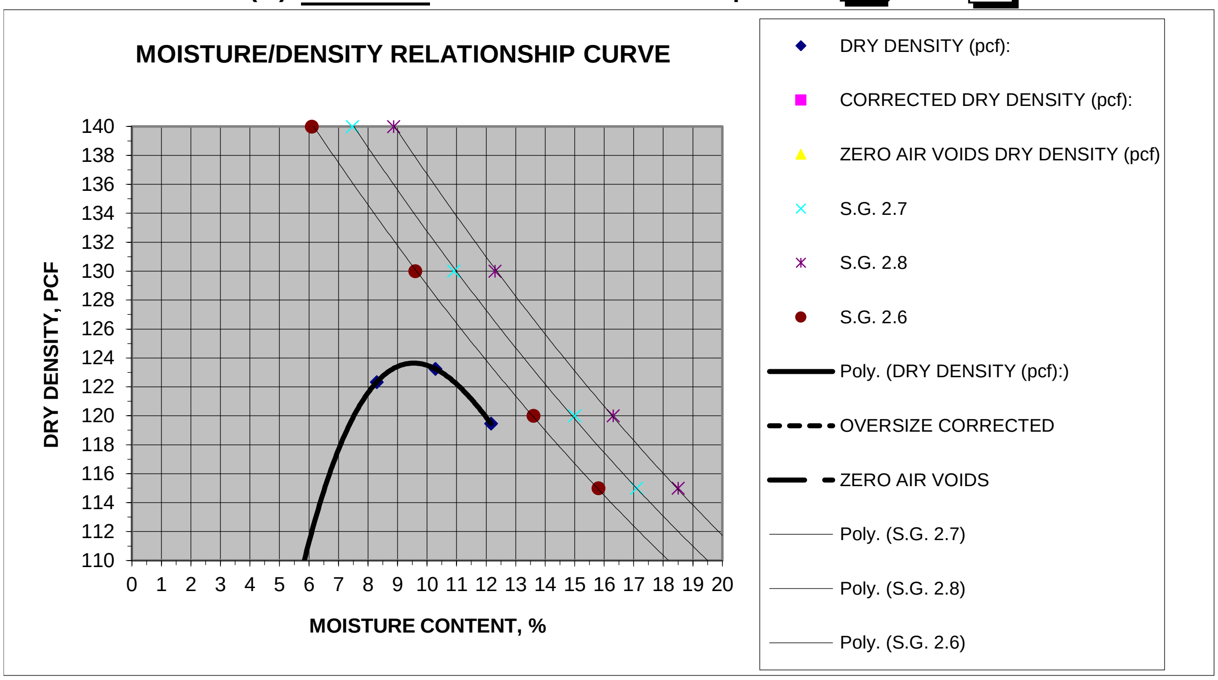
3 - The tests were run at a shear rate of 0.01 in/min.

GEOTEK

MOISTURE/DENSITY RELATIONSHIP

Client: Dr. Horton
Job No.: 2593-CR
Project: TR: 31513
Lab No.: Corona
Location: Hemet
Material Type: Brown Silty Sand
Material Supplier: -
Material Source: -
Sample Location: S3 @ 0-3'
-
Sampled By: JE
Date Sampled: 1/20/2021
Received By: RJ
Date Received: 1/20/2021
Tested By: FS
Date Tested: 1/21/2021
Reviewed By: RJ
Date Reviewed: 1/22/2021

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



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	123.0	**@ Optimum Moisture, %**	9.5
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

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

Atterberg Limits:

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

Classification:

Unified Soils Classification: ____________

AASHTO Soils Classification: ____________

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client:	Dr. Horton	***Tested/ Checked By:***	FS Lab No Corona
Project Number:	2593-CR	***Date Tested:***	1/22/2021
Project Location:	TR: 31513	***Sample Source:***	S3 @ 0-3'
		Sample Description:	Silty Sand

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

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	760.8
B	Weight of ring (gm)	366.6
C	Net weight of sample (gm)	**394.2**
D	Wet Density, lb / ft3 (C*0.3016)	**118.9**
E	Dry Density, lb / ft3 (D/1.F)	**107.6**

SATURATION DETERMINATION

F	Moisture Content, %	10.5
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20 °C, (pcf)	**62.4**
I	% Saturation	**50.1**

READINGS

DATE	TIME	READING	
1/22/2021	10am	0.6640	Initial
1/22/2021		0.6640	10 min/Dry
1/23/2021		0.6640	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
760.1	**10.3**

EXPANSION INDEX = 0

Results Only Soil Testing
for
TR31513

January 25, 2021

Prepared for:
Anna Scott
GeoTek, Inc.
1548 North Maple Street
Corona, CA 92280
ascott@geotekusa.com

Project X Job#: S210121B
Client Job or PO#: 2593-CR

Respectfully Submitted,

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

REGISTERED PROFESSIONAL ENGINEER
EDUARDO HERNANDEZ
No. M37102
MECHANICAL
STATE OF CALIFORNIA

Soil Analysis Lab Results

Client: GeoTek, Inc.
Job Name: TR31513
Client Job Number: 2593-CR
Project X Job Number: S210121B
January 25, 2021

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM D4972	ASTM G200	SM 4500-S2-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity**		**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^{--}	**Phosphate** PO_4^{3-}
						As Rec'd	**Minimum**												
	(ft)	**(mg/kg)**	**(wt%)**	**(mg/kg)**	**(wt%)**	**(Ohm-cm)**	**(Ohm-cm)**		**(mV)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**	**(mg/kg)**
S1	0-1	17.2	0.0017	9.5	0.0010	140,700	20,770	7.6	189	<0.01	8.7	12.1	ND	33.8	1.8	15.3	62.7	1.5	9.9
S2	0-1	271.8	0.0272	102.6	0.0103	18,760	2,144	9.0	138	<0.01	5.6	77.5	0.03	353.9	0.1	11.5	43.9	6.5	23.1
S3	0-1	14.2	0.0014	6.1	0.0006	67,000	11,390	7.4	172	<0.01	13.4	5.2	0.03	18.1	4.8	12.5	42.0	1.7	7.4
S4	0-1	13.1	0.0013	7.1	0.0007	281,400	10,720	8.3	183	<0.01	7.9	11.0	0.01	26.0	3.2	14.7	45.0	0.9	12.8
S5	0-1	41.8	0.0042	45.4	0.0045	127,300	5,360	7.4	191	<0.01	20.7	13.3	0.07	27.7	10.6	15.0	65.3	0.8	0.1
S6	0-1	6.3	0.0006	8.2	0.0008	167,500	10,720	8.1	182	<0.01	5.4	16.4	0.08	13.7	2.8	14.1	52.3	0.4	0.8
S7	0-1	9.7	0.0010	4.1	0.0004	80,400	24,120	7.0	181	<0.01	2.5	23.3	0.08	26.3	2.4	15.1	47.9	0.4	17.4
S8	0-1	16.6	0.0017	8.5	0.0009	>737,000	13,400	7.0	188	<0.01	15.7	25.7	0.05	27.6	6.2	13.2	36.0	1.0	1.4
S9	0-1	11.6	0.0012	7.4	0.0007	67,000	12,060	6.6	203	<0.01	48.9	11.9	0.08	18.4	8.9	11.8	36.6	1.1	3.4
S10	0-1	9.4	0.0009	5.9	0.0006	234,500	18,760	7.0	189	<0.01	9.5	15.0	0.07	11.4	13.3	14.0	40.6	0.9	6.6
S11	0-1	72.5	0.0072	93.6	0.0094	>737,000	2,747	8.3	208	<0.01	98.3	39.5	0.02	113.4	2.1	15.5	33.8	1.3	2.8
S12	0-1	6.7	0.0007	3.1	0.0003	>737,000	20,100	7.3	202	<0.01	9.8	14.0	0.02	14.0	8.0	10.3	22.0	1.0	2.6
S13	0-1	7.3	0.0007	3.6	0.0004	127,300	43,550	7.2	188	<0.01	3.9	4.8	0.06	6.8	3.6	9.5	109.7	1.1	0.0
S14	0-1	38.4	0.0038	35.8	0.0036	>737,000	5,360	7.8	195	<0.01	5.0	24.0	0.05	51.1	0.9	9.7	40.2	1.8	3.9
S15	0-1	8.2	0.0008	3.3	0.0003	100,500	23,450	7.0	198	<0.01	11.5	11.7	ND	11.4	12.6	7.8	28.4	0.9	2.6

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

Project X
Corrosion Engineering
Corrosion Control - Soil, Water, and Metallurgy Lab

Lab Request Sheet Chain of Custody
Phone: (213) 928-7213 · Fax (951) 226-1720 · www.projectxcorrosion.com
Ship Samples To: 29970 Technology Dr, Suite 105F, Murrieta, CA 92563

Project X Job Number S210121 B Geotek 2593-CR TR 31513 15 Full

IMPORTANT: Please complete Project and Sample Identification Data as you would like it to appear in report & include this form with samples.

Company Name:	GEOTEK Inc.	**Contact Name:**		**Phone No. :**	
Mailing Address:	1548 North Maple St., Corona, CA 92280	**Contact Email:**	ascott@geotekusa.com		
Accounting Contact:		**Invoice Email:**	ascott@geotekusa.com		
Project Name:	TR 31513	**Client Project No:**	2593-CR		
P.O. #:					

	5 Day Normal	3 Day RUSH 75% mark-up	2 Day RUSH 100% mark-up
Turn Around Time:	X		

Results By: ☐ Phone ☐ Fax ☒ Email ☐ Mail ☐ Overnight Mail (charges apply)

Date & Received by : 1/21/20

SPECIAL INSTRUCTIONS:

ANALYSIS REQUESTED (Please circle)

	Default Method
CORROSION SERIES	Min. Resistivity, Sulfate, Chloride, Sulfide, Redox, pH, Ammonia, Nitrate.
Soil Resistivity	Caltrans CTM64 / AASHTO T2888 / ASTM G187
pH	Caltrans CTM64 / AASHTO T 289 / ASTM G 51
Sulfate	Caltrans CTM41 / AASHTO T 290 / ASTM D516
Chloride	Caltrans CTM42 / AASHTO T 291 / ASTM D512B
Redox Potential	SM 2580B
BiCarbonate	SM 2320B
Alkalinity	SM 2520B
Acidity	SM 2510B
Nitrate	SM 4500-NO3 / Hach 835
Ammonia	SM 4500-NH3 / Hach 830
Sulfide	SM 4500-S2 / SM 4500-S2
Moisture Content	ASTM D2216
Soil Corrosivity Evaluation Report	
Metallurgical Analysis	

	#	SAMPLE ID - BORE #	DESCRIPTION	DEPTH (ft)	DATE COLLECTED	CORROSION SERIES	Soil Resistivity	pH	Sulfate	Chloride	Redox Potential	BiCarbonate	Alkalinity	Acidity	Nitrate	Ammonia	Sulfide	Moisture Content	Soil Corrosivity Evaluation Report	Metallurgical Analysis	NOTES
133	1	S1		0-1		X															
134	2	S2		0-1		X															
135	3	S3		0-1		X															
136	4	S4		0-1		X															
137	5	S5		0-1		X															
138	6	S6		0-1		X															
139	7	S7		0-1		X															
140	8	S8		0-1		X															
141	9	S9		0-1		X															
142	10	S10		0-1		X															
143	11	S11		0-1		X															
144	12	S12		0-1		X															
145	13	S13		0-1		X															
146	14	S14		0-1		X															
147		S15		0-1		X															

APPENDIX C

INFILTRATION TEST DATA (GeoTek, 2021)

Updated Geotechnical Evaluation
Proposed Single-Family Residential Development
Tract No. 31513
Hemet, Riverside County, California
Project No. 2593-CR

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Percolation Test Data Sheet

Project: Tentative Tract Map No. 31513 | Job No.: 2593-CR
Test Hole No.: I-1 | Date Excavated: 1/12/21
Depth of Test Hole (ft): 5 | Soil Description: SM
Percolation Test By: DW | Date: 1/13/21 | 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 AM 7:35 AM	25	25	20.00	5.40	14.60	1.71
7:35 AM 8:00 AM	25	50	20.00	6.00	14.00	1.79
8:00 AM 8:10 AM	10	60	20.00	15.60	4.40	2.27
8:11 AM 8:21 AM	10	71	20.00	15.70	4.30	2.33
8:21 AM 8:31 AM	10	81	20.00	15.90	4.10	2.44
8:32 AM 8:42 AM	10	92	20.00	15.90	4.10	2.44
8:42 AM 8:52 AM	10	102	20.00	16.00	4.00	2.50
8:53 AM 9:03 AM	10	113	20.00	16.00	4.00	2.50

Design Percolation Rate: **min/inch**

Percolation Test Data Sheet

Project: Tentative Tract Map No. 31513 Job No.: 2593-CR
Test Hole No.: I-2 Date Excavated: 1/12/21
Depth of Test Hole (ft): 5 Soil Description: SM
Percolation Test By: DW Date: 1/13/21 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 AM 7:37 AM	25	25	20.00	6.70	13.30	1.88
7:37 AM 8:02 AM	25	50	20.00	8.10	11.90	2.10
8:02 AM 8:12 AM	10	60	20.00	16.50	3.50	2.86
8:12 AM 8:22 AM	10	70	20.00	16.50	3.50	2.86
8:23 AM 8:33 AM	10	81	20.00	16.60	3.40	2.94
8:33 AM 8:43 AM	10	91	20.00	16.80	3.20	3.13
8:43 AM 8:53 AM	10	101	20.00	16.80	3.20	3.13
8:54 AM 9:04 AM	10	112	20.00	16.80	3.20	3.13

Design Percolation Rate: **min/inch**

Percolation Test Data Sheet

Project:	Tentative Tract Map No. 31513			Job No.:	2593-CR
Test Hole No.:	I-3			Date Excavated:	1/12/21
Depth of Test Hole (ft):	5	Soil Description:	SM		
Percolation Test By:	JE	Date:	1/13/21	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)
7:31 AM 7:56 AM	25	25	20.00	19.75	0.25	100.00
7:57 AM 8:22 AM	25	51	20.00	19.75	0.25	100.00
8:23 AM 8:53 AM	30	82	20.00	19.75	0.25	120.00
8:54 AM 9:24 AM	30	113	20.00	0.00	20.00	1.50
9:25 AM 9:55 AM	30	144	20.00	0.00	20.00	1.50
9:56 AM 10:26 AM	30	175	20.00	0.00	20.00	1.50
10:27 AM 10:57 AM	30	206	20.00	0.00	20.00	1.50
10:58 AM 11:28 AM	30	237	20.00	0.00	20.00	1.50
11:29 AM 11:59 AM	30	268	20.00	0.00	20.00	1.50
12:00 PM 12:30 PM	30	299	20.00	0.00	20.00	1.50
12:31 PM 1:01 PM	30	330	20.00	0.00	20.00	1.50
1:02 PM 1:32 PM	30	361	20.00	0.00	20.00	1.50
1:33 PM 2:03 PM	30	392	20.00	0.00	20.00	1.50

Design Percolation Rate: **min/inch**

Percolation Test Data Sheet

Project: Tentative Tract Map No. 31513 Job No.: 2593-CR
Test Hole No.: I-4 Date Excavated: 1/12/21
Depth of Test Hole (ft): 5 Soil Description: SM
Percolation Test By: JE Date: 1/13/21 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)
7:29 AM 7:54 AM	25	25	20.00	20.00	0.00	#DIV/0!
7:55 AM 8:20 AM	25	51	20.00	20.00	0.00	#DIV/0!
8:21 AM 8:51 AM	30	82	20.00	20.00	0.00	#DIV/0!
8:52 AM 9:22 AM	30	113	20.00	20.00	0.00	#DIV/0!
9:23 AM 9:53 AM	30	144	20.00	20.00	0.00	#DIV/0!
9:54 AM 10:24 AM	30	175	20.00	20.00	0.00	#DIV/0!
10:25 AM 10:55 AM	30	206	20.00	20.00	0.00	#DIV/0!
10:56 AM 11:26 AM	30	237	20.00	20.00	0.00	#DIV/0!
11:27 AM 11:57 AM	30	268	20.00	20.00	0.00	#DIV/0!
11:58 AM 12:28 PM	30	299	20.00	20.00	0.00	#DIV/0!
12:29 PM 12:59 PM	30	330	20.00	20.00	0.00	#DIV/0!
1:00 PM 1:30 PM	30	361	20.00	20.00	0.00	#DIV/0!
1:31 PM 2:01 PM	30	392	20.00	20.00	0.00	#DIV/0!

Design Percolation Rate: **min/inch**

Percolation Test Data Sheet

Project: Tentative Tract Map No. 31513 | Job No.: 2593-CR

Test Hole No.: I-5 | Date Excavated: 1/12/21

Depth of Test Hole (ft): 5 | Soil Description: SM

Percolation Test By: JE | Date: 1/13/21 | 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)
7:37 AM 8:02 AM	25	25	20.00	8.00	12.00	2.08
8:03 AM 8:28 AM	25	51	20.00	9.75	10.25	2.44
8:29 AM 8:39 AM	10	62	20.00	16.00	4.00	2.50
8:40 AM 8:50 AM	10	73	20.00	16.00	4.00	2.50
8:50 AM 9:00 AM	10	83	20.00	16.25	3.75	2.67
9:01 AM 9:11 AM	10	94	20.00	16.50	3.50	2.86
9:11 AM 9:21 AM	10	104	20.00	16.50	3.50	2.86
9:21 AM 9:31 AM	10	114	20.00	16.75	3.25	3.08

Design Percolation Rate: **min/inch**

Percolation Test Data Sheet

Project:	Tentative Tract Map No. 31513			Job No.:	2593-CR
Test Hole No.:	I-6			Date Excavated:	1/12/21
Depth of Test Hole (ft):	5	Soil Description:	SM		
Percolation Test By:	JE	Date:	1/13/21	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)
7:35 AM 8:00 AM	25	25	20.00	19.00	1.00	25.00
8:01 AM 8:26 AM	25	51	20.00	19.00	1.00	25.00
8:27 AM 8:57 AM	30	82	20.00	19.00	1.00	30.00
8:58 AM 9:28 AM	30	113	20.00	19.00	1.00	30.00
9:29 AM 9:59 AM	30	144	20.00	19.25	0.75	40.00
10:00 AM 10:30 AM	30	175	20.00	19.25	0.75	40.00
10:31 AM 11:01 AM	30	206	20.00	19.50	0.50	60.00
11:02 AM 11:32 AM	30	237	20.00	19.50	0.50	60.00
11:33 AM 12:03 PM	30	268	20.00	19.50	0.50	60.00
12:04 PM 12:34 PM	30	299	20.00	19.50	0.50	60.00
12:35 PM 1:05 PM	30	330	20.00	19.50	0.50	60.00
1:06 PM 1:36 PM	30	361	20.00	19.50	0.50	60.00
1:37 PM 2:07 PM	30	392	20.00	19.50	0.50	60.00

Design Percolation Rate: **min/inch**

Client:	**D.R. Horton**
Project:	**Hemet**
Project No:	**2593-CR**
Date:	**1/13/2021**

Boring No. **I-1**

Percolation Rate (Porchet Method)

Time Interval, Δt =	10
Final Depth to Water, D_F =	44
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
$H_F = D_T - D_F$ =	16
$\Delta H = \Delta D = H_O - H_F$ =	4
Havg = $(H_O+H_F)/2$ =	18

I_t = 2.40 **Inches per Hour**

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Client:	**D.R. Horton**
Project:	**Hemet**
Project No:	**2593-CR**
Date:	**1/13/2021**

Boring No. **I-2**

Percolation Rate (Porchet Method)

Time Interval, Δt =	10
Final Depth to Water, D_F =	43.2
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	40
Total Test Hole Depth, D_T =	60

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

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	16.8
$\Delta H = \Delta D = H_O - H_F$ =	3.2
Havg = $(H_O+H_F)/2$ =	18.4

I_t = 1.88 **Inches per Hour**

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Client:	**D.R. Horton**
Project:	**Hemet**
Project No:	**2593-CR**
Date:	**1/13/2021**

Boring No. **I-3**

Percolation Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	40
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
$H_F = D_T - D_F$ =	20
$\Delta H = \Delta D = H_O - H_F$ =	0
$Havg = (H_O+H_F)/2$ =	20

I_t = 0.00 **Inches per Hour**

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Client:	**D.R. Horton**
Project:	**Hemet**
Project No:	**2593-CR**
Date:	**1/13/2021**

Boring No. **I-4**

Percolation Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	40
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	40
Total Test Hole Depth, D_T =	60

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

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	20
$\Delta H = \Delta D = H_O - H_F$ =	0
Havg = $(H_O+H_F)/2$ =	20

I_t = 0.00 **Inches per Hour**

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Client:	**D.R. Horton**
Project:	**Hemet**
Project No:	**2593-CR**
Date:	**1/13/2021**

Boring No. **I-5**

Percolation Rate (Porchet Method)

Time Interval, Δt =	10
Final Depth to Water, D_F =	43.25
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	40
Total Test Hole Depth, D_T =	60

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

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	16.75
$\Delta H = \Delta D = H_O - H_F$ =	3.25
Havg = $(H_O+H_F)/2$ =	18.375

I_t = 1.91 **Inches per Hour**

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Client:	**D.R. Horton**
Project:	**Hemet**
Project No:	**2593-CR**
Date:	**1/13/2021**

Boring No. **I-6**

Percolation Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	40.5
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	40
Total Test Hole Depth, D_T =	60

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

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	19.5
$\Delta H = \Delta D = H_O - H_F$ =	0.5
Havg = $(H_O+H_F)/2$ =	19.75

I_t = 0.09 **Inches per Hour**

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APPENDIX D

LIQUEFACTION ANALYSIS (GeoTek, 2021)

Updated Geotechnical Evaluation

Proposed Single-Family Residential Development

Tract No. 31513

Hemet, Riverside County, California

Project No. 2593-CR

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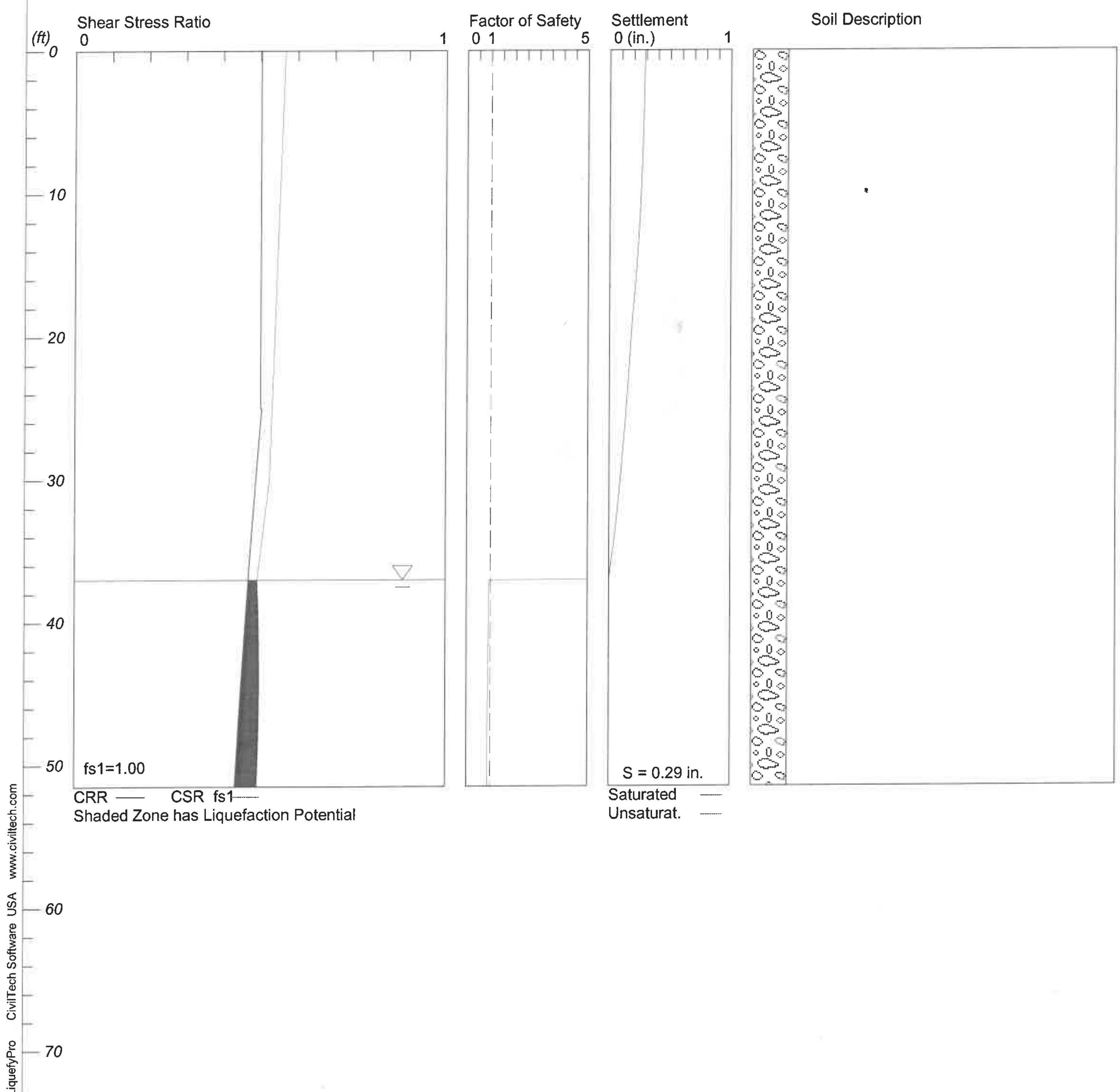
LIQUEFACTION ANALYSIS

Tract 31513

Hole No.=B-5 ***Water Depth=37 ft***

Magnitude=7.49
Acceleration=0.87g



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LIQUEFACTION ANALYSIS SUMMARY

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Font: Courier New, Regular, Size 8 is recommended for this report.
Licensed to , 3/12/2021 11:03:35 AM

Input File Name: G:\Projects\2551 to 2600\2593CR DR Horton LA Holding Company Tract No. 31513 Hemet\Geotechnical Investigation\liq input.liq
Title: Tract 31513
Subtitle: Tres Cerritos West

Surface Elev.=
Hole No.=B-5
Depth of Hole= 51.50 ft
Water Table during Earthquake= 37.00 ft
Water Table during In-Situ Testing= 60.00 ft
Max. Acceleration= 0.87 g
Earthquake Magnitude= 7.49

Input Data:
Surface Elev.=
Hole No.=B-5
Depth of Hole=51.50 ft
Water Table during Earthquake= 37.00 ft
Water Table during In-Situ Testing= 60.00 ft
Max. Acceleration=0.87 g
Earthquake Magnitude=7.49
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= 1.15
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1
   Plot one CSR curve (fs1=User)
10. Use Curve Smoothing: Yes*
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	gamma pcf	Fines %
0.00	43.00	130.00	15.00
7.00	65.00	135.00	19.00
15.00	65.00	128.00	19.00
20.00	64.00	127.00	19.00
25.00	69.00	130.00	15.00
30.00	76.00	130.00	34.00
35.00	50.00	130.00	34.00
40.00	82.00	130.00	34.00
45.00	41.00	130.00	34.00
50.00	52.00	130.00	15.00

Output Results:

Settlement of Saturated Sands=0.00 in.
Settlement of Unsaturated Sands=0.29 in.
Total Settlement of Saturated and Unsaturated Sands=0.29 in.
Differential Settlement=0.143 to 0.189 in.

Depth ft	CRRm	CSRfs	F.S.	S_sat. in.	S_dry in.	S_all in.
0.00	0.50	0.57	5.00	0.00	0.29	0.29
1.00	0.50	0.56	5.00	0.00	0.29	0.29
2.00	0.50	0.56	5.00	0.00	0.28	0.28
3.00	0.50	0.56	5.00	0.00	0.28	0.28
4.00	0.50	0.56	5.00	0.00	0.28	0.28
5.00	0.50	0.56	5.00	0.00	0.28	0.28
6.00	0.50	0.56	5.00	0.00	0.27	0.27
7.00	0.50	0.56	5.00	0.00	0.27	0.27
8.00	0.50	0.55	5.00	0.00	0.27	0.27
9.00	0.50	0.55	5.00	0.00	0.26	0.26
10.00	0.50	0.55	5.00	0.00	0.26	0.26
11.00	0.50	0.55	5.00	0.00	0.25	0.25
12.00	0.50	0.55	5.00	0.00	0.25	0.25
13.00	0.50	0.55	5.00	0.00	0.24	0.24
14.00	0.50	0.55	5.00	0.00	0.23	0.23
15.00	0.50	0.55	5.00	0.00	0.23	0.23
16.00	0.50	0.54	5.00	0.00	0.22	0.22
17.00	0.50	0.54	5.00	0.00	0.21	0.21
18.00	0.50	0.54	5.00	0.00	0.20	0.20
19.00	0.50	0.54	5.00	0.00	0.19	0.19
20.00	0.50	0.54	5.00	0.00	0.18	0.18
21.00	0.50	0.54	5.00	0.00	0.17	0.17
22.00	0.50	0.54	5.00	0.00	0.17	0.17

23.00	0.50	0.54	5.00	0.00	0.16	0.16
24.00	0.50	0.53	5.00	0.00	0.15	0.15
25.00	0.50	0.53	5.00	0.00	0.14	0.14
26.00	0.50	0.53	5.00	0.00	0.13	0.13
27.00	0.50	0.53	5.00	0.00	0.12	0.12
28.00	0.50	0.53	5.00	0.00	0.11	0.11
29.00	0.49	0.53	5.00	0.00	0.10	0.10
30.00	0.49	0.53	5.00	0.00	0.09	0.09
31.00	0.49	0.52	5.00	0.00	0.08	0.08
32.00	0.48	0.52	5.00	0.00	0.07	0.07
33.00	0.48	0.51	5.00	0.00	0.06	0.06
34.00	0.48	0.51	5.00	0.00	0.04	0.04
35.00	0.47	0.50	5.00	0.00	0.03	0.03
36.00	0.47	0.50	5.00	0.00	0.01	0.01
37.00	0.47	0.49	5.00	0.00	0.00	0.00
38.00	0.47	0.49	0.94*	0.00	0.00	0.00
39.00	0.46	0.50	0.93*	0.00	0.00	0.00
40.00	0.46	0.50	0.93*	0.00	0.00	0.00
41.00	0.46	0.50	0.92*	0.00	0.00	0.00
42.00	0.46	0.50	0.91*	0.00	0.00	0.00
43.00	0.45	0.50	0.91*	0.00	0.00	0.00
44.00	0.45	0.50	0.90*	0.00	0.00	0.00
45.00	0.45	0.50	0.90*	0.00	0.00	0.00
46.00	0.45	0.50	0.90*	0.00	0.00	0.00
47.00	0.44	0.50	0.89*	0.00	0.00	0.00
48.00	0.44	0.50	0.89*	0.00	0.00	0.00
49.00	0.44	0.50	0.89*	0.00	0.00	0.00
50.00	0.44	0.50	0.88*	0.00	0.00	0.00
51.00	0.44	0.49	0.88*	0.00	0.00	0.00

* F.S.<1, Liquefaction Potential Zone
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

1 atm (atmosphere) = 1 tsf (ton/ft2)

CRRm	Cyclic resistance ratio from soils
CSRsf	Cyclic stress ratio induced by a given earthquake (with user request factor of safety)
F.S.	Factor of Safety against liquefaction, F.S.=CRRm/CSRsf
S_sat	Settlement from saturated sands
S_dry	Settlement from Unsaturated Sands
S_all	Total Settlement from Saturated and Unsaturated Sands
NoLiq	No-Liquefy Soils

APPENDIX E

SOIL CORROSIVITY EVALUATION
(Project X Corrosion Engineering, Inc., 2021)

Updated Geotechnical Evaluation
Proposed Single-Family Residential Development
Tract No. 31513
Hemet, Riverside County, California
Project No. 2593-CR

Soil Corrosivity Evaluation Report for TR31513

October 29, 2021

Prepared for:
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Project X Job #: S210121B
Client Job or PO #: 2593-CR

Contents

1 Executive Summary

A corrosion evaluation of the soils at located within Tract 31513 in the city of Hemet, Riverside County, California was performed to provide corrosion control recommendations for general construction materials. The site is located at **situated north and adjacent to Rose Road and east of the intersection of Old Warren Road, Hemet, CA** (33°45'18.5"N 117°01'47.8"W). Fifteen (15) samples were tested to a depth of 1.0 ft.

Every material has its weakness. Aluminum alloys, galvanized/zinc coatings, and copper alloys do not survive well in very alkaline or very acidic pH environments. Copper and brasses do not survive well in high nitrate or ammonia environments. Steels and irons do not survive well in low soil resistivity and high chloride environments. High chloride environments can even overcome and attack steel encased in normally protective concrete. Concrete does not survive well in high sulfate environments. And nothing survives well in high sulfide and low redox potential environments with corrosive bacteria. This is why Project X tests for these 8 factors to determine a soil's corrosivity towards various construction materials. **Depending solely on soil resistivity or Caltrans corrosion guidelines (which concentrate on concrete/steel highways), will over-simplify descriptions as corrosive or non-corrosive. This approach will not detect these other factors attacking other metals because it is possible to have bad levels of corrosive ions and still have greater than 1,100 ohm-cm soil resistivity. We have observed this fact on thousands of soil samples tested in our laboratory.**

It should not be forgotten that import soil should also be tested for all factors to avoid making your site more corrosive than it was to begin with.

The recommendations outlined herein are not a substitute for any design documents previously prepared for the purpose of construction and apply only to the depth of samples collected.

Soil samples were tested for minimum resistivity, pH, chlorides, sulfates, ammonia, nitrates, sulfides and redox.

As-Received soil resistivities ranged between 18,760 ohm-cm and 281,400.0 ohm-cm. This data would be similar to a Wenner 4 pin test in the field and used in the design of a cathodic protection or grounding bed system. This resistivity can change seasonally depending on the weather and moisture in the ground. This reading alone can be misleading because condensation or minor water leaks will occur underground along pipe surfaces creating a saturated soil environment in the trench on infrastructure surfaces. This is why minimum or saturated soil resistivity measurements are more important than as-received resistivities.

Saturated soil resistivities ranged between 2,144 ohm-cm to 43,550 ohm-cm. The worst of these values is considered to be moderately corrosive to general metals.

PH levels ranged between 6.6 to 9.0 pH. PH levels were determined to be at levels not detrimental to copper or aluminum alloys. The pH of these samples can allow corrosion of steel and iron in moist environments.

Chlorides ranged between 3 mg/kg to 103 mg/kg. Chloride levels in these samples are low and may cause insignificant corrosion of metals.

Sulfates ranged between 6 mg/kg to 272 mg/kg. Sulfate levels in these samples are negligible for corrosion of cement. Any type of cement can be used that does not contain encased metal.

Ammonia ranged between 4.8 mg/kg to 77.5 mg/kg. Nitrates ranged between 2.5 mg/kg to 98.3 mg/kg. Concentrations of these elements were high enough to cause accelerated corrosion of copper and copper alloys such as brass.

Sulfides presence was determined to be negative. REDOX ranged between + 138 mV to + 208 mV. The probability of corrosive bacteria was determined to be low due to the sulfide and positive REDOX levels determined in these samples.

2 Corrosion Control Recommendations

The following recommendations are based upon the results of soil testing.

2.1 Cement

The highest reading for sulfates was 272 mg/kg or 0.0272 percent by weight.

Per ACI 318-14, Table 19.3.1.1, sulfate levels in these samples categorized as S0 and are negligible for corrosion of metals and cement. Per ACI 318-14 Table 19.3.2.1 any type of cement not containing steel or other metal can be used.

2.2 Steel Reinforced Cement/ Cement Mortar Lined & Coated (CML&C)

Chlorides in soil can overcome the corrosion inhibiting property of cement for steel, as it can also break through passivated surfaces of aluminum and stainless steels.[1,2] The highest concentration of chlorides was 103 mg/kg.

Chloride levels in these samples are not significantly corrosive to metals not in tension. Standard cement cover may be used in these soils.

Though soils at some locations are significantly corrosive to various metals, per ACI 318-14 Chapter 19 Table 19.3.1.1, all slabs on this site exposure categories and class for **Corrosion Protection of Reinforcement (C) would be considered C1** as Concrete exposed to moisture [mud/rain] (slab sides and bottom) but not to an external source of chlorides. Though there are chlorides in the soil, ACI 318's definition of "external source of chlorides" consists of deicing chemicals, salt, brackish water, seawater, or spray from these sources. The chloride levels in seawater are typically over 19,000 mg/L or 19,000 ppm.

When concrete is tested for water-soluble chloride ion content, the tests should be made at an age of 28 to 42 days. The limits in Per ACI 318-14 Table 5.3.2.1 are to be applied to chlorides contributed from the concrete ingredients, not those from the environment surrounding the concrete.[3]

[1] Design Manual 303: Cement Cylinder Pipe. Ameron. p.65
[2] *Chapter 19, Table 1904.2.2(1), 2012 International Building Code*
[3] ACI 381-14., BUILDING CODE REQUIREMENTS FOR STRUCTURAL CONCRETE (ACI 318-14) AND COMMENTARY (ACI 318R-14)

2.3 Stainless Steel Pipe/Conduit/Fittings

Stainless steels derive their corrosion resistance from their chromium content and oxide layer which needs oxygen to regenerate if damaged. Thus stainless steel is not good for deep soil applications where oxygen levels are extremely low. Stainless steels should not be installed deeper than a plant root zone. Stainless steels typically have the same nobility as copper on the galvanic series and can be connected to copper. If stainless steel must be used, it must be backfilled with soil having greater than 10,000 ohm-cm resistivity and excellent drainage. 304 Stainless steel will also corrode if in contact with carbon materials such as activated carbon. Stainless steel welds should be pickled.

The soil at this site has low probability for anaerobic corrosive bacteria and low chloride levels. Per Nickel Institute guidelines, 304 or 316 Stainless steels can be used in these soils.

2.4 Steel Post Tensioning Systems

The proper sealing of stressing holes is of utmost importance in PT Systems. Cut off excess strand 1/2" to 3/4" back in the hole. Coat or paint exposed anchorage, grippers, and stub of strands with "Rust-o-leum" or equal. After tendons have been coated, the cement contractor shall dry pack blockouts within ten (10) days. A non-shrink, non-metallic, non-porous moisture-insensitive grout (Master EMACO S 488 or equivalent), or epoxy grout shall be used for this purpose. If an encapsulated post-tension system is used, regular non-shrink grout can be used.

Due to the low chloride concentrations measured on samples obtained from this site, post-tensioned slabs should be protected in accordance with soil considered normal (non-corrosive).[4,5] Addition of grease caps to the cut strand at live end anchors can deter construction defect accusations but are not needed.

2.5 Steel Piles

Steel piles are most susceptible to corrosion in disturbed soil where oxygen is available. Further, a dissimilar environment corrosion cell would exist between the steel embedded in cement, such as pile caps and the steel in the soil. In the cell, the steel in the soil is the anode (corroding metal), and the steel in cement is the cathode (protected metal). This cell can be minimized by coating the part of the steel piles that will be embedded in cement to prevent contact with cement and reinforcing steel.

Piles driven into soils without disturbing soils will avoid oxygen introduction and low corrosion rates unless there is a probability for corrosive anaerobic bacteria. Galvanized steel's zinc coating can provide significant protection for driven piles. In corrosive soils in which normal zinc coatings are not enough, the life of piles can be extended by increasing zinc coating thickness, using sacrificial metal, or providing a combination of epoxy coatings and cathodic protection. Corrosion has been observed to be extremely localized even at and below underground water tables. Pit depths of this magnitude do not have an appreciable effect on the strength or useful life of piling structures because the reduction in pile cross section is not

[4] *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils, PTI DC10.5-12,Table 4.1, pg 16*
[5] *Specification for Unbonded Single Strand Tendons. Post-tensioning Institute (PTI), Phoenix, AZ, 2000.*

significant.[6] Pitting is of more importance to pipes transporting liquids or gases which should not be leaked into the ground.

The following recommendations are recommended to achieve desired life. We defer to structural engineers to use our estimated corrosion rates and to choose from the corrosion control options listed below.

1) Sacrificial metal by use of thicker piles per non-disturbed soil corrosion rates, or
2) Galvanized steel piles per non-disturbed soil corrosion rates, or
3) Combination of galvanized and sacrificial metal per non-disturbed soil corrosion rates, or
4) For no loss of metal, coat entire pile with abrasion resistant epoxy coating such as 3M Scotchkote 323, or PowercreteDD, or equivalent, or
5) Use high yield steel which will corrode at the same rate as mild steel but have greater yield strength and thus be able to suffer more material loss than mild steel.

2.5.1 Expected Corrosion Rate of Steel and Zinc in disturbed soil

In general, the corrosion rate of metals in soil depends on the electrical resistivity, the elemental composition, and the oxygen content of the soil. Soils can vary greatly from one acre to the next, especially at earthquake faults. The better a soil is for farming; the easier it will be for corrosion to take place. Expansive soils will also be considered disturbed simply because of their nature from dry to wet seasons.

In Melvin Romanoff's NBS Circular 579, the corrosion rates of carbon steels and various metals was studied over long term periods. Various metals were placed in various soil types to gather corrosion rate data of all metals in all soil types. Samples were collected and material loss measured over the course of 20 years in some sites. The following corrosion rates were estimated by comparing the worst results of soils tested with similar soils in Romanoff's studies and Highway Research Board's publications.[7] The corrosion rate of zinc in disturbed soils is determined per Romanoff studies and King Nomograph.[8]

Expected Corrosion Rate for Steel = 0.69 mils/year for one sided attack

Expected Corrosion Rate for Zinc = 0.08 mils/year for one sided attack.

Note: 1 mil = 0.001 inch

In undisturbed soils, a corrosion rate of 1 mil/year for steel is expected with little change in the corrosion rate of zinc due to it's low nobility in the galvanic series.

Per CTM 643: Years to perforation of corrugated galvanized steel culverts

- 75.7 Years to Perforation for a 18 gage metal culvert
- 98.4 Years to Perforation for a 16 gage metal culvert
- 121.2 Years to Perforation for a 14 gage metal culvert

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[6] Melvin Romanoff, Corrosion of Steel Pilings in Soils, National Bureau of Standards Monograph 58, pg 20.

[7] Field test for Estimating Service Life of Corrugated Metal Culverts, J.L. Beaton, Proc. Highway Research Board, Vol 41, P. 255, 1962

[8] King, R.A. 1977, Corrosion Nomograph, TRRC Supplementary Report, British Corrosion Journal

- 166.6 Years to Perforation for a 12 gage metal culvert
- 212.0 Years to Perforation for a 10 gage metal culvert
- 257.5 Years to Perforation for a 8 gage metal culvert

2.5.2 ***Expected Corrosion Rate of Steel and Zinc in Undisturbed soil***

Expected Corrosion Rate for Steel = 0.69 mils/year for one sided attack

Expected Corrosion Rate for Zinc = 0.08 mils/year for one sided attack.

Note: 1 mil = 0.001 inch

2.6 Steel Storage tanks

Underground fuel tanks must be constructed and protected in accordance with California Underground Storage Tank Regulations, CCR, Title 23, Division 3, Chapter 16. Metals should be protected with cathodic protection or isolated from backfill material with an epoxy coating.

2.7 Steel Pipelines

Though a site may not be corrosive in nature at the time of construction, **installation of corrosion test stations and electrical continuity joint bonding should be performed during construction** so that future corrosion inspections can be performed. If steel pipes with gasket joints or other possibly non-conductive type joints are installed, their joints should be bonded across by welding or pin brazing a #8 AWG copper strand bond cable. Electrical continuity is necessary for corrosion inspections and for cathodic protection.

Corrosion test stations should be installed every 1,000 feet of pipeline.

Test stations shall have two #8 HMWPE copper strand wire test leads welded or pin brazed to the underground pipe, brought up into the test station hand hole and marked CTS. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

At isolation joints and pipe casings, 4 wire test stations shall be installed using #8 HMWPE copper strand wire test leads. Use different color wires to distinguish which wires are bonded to one side of isolation joint or to casing. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

Prevent dissimilar metal corrosion cells per NACE SP0286:

1) Electrically isolate dissimilar metal connections
2) Electrically isolate dissimilar coatings (Epoxy vs CML&C) segments connections
3) Electrically isolate river crossing segments
4) Electrically isolate freeway crossing segments
5) Electrically isolate old existing pipelines from new pipelines
6) Electrically isolate aboveground and underground pipe segments with flange isolation joint kits per NACE SP0286 to avoid galvanic corrosion cells. **These are especially important for fire risers.**

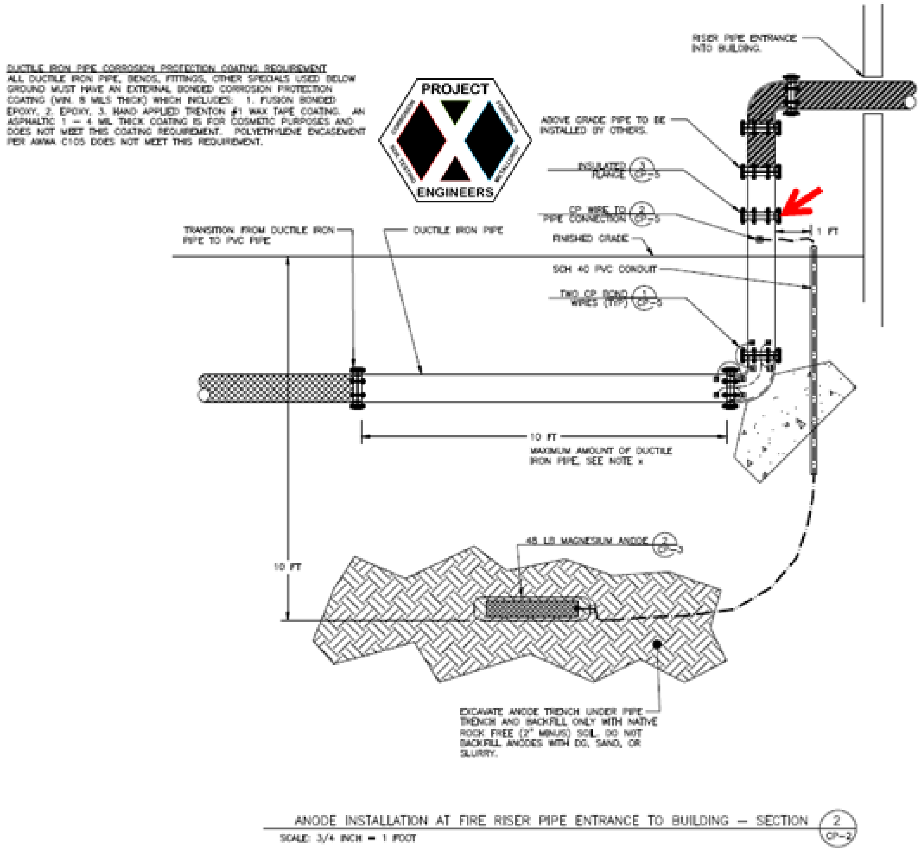

Figure 1- Fire Riser Detail: Install Isolation joint at red arrow

The bare steel surfaces, the corrosivity at this site is mildly corrosive to steel. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, or
2) Tape coating system per AWWA C214, or
3) Wax tape per AWWA C217, or
4) Coal tar enamel per AWWA C203, or
5) Fusion bonded epoxy per AWWA C213, or
6) For bare steel surfaces, such as welded pipe joints, apply 3 inch thick field coating of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide. (For CML&C pipes, CML&C factory applied 3/4 inch thick coating is equivalent and needs no extra thickness added.)

It is critical for the life of the pipe that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.8 Steel Fittings

The corrosivity at this site is mildly corrosive to steel. The corrosion control options for this site can be one of the following:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, or
2) Tape coating system per AWWA C214, or
3) Wax tape per AWWA C217, or
4) Coal tar enamel per AWWA C203, or
5) Fusion bonded epoxy per AWWA C213
6) Use powder coated steel with minimum 60 micron (2-3 mil) thick coating[9], or
7) Galvanized steel, or
8) Apply standard concrete cover of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.9 Ductile Iron (DI) & Cast Iron Fittings

AWWA C105 developed a 10 point system to classify sites as aggressive or non-aggressive to ductile iron materials. The 10-point system does not, and was never intended to, quantify the corrosivity of a soil. It is a tool used to distinguish nonaggressive from aggressive soils relative to iron pipe. Soils <10 points are considered nonaggressive to iron pipe, whereas soils ≥10 points are considered aggressive. A 15 and a 20 point soil are both considered aggressive to iron pipe, however, because of the nature of the soil parameters measured, the 20 point soil may not necessarily be more aggressive than the 15 point soil. The criterion is based upon soil resistivities, soil drainage, pH, sulfide presence, and reduction-oxidation (REDOX) potential. The soil samples tested for this site resulted in a score of 4 out of 25.5. A score greater or equal

[9] Manish Kumar Bhadu, Akshya Kumar Guin, Veena Singh, Shyam K. Choudhary, "Corrosion Study of Powder-Coated Galvanised Steel", International Scholarly Research Notices, vol. 2013, Article ID 464710, 9 pages, 2013

to 10 points classifies soils as aggressive to iron materials. The black coating on iron pipes is purely for aesthetic purposes and should not be relied upon for corrosion protection.[10]

The corrosivity at this site is mildly corrosive to iron. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, or
2) Tape coating system per AWWA C214, or
3) Wax tape per AWWA C217, or
4) Coal tar enamel per AWWA C203, or
5) Fusion bonded epoxy per AWWA C213
6) Apply standard concrete cover of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.10 Ductile Iron & Cast Iron Pipe

AWWA C105 developed a 10 point system to classify sites as aggressive or non-aggressive to ductile iron materials. The 10-point system does not, and was never intended to, quantify the corrosivity of a soil. It is a tool used to distinguish nonaggressive from aggressive soils relative to iron pipe. Soils <10 points are considered nonaggressive to iron pipe, whereas soils ≥10 points are considered aggressive. A 15 and a 20 point soil are both considered aggressive to iron pipe, however, because of the nature of the soil parameters measured, the 20 point soil may not necessarily be more aggressive than the 15 point soil. The criterion is based upon soil resistivities, soil drainage, pH, sulfide presence, and reduction-oxidation (REDOX) potential. The soil samples tested for this site resulted in a score of 4 out of 25.5. A score greater or equal to 10 points classifies soils as aggressive to iron materials. The black coating on iron pipes is purely for aesthetic purposes and should not be relied upon for corrosion protection.[11]

Though a site may not be corrosive in nature at the time of construction, <u>**installation of corrosion test stations and electrical continuity joint bonding should be performed during construction**</u> so that future corrosion inspections can be performed. If steel pipes with gasket joints or other possibly non-conductive type joints are installed, their joints should be bonded across by welding or pin brazing a #8 AWG copper strand bond cable. Electrical continuity is necessary for corrosion inspections and for cathodic protection. **If using thermite, perform one**

[10] https://www.dipra.org/ductile-iron-pipe-resources/frequently-asked-questions/corrosion-control

[11] https://www.dipra.org/ductile-iron-pipe-resources/frequently-asked-questions/corrosion-control

test bond using a half-charge then pressure test to confirm excess heat and pinholes were not created.

Pea gravel is used by plumbers to lay pipes and establish slopes. If the gravel has more than 200 ppm chlorides or is not tested, a 25 mil plastic should be placed between the gravel and pipe to avoid corrosion.

Corrosion test stations should be installed every 1,000 feet of pipeline.

Test stations shall have two #8 HMWPE copper strand wire test leads welded or pin brazed to the underground pipe, brought up into the test station hand hole and marked CTS. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

At isolation joints and pipe casings, 4 wire test stations shall be installed using #8 HMWPE copper strand wire test leads. Use different color wires to distinguish which wires are bonded to one side of isolation joint or to casing. Wires should be brought into test station hand hole at finished grade with 12 inches of wire coiled within test station.

Prevent dissimilar metal corrosion cells per NACE SP0286:

1) Electrically isolate dissimilar metal connections
2) Electrically isolate dissimilar coatings (Epoxy vs CML&C) segments connections
3) Electrically isolate river crossing segments
4) Electrically isolate freeway crossing segments
5) Electrically isolate old existing pipelines from new pipelines
6) Electrically isolate aboveground and underground pipe segments with flange isolation joint kits per NACE SP0286**. These are especially important for fire risers.**

The corrosivity at this site is mildly corrosive to iron. The corrosion control options for this site are as follows:

1) Apply impermeable dielectric coating such as minimum 10 mil thick polyethylene, or
2) Tape coating system per AWWA C214, or
3) Wax tape per AWWA C217, or
4) Coal tar enamel per AWWA C203, or
5) Fusion bonded epoxy per AWWA C213
6) Apply standard concrete cover of Type II cement or high pH slurry that will maintain pH higher than 12. Cement is both a corrosion inhibitor and a coating for ferrous metals. Cement naturally holds a pH of 12 or higher for many years if not exposed to high levels of carbon dioxide.

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11 Copper Materials

Copper is an amphoteric material which is susceptible to corrosion at very high and very low pH. It is one of the most noble metals used in construction thus typically making it a cathode when connected to dissimilar metals. Copper's nobility can change with temperature, similar to the phenomenon in zinc. When zinc is at room temperature, it is less noble than steel and can provide cathodic protection to steel. But when zinc is at a temperature above 140F such as in a water heater, it becomes more noble than the steel and the steel becomes the sacrificial anode. This is why zinc is not used in steel water heaters or boilers. Cold copper has one native potential, but when heated it develops a more electronegative electro-potential aka open circuit potential. Thus hot and cold copper pipes should be electrically isolated from each other to avoid creation of a thermo-galvanic corrosion cell.

2.11.1 Copper Pipes

The lowest pH for this area was measured to be 6.6. Copper is greatly affected by pH, ammonia and nitrate concentrations[12]. The highest nitrate concentration was 98.3 mg/kg and the highest ammonia concentration was 7.7 mg/kg at this site.

These soils were determined to be corrosive to copper and copper alloys such as brass.

Aboveground, underground, cold water and hot water pipes should be electrically isolated from each other by use of dielectric unions and plastic in-wall pipe supports per NACE SP0286. The following are corrosion control options for underground copper water pipes.

1) Run copper pipes within PVC pipes to prevent soil contact, or
2) Cover piping with a 20 mil epoxy coating, or 8-mil polyethylene sleeve, or encase in double 4-mil thick polyethylene sleeves free of scratches and defects then backfill with clean sand with 2 inch minimum cover above and below tubing. Backfill should have a pH between 6 and 8 with electrical resistivity greater than 2,000 ohm-cm
3) Cover copper pipes with minimum 8 mil polyethylene sleeve or incase in double 4-mil thick polyethylene sleeves over a suitable primer and apply cathodic protection per NACE SP0169

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

[12] Corrosion Data Handbook, Table 6, Corrosion Resistance of copper alloys to various environments, 1995

2.11.2 Brass Fittings

Brass fittings should be electrically isolated from dissimilar metals by use of dielectric unions or isolation joint kits per NACE SP0286. These soils were determined to be corrosive to copper and copper alloys such as brass.

The following are corrosion control options for underground brass.

1) Prevent soil contact by use of impermeable coating system such as wax tape, or
2) Prevent soil contact by use of a 20 mil epoxy coating free of scratches and defects and backfill with clean sand with 4 inch minimum cover above and below brass. Backfill should have a pH between 6 and 8 with electrical resistivity greater than 2,000 ohm-cm, or
3) Cover brass with minimum 10 mil polyethylene sleeve over a suitable primer and apply cathodic protection per NACE SP0169

It is critical for the life of the metal that the protective wrap contains no openings or holes. Prevent damage to the protective sleeve during backfilling of the pipe trench. Penetrations of any kind within these or other protective materials generally leads to accelerated corrosion failure due to the fact that the corrosion attack is concentrated at the location of these penetrations. Cathodic protection will protect these defects. The better the coating, the less expensive a cathodic protection system will be in anode material and power requirement if needed.

2.11.3 Bare Copper Grounding Wire

It is assumed that corrosion will occur at all sides of the bare wire, thus the corrosion rate is calculated as a two sided attack determining the time it takes for the corrosion from two sides to meet at the center of the wire. The estimated life of bare copper wire for this site is the following:[13]

Size (AWG)	**Diameter (mils)**	**Est. Time to penetration (Yrs)**
14	64.1	5.5
13	72	6.2
12	80.8	7.0
11	90.7	7.8
10	101.9	8.8
9	114.4	9.9
8	128.5	11.1
7	144.3	12.4
6	162	14.0
5	181.9	15.7
4	204.3	17.6
3	229.4	19.8
2	257.6	22.2

[13] Soil-Corrosion studies 1946 and 1948: Copper Alloys, Lead, and Zinc, Melvin Romanoff, National Bureau of Standards, Research Paper RP2077, 1950

Size (AWG)	**Diameter (mils)**	**Est. Time to penetration (Yrs)**
1	289.3	24.9

If the bare copper wire is being used as a grounding wire connected to less noble metals such as galvanized steel or carbon steel, the less noble metals will provide additional cathodic protection to the copper reducing the corrosion rate of the copper.

It is recommended that a corrosion inhibiting and water-repelling coating be applied to aboveground and belowground copper-to-dissimilar metal connections to reduce risk of dissimilar corrosion. This can be wax tape, or other epoxy coating.

Tinned copper wiring or laying copper wire in conductive concrete can protect against chemical attack in soils with high nitrates, ammonia, sulfide and severely low soil electrical resistivity.

2.12 Aluminum Pipe/Conduit/Fittings

Aluminum is an amphoteric material prone to pitting corrosion in environments that are very acidic or very alkaline or high in chlorides.

Conditions at this site are safe for aluminum.

Aluminum derives its corrosion resistance from its oxide layer which needs oxygen to regenerate if damaged, similar to stainless steels. Thus aluminum is not good for deep soil applications. Since aluminum corrodes at very alkaline environments, it cannot be encased or placed against cement or mortar such as brick wall mortar up against an aluminum window frame.

Aluminum is also very low on the galvanic series scale making it most likely to become a sacrificial anode when in contact with dissimilar metals in moist environments. Avoid electrical continuity with dissimilar metals by use of insulators, dielectric unions, or isolation joints per NACE SP0286. Pooling of water at post bottoms or surfaces should be avoided by integrating good drainage.

2.13 Carbon Fiber or Graphite Materials

Carbon fiber or other graphite materials are extremely noble on the galvanic series and should always be electrically isolated from dissimilar metals. They can conduct electricity and will create corrosion cells if placed in contact within a moist environment with any metal.

2.14 Plastic and Vitrified Clay Pipe

No special precautions are required for plastic and vitrified clay piping from a corrosion viewpoint.

Protect all metallic fittings and pipe restraining joints with wax tape per AWWA C217, cement if previously recommended, or epoxy.

3 CLOSURE

In addition to soils chemistry and resistivity, another contributing influence to the corrosion of buried metallic structures is stray electrical currents. These electrical currents flowing through the earth originate from buried electrical systems, grounding of electrical systems in residences, commercial buildings, and from high voltage overhead power grids. Therefore, it is imperative that the application of protective wraps and/or coatings and electrical isolation joints be properly applied and inspected.

It is the responsibility of the builder and/or contractor to closely monitor the installation of such materials requiring protection in order to assure that the protective wraps or coatings are not damaged.

The recommendations outlined herein are in conformance with current accepted standards of practice that meet or exceed the provisions of the Uniform Building Code (UBC), the International Building Code (IBC), California Building Code (CBC), the American Cement Institute (ACI), Nickel Institute, National Association of Corrosion Engineers (NACE International), Post-Tensioning Institute Guide Specifications and State of California Department of Transportation, Standard Specifications, American Water Works Association (AWWA) and the Ductile Iron Pipe Research Association (DIPRA).

Our 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,

Ed

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4 SOIL ANALYSIS LAB RESULTS

Client: GeoTek, Inc.
Job Name: TR31513
Client Job Number: 2593-CR
Project X Job Number: S210121B
October 29, 2021

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM D4972	ASTM G200	SM 4500-S2-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd	Minimum	**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^{--}	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
S1	0-1	17.2	0.0017	9.5	0.0010	140,700	20,770	7.6	189	<0.01	8.7	12.1	ND	33.8	1.8	15.3	62.7	1.5	9.9
S2	0-1	271.8	0.0272	102.6	0.0103	18,760	2,144	9.0	138	<0.01	5.6	77.5	0.03	353.9	0.1	11.5	43.9	6.5	23.1
S3	0-1	14.2	0.0014	6.1	0.0006	67,000	11,390	7.4	172	<0.01	13.4	5.2	0.03	18.1	4.8	12.5	42.0	1.7	7.4
S4	0-1	13.1	0.0013	7.1	0.0007	281,400	10,720	8.3	183	<0.01	7.9	11.0	0.01	26.0	3.2	14.7	45.0	0.9	12.8
S5	0-1	41.8	0.0042	45.4	0.0045	127,300	5,360	7.4	191	<0.01	20.7	13.3	0.07	27.7	10.6	15.0	65.3	0.8	0.1
S6	0-1	6.3	0.0006	8.2	0.0008	167,500	10,720	8.1	182	<0.01	5.4	16.4	0.08	13.7	2.8	14.1	52.3	0.4	0.8
S7	0-1	9.7	0.0010	4.1	0.0004	80,400	24,120	7.0	181	<0.01	2.5	23.3	0.08	26.3	2.4	15.1	47.9	0.4	17.4
S8	0-1	16.6	0.0017	8.5	0.0009	>737,000	13,400	7.0	188	<0.01	15.7	25.7	0.05	27.6	6.2	13.2	36.0	1.0	1.4
S9	0-1	11.6	0.0012	7.4	0.0007	67,000	12,060	6.6	203	<0.01	48.9	11.9	0.08	18.4	8.9	11.8	36.6	1.1	3.4
S10	0-1	9.4	0.0009	5.9	0.0006	234,500	18,760	7.0	189	<0.01	9.5	15.0	0.07	11.4	13.3	14.0	40.6	0.9	6.6
S11	0-1	72.5	0.0072	93.6	0.0094	>737,000	2,747	8.3	208	<0.01	98.3	39.5	0.02	113.4	2.1	15.5	33.8	1.3	2.8
S12	0-1	6.7	0.0007	3.1	0.0003	>737,000	20,100	7.3	202	<0.01	9.8	14.0	0.02	14.0	8.0	10.3	22.0	1.0	2.6
S13	0-1	7.3	0.0007	3.6	0.0004	127,300	43,550	7.2	188	<0.01	3.9	4.8	0.06	6.8	3.6	9.5	109.7	1.1	0.0
S14	0-1	38.4	0.0038	35.8	0.0036	>737,000	5,360	7.8	195	<0.01	5.0	24.0	0.05	51.1	0.9	9.7	40.2	1.8	3.9
S15	0-1	8.2	0.0008	3.3	0.0003	100,500	23,450	7.0	198	<0.01	11.5	11.7	ND	11.4	12.6	7.8	28.4	0.9	2.6

Unk = Unknown
NT = Not Tested
ND = 0 = Not Detected
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
Chemical Analysis performed on 1:3 Soil-To-Water extract
Anions and Cations tested via Ion Chromatograph except Sulfide.

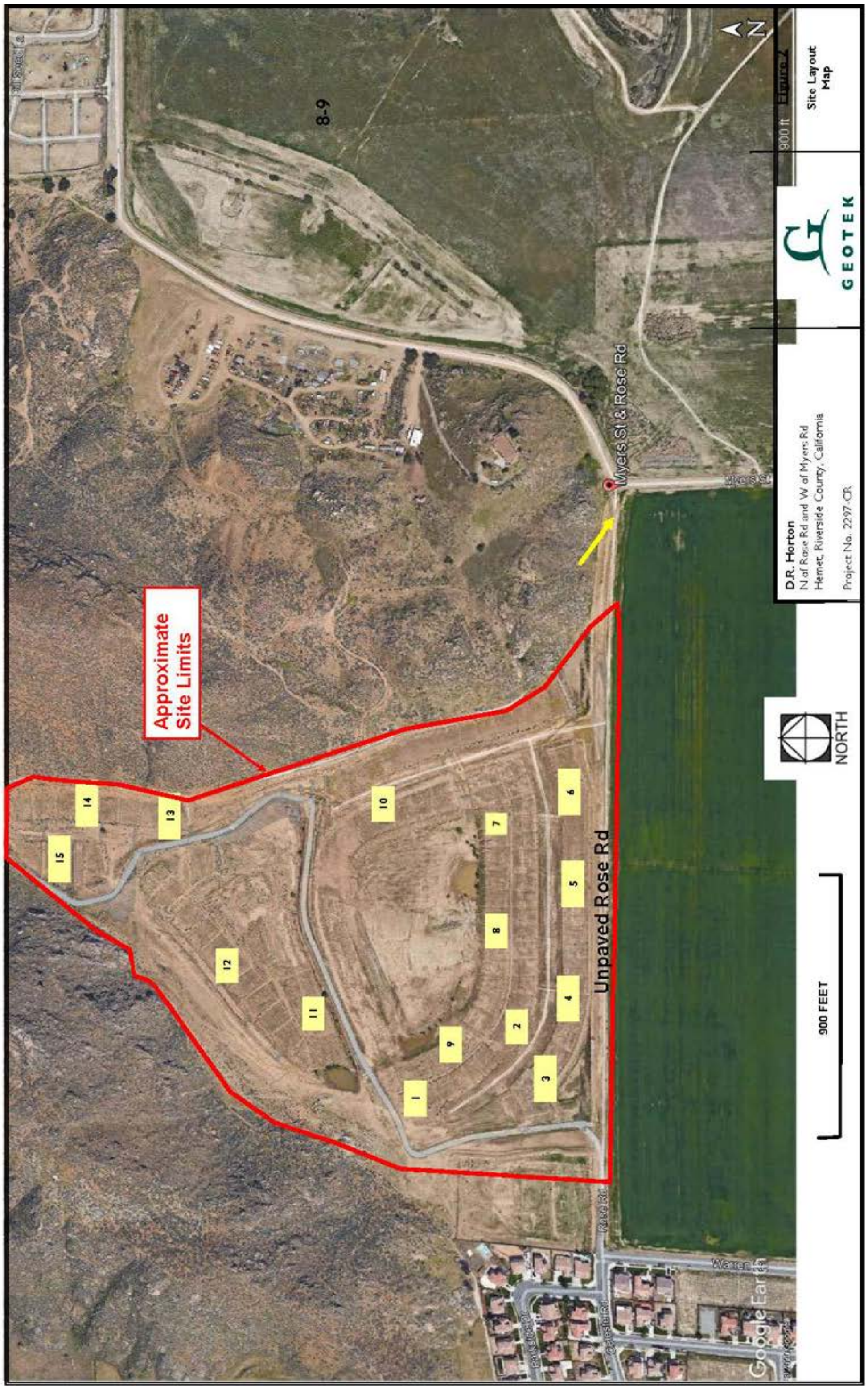


Figure 2- Soil Sample Locations, located within Tract 31513 in the city of Hemet, Riverside County, California

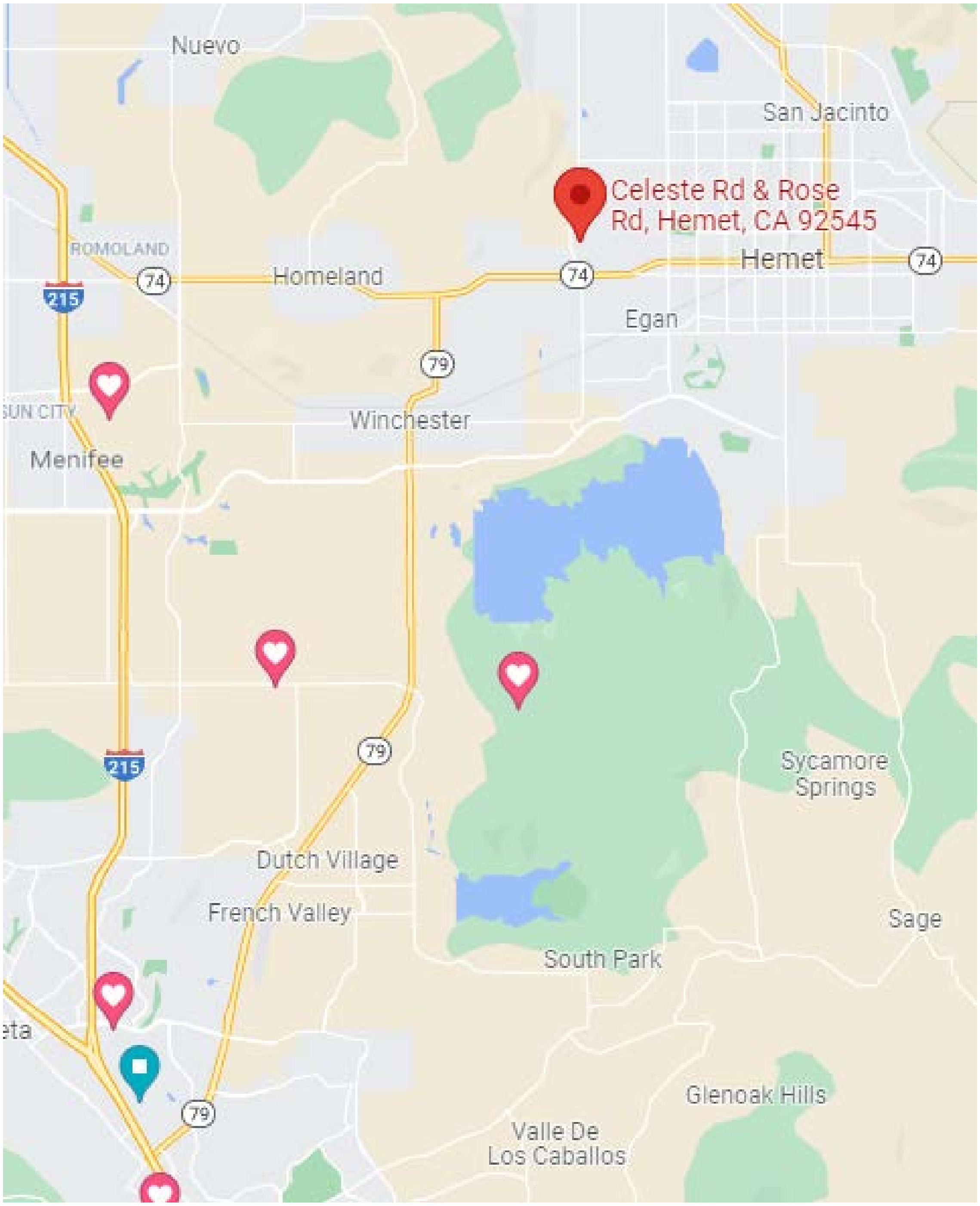


Figure 3- Vicinity Map, Rose Road & Old Warren Road, Hemet, CA (33°45'18.5"N 117°01'47.8"W)

Figure 4- Satellite View (33°45'18.5"N 117°01'47.8"W)

5 Corrosion Basics

In general, the corrosion rate of metals in soil depends on the electrical resistivity, the elemental composition, and the oxygen content of the soil. Soils can vary greatly from one acre to the next, especially at earthquake faults. The better a soil is for farming; the easier it will be for corrosion to take place. Expansive soils should be considered disturbed simply because of their nature from dry to wet seasons.

5.1 Pourbaix Diagram – In regards to a material's environment

All metals are unique and have a weakness. Some metals do not like acidic (low pH) environments. Some metals do not like alkaline (high pH) environments. Some metals don't like either high or low pH environments such as aluminum. These are called amphoteric materials. Some metals become passivated and do not corrode at high pH environments such as steel. These characteristics are documented in Marcel Pourbaix's book "Atlas of electrochemical equilibria in aqueous solutions"

In the mid 1900's, Marcel Pourbaix developed the Pourbaix diagram which describes a metal's reaction to an environment dependent on pH and voltage conditions. It describes when a metal remains passive (non-corroding) and in which conditions metals become soluble (corrode). Steels are passive in pH over 12 such as the condition when it is encased in cement. If the cement were to carbonate and its pH reduce to below 12, the cement would no longer be able to act as a corrosion inhibitor and the steel will begin to corrode when moist.

Some metals such as aluminum are amphoteric, meaning that they react with acids and bases. They can corrode in low pH and in high pH conditions. Aluminum alloys are generally passive within a pH of 4 and 8.5 but will corrode outside of those ranges. This is why aluminum cannot be embedded in cement and why brick mortar should not be laid against an aluminum window frame without a protective barrier between them.

5.2 Galvanic Series – In regards to dissimilar metal connections

All metals have a natural electrical potential. This electrical potential is measured using a high impedance voltmeter connected to the metal being tested and with the common lead connected to a copper copper-sulfate reference electrode (CSE) in water or soil. There are many types of reference electrodes. In laboratory measurements, a Standard Hydrogen Electrode (SHE) is commonly used. When different metal alloys are tested they can be ranked into an order from most noble (less corrosion), to least noble (more active corrosion). When a more noble metal is connected to a less noble metal, the less noble metal will become an anode and sacrifice itself through corrosion providing corrosion protection to the more noble metal. This hierarchy is known as the galvanic series named after Luigi Galvani whose experiments with electricity and muscles led Alessandro Volta to discover the reactions between dissimilar metals leading to the early battery. The greater the voltage difference between two metals, the faster the corrosion rate will be.

Table 1- Dissimilar Metal Corrosion Risk

	Zinc	Galvanized Steel	Aluminum	Cast Iron	Lead	Mild Steel	Tin	Copper	Stainless Steel
Zinc	None	Low	Medium	High	High	High	High	High	High
Galvanized Steel	Low	None	Medium	Medium	Medium	High	High	High	High
Aluminum	Medium	Medium	None	Medium	Medium	Medium	Medium	High	High
Cast Iron	High	Medium	Medium	None	Low	Low	Low	Medium	Medium
Lead	High	Medium	Medium	Low	None	Low	Low	Medium	Medium
Mild Steel	High	High	Medium	Low	Low	None	Low	Medium	Medium
Tin	High	High	Medium	Low	Low	Low	None	Medium	Medium
Copper	High	High	High	Medium	Medium	Medium	Medium	None	Low
Stainless Steel	High	High	High	Medium	Medium	Medium	Medium	Low	None

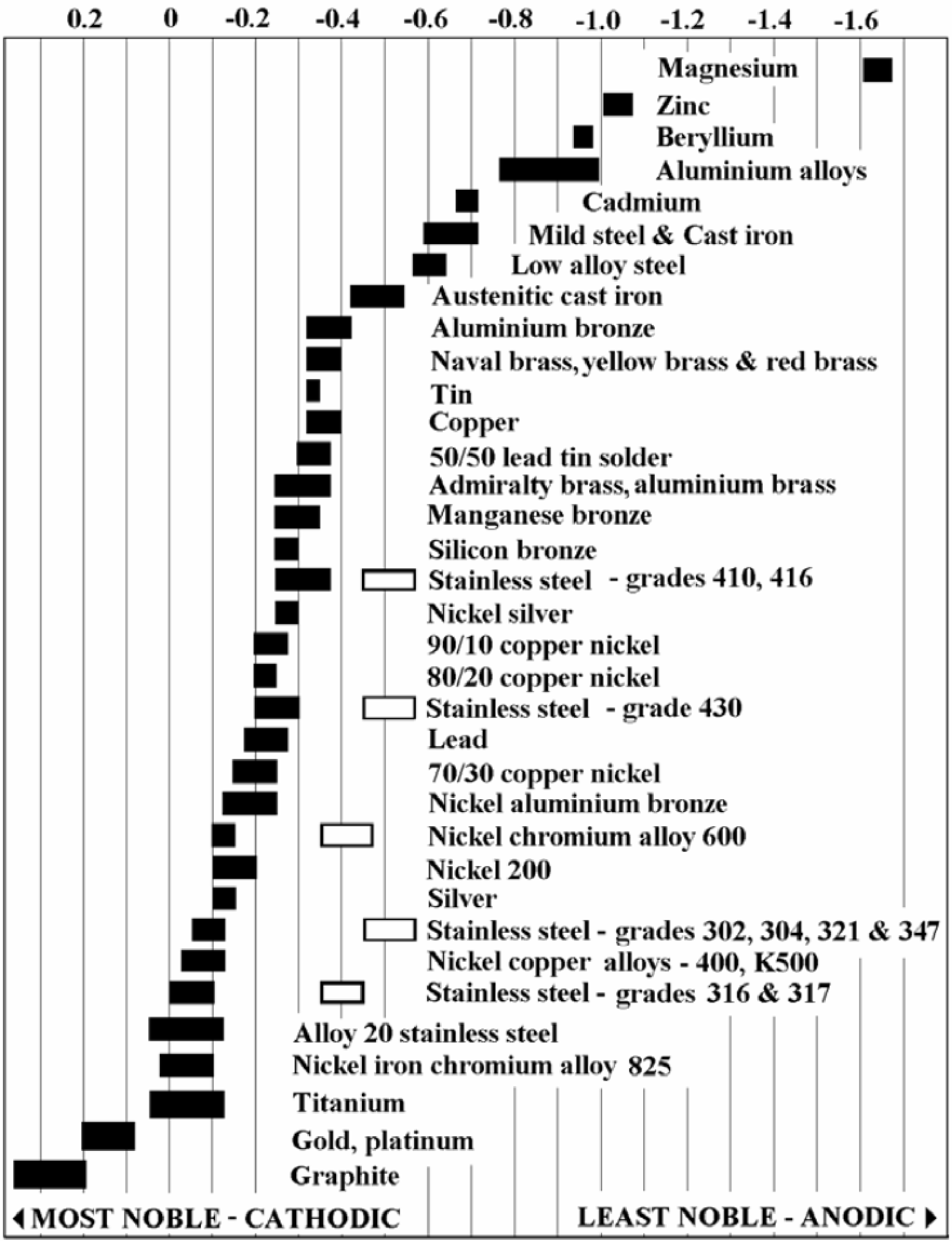
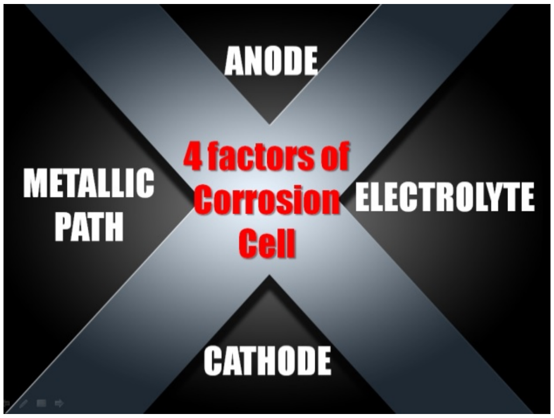
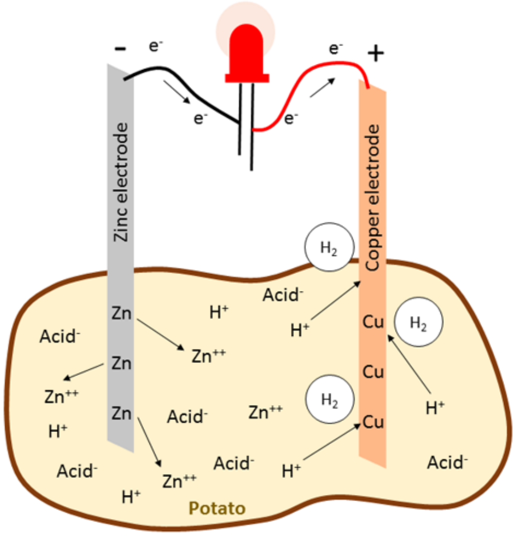


Figure 5 - Galvanic series of metals relative to CSE half cell.

5.3 Corrosion Cell



In order for corrosion to occur, four factors must be present. (1) The anode (2) the cathode (3) the electrolyte and (4) the metallic or conductive path joining the anode and the cathode. If any one of these is removed, corrosion activity will stop. This is how a simple battery produces electricity. An example of a non-metallic yet conductive material is graphite. Graphite is similar in nobility to gold. Do not connect graphite to anything in moist environments.



The anode is where the corrosion occurs, and the cathode is the corrosion free material. Sometimes the anode and cathode are different materials connected by a wire or union. Sometimes the anode and cathode are on the same pipe with one area of the pipe in a low oxygen zone while the other part of the pipe is in a high oxygen zone. A good example of this is a post in the ocean that is repeatedly splashed. Deep underwater, corrosion is minimal, but at the splash zone, the corrosion rate is greatest.

Low oxygen zones and crevices can also harbor corrosive bacteria which in moist environments will lead to corrosion. This is why pipes are laid on backfill instead of directly on native cut soil in a trench. Filling a trench slightly with backfill before installing pipe then finishing the backfill creates a uniform environment around the entire surface of the pipe.

The electrolyte is generally water, seawater, or moist soil which allows for the transfer of ions and electrical current. Pure water itself is not very conductive. It is when salts and minerals dissolve into pure water that it becomes a good conductor of electricity and chemical reactions. Metal ores are turned into metal alloys which we use in construction. They naturally want to return to their natural metal ore state but it requires energy to return to it. The corrosion cell, creates the energy needed to return a metal to its natural ore state.

The metallic or conductive path can be a wire or coupling. Examples are steel threaded into a copper joint, or an electrician grounding equipment to steel pipes inadvertently connecting electrical grid copper grounding systems to steel or iron underground pipes.

The ratio of surface area between the anode and the cathode is very important. If the anode is very large, and the cathode is very small, then the corrosion rate will be very small and the anode may live a long life. An example of this is when short copper laterals were connected to a large and long steel pipeline. The steel had plenty of surface area to spread the copper's attack, thus corrosion was not noticeable. But if the copper was the large pipe and the steel the short laterals, the steel would corrode at an amazing rate.

5.4 Design Considerations to Avoid Corrosion

The following recommendations are based upon typical observations and conclusions made by forensic engineers in construction defect lawsuits and NACE International (Corrosion Society) recommendations.

5.4.1 Testing Soil Factors (Resistivity, pH, REDOX, SO, CL, NO3, NH3)

As previously mentioned, different factors can cause corrosion. The most useful and common test for categorizing a soil's corrosivity has been the measure of soil resistivity which is typically measured in units of (ohm-cm) by corrosion engineers and geologists. Soil resistivity is the ability of soil to conduct or resist electrical currents and ion transfer. The lower the soil resistivity, the more conductive and corrosive it is. The following are "generally" accepted categories but keep in mind, the question is not "Is my soil corrosive?", the question should be, "What is my soil corrosive to?" and to answer that question, soil resistivity and chemistry must be tested. Though **soil resistivity is a good corrosivity indicator for steel materials, high chlorides or other corrosive elements do not always lower soil resistivity, thus if you don't test for chlorides and other water soluble salts, you can get an unpleasant surprise.** The largest contributing factor to a soil's electrical resistivity is its clay, mineral, metal, or sand make-up.

Table 2 - Corrosion Basics- An Introduction, NACE, 1984, pg 191

(Ohm-cm)	Corrosivity Description
0-500	Very Corrosive
500-1,000	Corrosive
1,000-2,000	Moderately Corrosive
2,000-10,000	Mildly Corrosive
Above 10,000	Progressively less corrosive

Testing a soil's pH provides information to reference the Pourbaix diagram of specific metals. Some elements such as ammonia and nitrates can create localized alkaline conditions which will greatly affect amphoteric materials such as aluminum and copper alloys.

Excess sulfates can break-down the structural integrity of cement and high concentrations of chlorides can overcome cement's corrosion inhibiting effect on encased ferrous metals and break down protective passivated surface layers on stainless steels and aluminum.

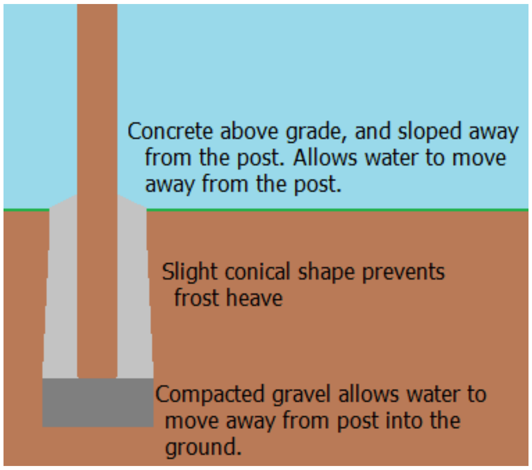
Corrosive bacteria are everywhere but can multiply significantly in anaerobic conditions with plentiful sulfates. The bacteria themselves do not eat the metal but their by-products can form corrosive sulfuric acids. The probability of corrosive bacteria is tested by measuring a soil's oxidation-reduction (REDOX) electro-potential and by testing for the presence of sulfides.

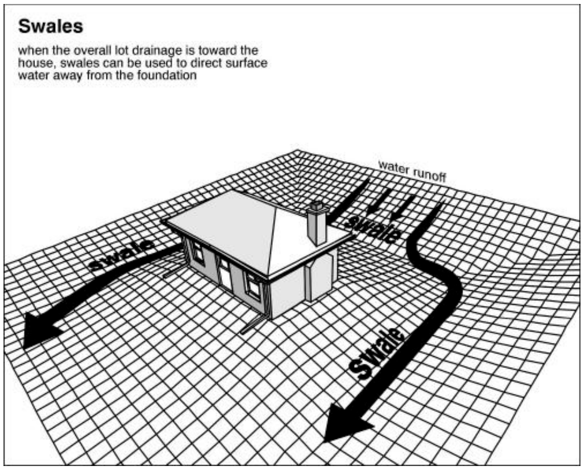
Only by testing a soil's chemistry for minimum resistivity, pH, chlorides, sulfates, sulfides, ammonia, nitrate, and redox potential can one have the information to evaluate the corrosion risk to construction materials such as steel, stainless steel, galvanized steel, iron, copper, brass, aluminum, and concrete.

5.4.2 Proper Drainage

It cannot be emphasized enough that pooled stagnant water on metals will eventually lead to corrosion. This stands for internal corrosion and external corrosion situations. In soils, providing good drainage will lower soil moisture content reducing corrosion rates. Attention to properly sealing polyethylene wraps around valves and piping will avoid water intrusion which would allow water to pool against metals. Above ground structures should not have cupped or flat surfaces that will pond water after rain or irrigation events.

Buildings typically are built on pads and have swales when constructed to drain water away from buildings directing it towards an acceptable exit point such as a driveway where it continues draining to a local storm drain. Many homeowners, landscapers and flatwork contractors appear to not be aware of this and destroy swales during remodeling. The majority of garage floor and finished grade elevations are governed by drainage during design. [14,15]





5.4.3 Avoiding Crevices

Crevices are excellent locations for oxygen differential induced corrosion cells to begin. Crevices can also harbor corrosive bacteria even in the most chemically treated waters. Crevices will also gather salts. If water's total alkalinity is low, its ability to maintain a stable pH can also become more difficult within a crevice allowing the pH to drop to acidic levels continuing a pitting process. Welds in extremely corrosive environments should be complete and well filleted without sharp edges to avoid crevices. Sharp edges should be avoided to allow uniform coating of protective epoxy. Detection of crevices in welds should be treated immediately. If pressures and loads are low, sanding and rewelding or epoxy patching can be suitable repairs. Damaged coatings can usually be repaired with Direct to Metal paints. **Scratches and crevice corrosion are like infections, they should not be left to fester or the infection will spread making things worse.**

[14] https://www.fencedaddy.com/blogs/tips-and-tricks/132606467-how-to-repair-a-broken-fence-post

[15] http://southdownstudio.co.uk/problme-drainage-maison.html

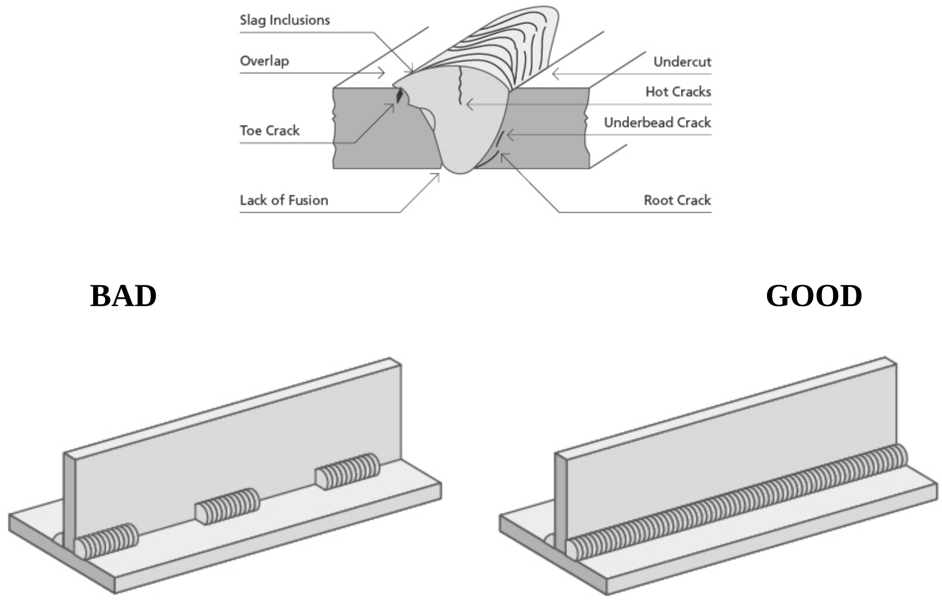


Figure 6 Defects which form weld crevices[16]

5.4.4 Coatings and Cathodic Protection

When faced with a corrosive environment, the best defense against corrosion is removing the electrolyte from the corrosion cell by applying coatings to separate the metal from the soil. During construction and installation, there is always some scratch or damage made to a coating. NACE training recommends that coatings be used as a first line of defense and that sacrificial or impressed current cathodic protection is used as a 2^{nd} line of defense to protect the scratched areas. Use of a good coating dramatically reduces the amount of anodes a CP system would need. If CP is not installed as a 2^{nd} line of defense in an extremely corrosive environment, the small scratched zones will suffer accelerated corrosion. CP details such as anode installation instructions must be designed by corrosion engineers or vessel manufacturers on a per project basis because it depends on electrolyte resistivity, surface area of infrastructure to be protected, and system geometry.

There are two types of cathodic protection systems, a Galvanic Anode Cathodic Protection (GACP) system and an Impressed Current Cathodic Protection (ICCP) system. A Galvanic Anode Cathodic Protection (GACP) system is simpler to install and maintain than an Impressed Current Cathodic Protection (ICCP) system. To protect the metals, they must all be electrically continuous to each other. In a GACP system, sacrificial zinc or magnesium anodes are then buried at locations per the CP design and connected by wire to a structure at various points in system. At the connection points, a wire connecting to the structure and the wire from the anode are joined in a Cathodic Protection Test Station hand hole which looks similar in size and shape to an irrigation valve pull box. By coating the underground structures, one can reduce the number of anodes needed to provide cathodic protection by 80% in many instances.

An ICCP system requires a power source, a rectifier, significantly more trenching, and more expensive type anodes. These systems are typically specified when bare metal is requiring protection

[16] http://www.daroproducts.co.uk/makes-good-weld/

in severely corrosive environments in which galvanic anodes do not provide enough power to polarize infrastructure to -850 mV structure-to-soil potential or be able to create a 100 mV potential shift as required by NACE SP169 to control corrosion. In severely corrosive environments, a GACP system simply may not last a required lifetime due to the high rate of consumption of the sacrificial anodes. ICCP system rectifiers must be inspected and adjusted quarterly or at a minimum bi-annually per NACE recommendations. Different anode installations may be possible but for large sites, anodes are placed evenly throughout the site and all anode wires must be trenched to the rectifier. For a large site, it may be beneficial to use two or more rectifiers to reduce wire lengths or trenching.

To simplify, a GACP system can be installed and practically forgotten with minor trenching because the anodes can be installed very close to the structures. An ICCP system must be inspected annually and anode wires run back to the rectifier which itself connects to the pile system. If any type of trenching or development is expected to occur at the site during the life of the site, it is a good idea to inspect the anode connections once a year to make sure wires are not cut and that the infrastructure is still being provided adequate protection. A common situation that occurs with ICCP systems is that a contractor accidently cuts the wires during construction then reconnects them incorrectly, turning the once cathode, into a sacrificing anode.

Design of a cathodic protection system protecting against soil side corrosion requires that Wenner Four Pin ground resistance measurements per ASTM G57 be performed by corrosion engineers at various locations of the site to determine the best depths and locations for anode installations. Ideally, a sample pile is installed and experiments determining current requirement are conducted. Using this data, the decision is made whether a GACP system is feasible or if an ICCP must be used.

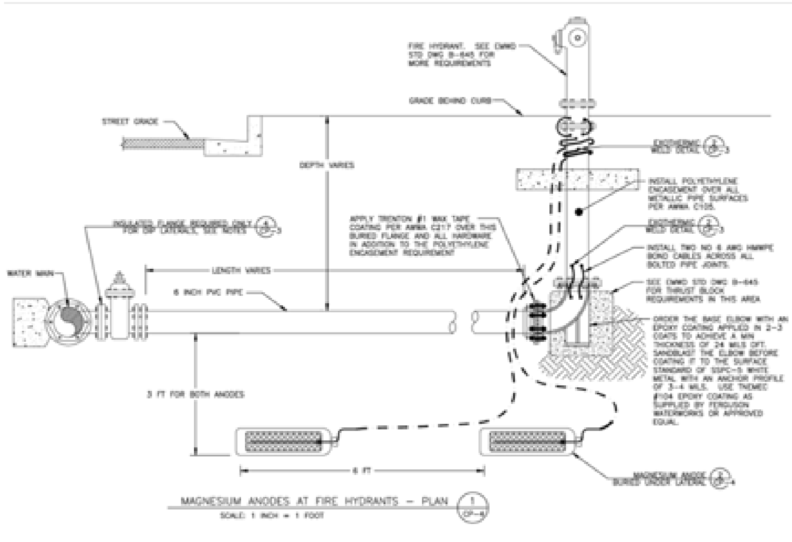


Figure 7 Sample anode design for fire hydrant underground piping

Vessels such as water tanks will have protective interior coatings and anodes to protect the interior surfaces. Anodes can also be buried on site and connected to system skid supports to protect the metal in contact with soil. A good example of a vessel cathodic protection system exists in all home water heaters which contain sacrificial aluminum or magnesium anodes. In environments that exceed 140F, zinc anodes cannot be used with carbon steel because they become the aggressor (Cathodic) to the steel instead of sacrificial (anodic). Anodes in vessels containing extremely brackish water with chloride levels over 2,000 ppm should inspect or change out their anodes every 6 months.

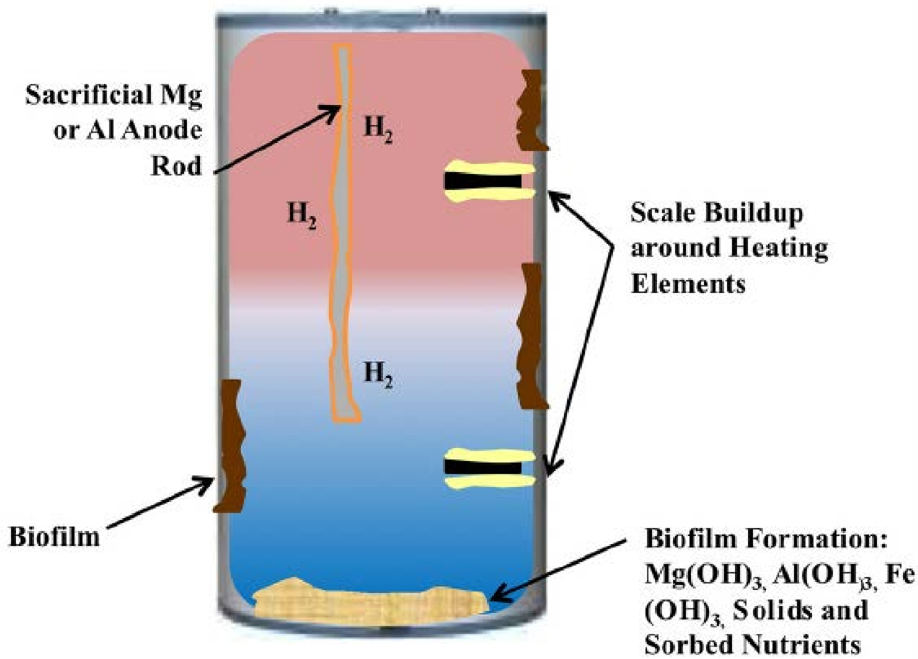


Figure 8 Cross section of boiler with anode

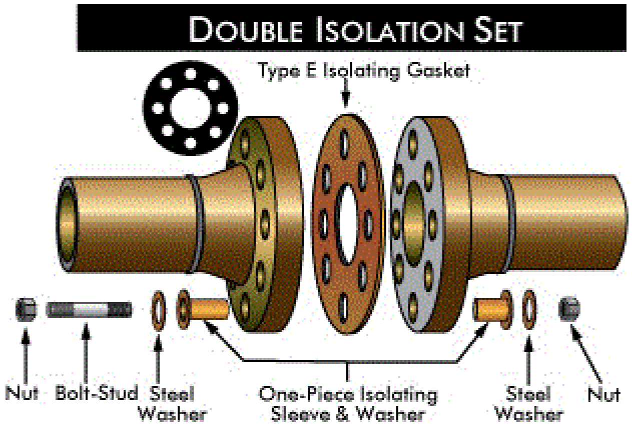
Cathodic protection can only protect a few diameters within a pipeline thus it is not recommended for small diameter pipelines and tubing internal corrosion protection. Anodes are like a lamp shining light in a room. They can only protect along their line of sight.

5.4.5 Good Electrical Continuity

In order for cathodic protection to protect a long pipeline or system of pipes from external soil side corrosion, they must all be electrically continuous to each other so that the electric current from the anode can travel along the pipes, then return through the earth to the anode. Electrical continuity is achieved by welding or pin brazing #8 AWG copper strand bond cable to the end of pipe sticks which have rubber gaskets at bell and spigots. If steel pipes are joined by full weld, bonding wires are not needed.

Electrical continuity between dissimilar metals is not desirable. Isolation joints or di-electric unions should be installed between dissimilar metals, such as steel pipes connecting to a brass valve per NACE SP0286. Bonding wires should then be welded onto the steel pipes by-passing the brass valve so that the cathodic protection system's current can continue to travel along the steel piping but isolate the brass valve from the steel pipeline. Another option would be to provide a separate cathodic protection system for steel pipes on both sides of the brass valve.

Typically, water heater inlets and outlets, gas meters and water meters have dielectric unions installed in them to separate utility property from homeowner property. This also protects them in the case that a home owner somehow electrically connects water pipes or gas pipes to a neighborhood electrical grounding system which can potentially have less noble steel in soil now connected to much more noble copper in soil which will then create a corrosion cell. This is exactly how a lemon powered clock works when a galvanized zinc nail and a steel nail are inserted into a lemon then connected to a clock. The clock is powered by the corrosion cell created.



5.4.6 Bad Electrical Continuity

Bad electrical continuity is when two different materials or systems are made electrically continuous (aka shorted) when they were not designed to be electrically continuous. Examples of this would be when gas lines are shorted to water lines or to electrical grounding beds. Very often, fire risers are shorted to electrical grounding systems, and water pipes at business parks. Since fire risers usually have a very short ductile iron pipe in the ground which connects to PVC pipe systems, they tend to experience leaks after 7 to 10 years of being attacked by underground copper systems.

It is absolutely imperative that any copper water piping or other metal conduits penetrating cement slab or footings, not come in contact with the reinforcing steel or post-tensioning tendons to avoid creation of galvanic corrosion cells.

5.4.7 Corrosion Test Stations

Corrosion test stations should be installed every 1,000 feet along pipelines in order to measure corrosion activity in the future. For a simple pipeline, two #8 AWG copper strand bond cable welded or pin brazed onto the pipeline are run up to finished grade and left in a hand hole. Corrosion test stations are used to measure pipe-to-soil electro potential relative to a copper copper-sulfate reference electrode to determine if the pipe is experiencing significant corrosion activity. By measuring test stations along a pipeline, hot spots can be determined, if any. The wires also allow for electrical continuity testing, condition assessment, and a multitude of other types of tests.

At isolation joints and pipe casings, two wires should be welded to either side of the isolation joint for a total of 4 wires to be brought up to the hand hole. This allows for future tests of the isolation joint, casing separation confirmation, and pipe-to-soil potential readings during corrosion surveys.

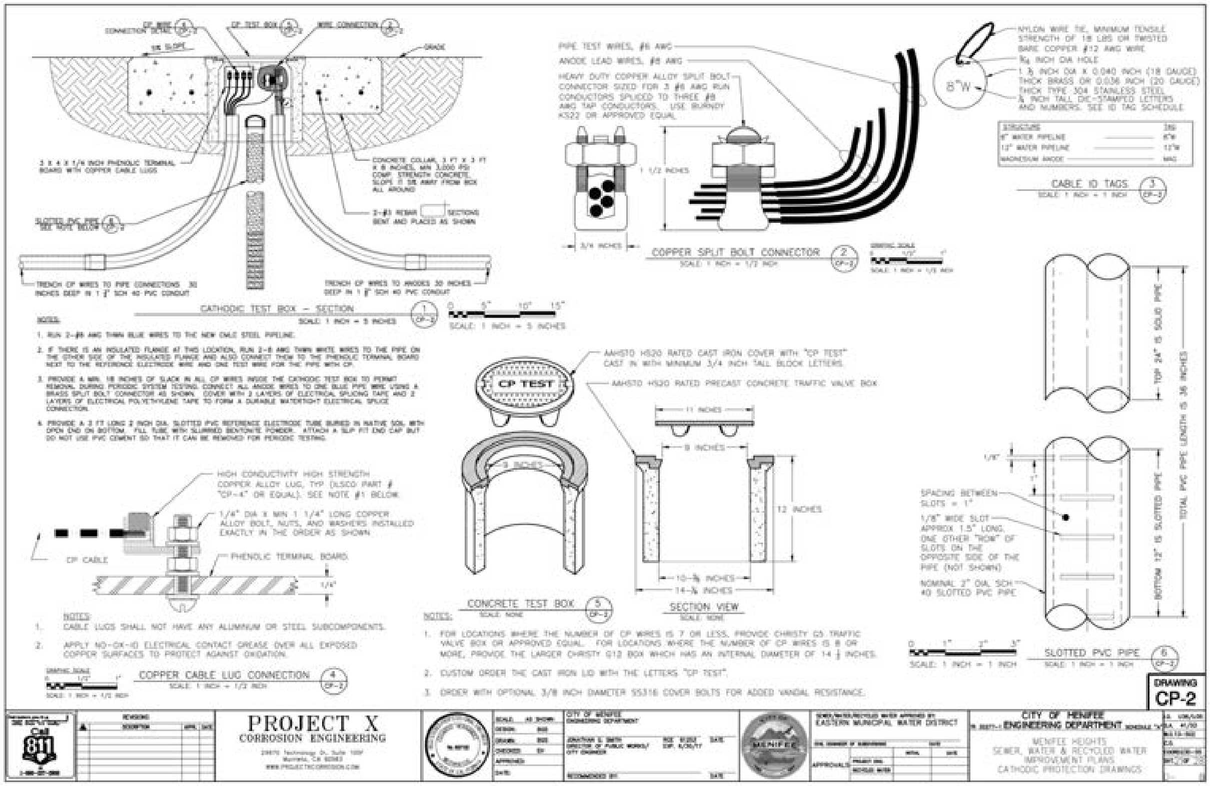


Figure 9 Sample of corrosion test station specification drawing

5.4.8 Excess Flux in Plumbing

Investigations of internal corrosion of domestic water plumbing systems almost always finds excess flux to be the cause of internal pitting of copper pipes. Some people believe that there is no such thing as too much flux. Flux runs have been observed to travel up to 20 feet with pitting occurring along the flux run. Flushing a soldered plumbing system with hot water for 15 minutes can remove significant amounts of excess flux left in the pipes. If a plumbing system is expected to be stagnant for some time, it should be drained to avoid stagnant water conditions that can lead to pitting and dezincification of yellow brasses.

5.4.9 Landscapers and Irrigation Sprinkler Systems

A significant amount of corrosion of fences is due to landscaper tools scratching fence coatings and irrigation sprinklers spraying these damaged fences. Recycled water typically has a higher salt content than potable drinking water, meaning that it is more corrosive than regular tap water. The same risk from damage and water spray exists for above ground pipe valves and backflow preventers. Fiber glass covers, cages, and cement footings have worked well to keep tools at an arm's length.

5.4.10 Roof Drainage splash zones

Unbelievably, even the location where your roof drain splashes down can matter. We have seen drainage from a home's roof valley fall directly down onto a gas meter causing it's piping to corrode at an accelerated rate reaching 50% wall thickness within 4 years. It is the same effect as a splash zone in the ocean or in a pool which has a lot of oxygen and agitation that can remove material as it corrodes.

5.4.11 Stray Current Sources

Stray currents which cause material loss when jumping off of metals may originate from direct-current distribution lines, substations, or street railway systems, etc., and flow into a pipe system or other steel structure. Alternating currents may occasionally cause corrosion. The corrosion resulting from stray currents (external sources) is similar to that from galvanic cells (which generate their own current) but different remedial measures may be indicated. In the electrolyte and at the metal-electrolyte interfaces, chemical and electrical reactions occur and are the same as those in the galvanic cell; specifically, the corroding metal is again considered to be the anode from which current leaves to flow to the cathode. Soil and water characteristics affect the corrosion rate in the same manner as with galvanic-type corrosion.

However, stray current strengths may be much higher than those produced by galvanic cells and, as a consequence, corrosion may be much more rapid. Another difference between galvanic-type currents and stray currents is that the latter are more likely to operate over long distances since the anode and cathode are more likely to be remotely separated from one another. Seeking the path of least resistance, the stray current from a foreign installation may travel along a pipeline causing severe corrosion where it leaves the line. Knowing when stray currents are present becomes highly important when remedial measures are undertaken since a simple sacrificial anode system is likely to be ineffectual in preventing corrosion under such circumstances.[17] Stray currents can be avoided by installing proper electrical shielding, installation of isolation joints, or installation of sacrificial jump off anodes at crossings near protected structures such as metal gas pipelines or electrical feeders.

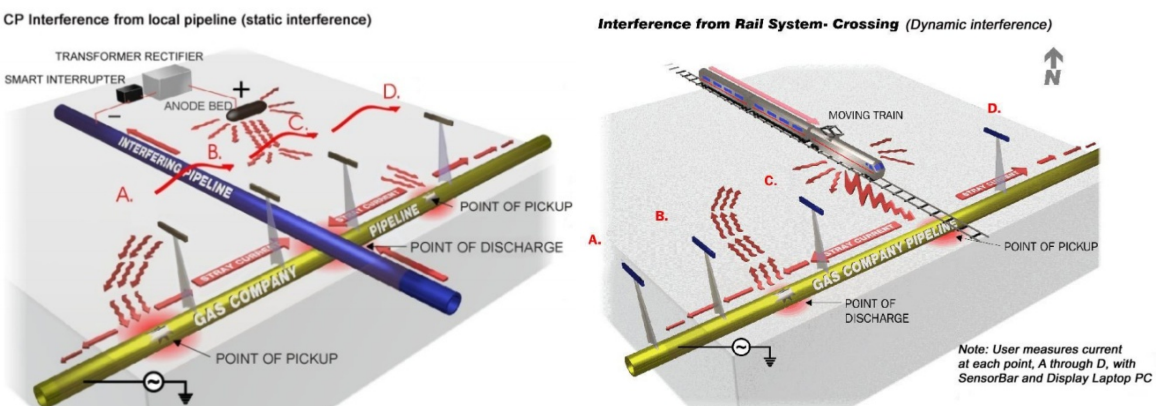


Figure 10 Examples of Stray Current[18]

[17] http://corrosion-doctors.org/StrayCurrent/Introduction.htm

[18] http://www.eastcomassoc.com/

Project X
Corrosion Engineering
Corrosion Control - Soil, Water, and Metallurgy Lab

Lab Request Sheet Chain of Custody
Phone: (213) 928-7213 · Fax (951) 226-1720 · www.projectxcorrosion.com
Ship Samples To: 29970 Technology Dr, Suite 105F, Murrieta, CA 92563

Project X Job Number S210121 B Geotek 2593-CR TR 31513 15 Full

IMPORTANT: Please complete Project and Sample Identification Data as you would like it to appear in report & include this form with samples.

Company Name:	GEOTEK Inc.	**Contact Name:**	**Phone No. :**
Mailing Address:	1548 North Maple St., Corona, CA 92280	**Contact Email:**	ascott@geotekusa.com
Accounting Contact:		**Invoice Email:**	ascott@geotekusa.com
Project Name:	TR 31513	**Client Project No:**	2593-CR

	5 Day Normal	3 Day RUSH 75% mark-up	2 Day RUSH 100% mark-up
P.O. #:			
Turn Around Time:	X		

Results By: ☐ Phone ☐ Fax ☒ Email ☐ Mail ☐ Overnight Mail (charges apply)

Date & Received by : 1/21/20

SPECIAL INSTRUCTIONS:

ANALYSIS REQUESTED (Please circle)

		SAMPLE ID - BORE #	**DESCRIPTION**	**DEPTH (ft)**	**DATE COLLECTED**	CORROSION SERIES	Soil Resistivity	pH	Sulfate	Chloride	Redox Potential	BiCarbonate	Alkalinity	Acidity	Nitrate	Ammonia	Sulfide	Moisture Content	Soil Corrosivity Evaluation Report	Metallurgical Analysis	**NOTES**
					Default Method	Min. Resistivity, Sulfate, Chloride, Sulfide, Redox, pH, Ammonia, Nitrate.	Caltrans CTM64 / AASHTO T2888 / ASTM G187	Caltrans CTM64 / AASHTO T 289 / ASTM G 51	Caltrans CTM41 / AASHTO T 290 / ASTM D516	Caltrans CTM42 / AASHTO T 291 / ASTM D512B	SM 2580B	SM 2320B	SM 2520B	SM 2510B	SM 4500-NO3 / Hach 835	SM 4500-NH3 / Hach 830	SM 4500-S2 / SM 4500-S2	ASTM D2216			
133	1	S1		0-1		X															
134	2	S2		0-1		X															
135	3	S3		0-1		X															
136	4	S4		0-1		X															
137	5	S5		0-1		X															
138	6	S6		0-1		X															
139	7	S7		0-1		X															
140	8	S8		0-1		X															
141	9	S9		0-1		X															
142	10	S10		0-1		X															
143	11	S11		0-1		X															
144	12	S12		0-1		X															
145	13	S13		0-1		X															
146	14	S14		0-1		X															
147		S15		0-1		X															

APPENDIX F

GENERAL GRADING GUIDELINES

Updated Geotechnical Evaluation
Proposed Single-Family Residential Development
Tract No. 31513
Hemet, Riverside County, California
Project No. 2593-CR

GEOTEK

GENERAL GRADING GUIDELINES

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

General

Grading should be performed to at least the minimum requirements of governing agencies, Chapters 18 and 33 of the Uniform Building Code, CBC (2022) 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 alluvium, and/or weathered bedrock 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.

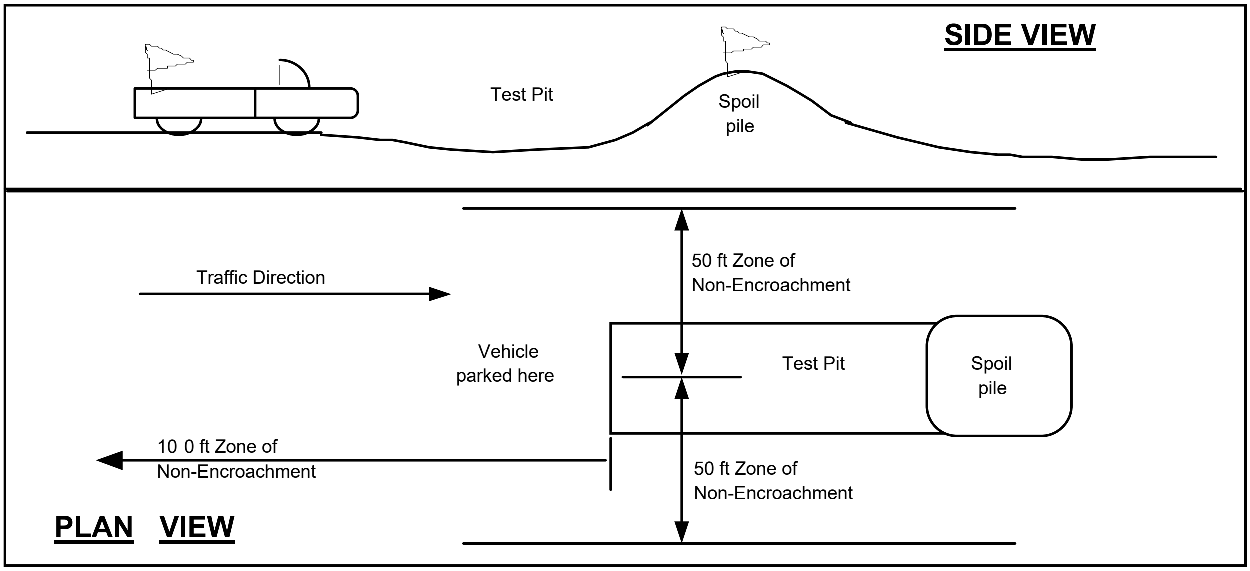
Test Pits Location, Orientation and Clearance

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 contractors 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

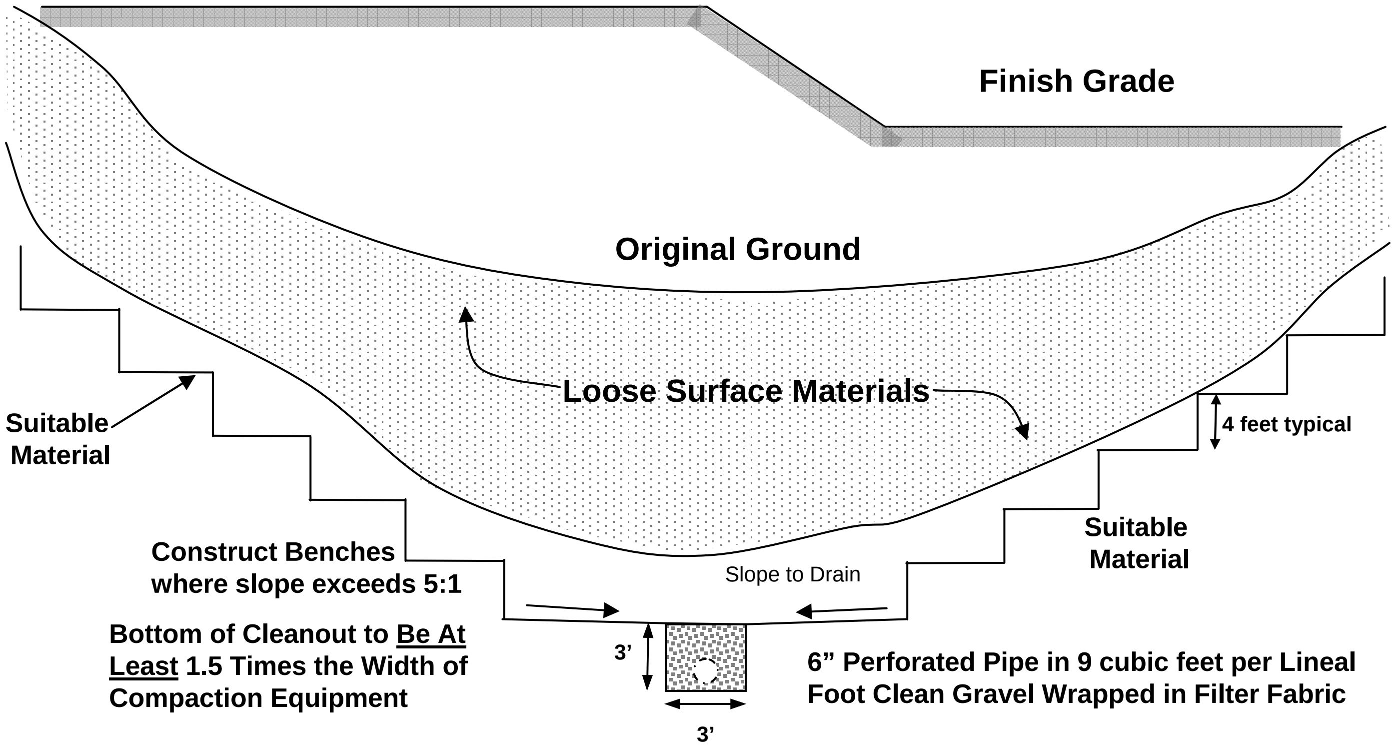
In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is directed to inform both the developer's and contractor's representatives. If the condition is not rectified, the technician is required, by 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.

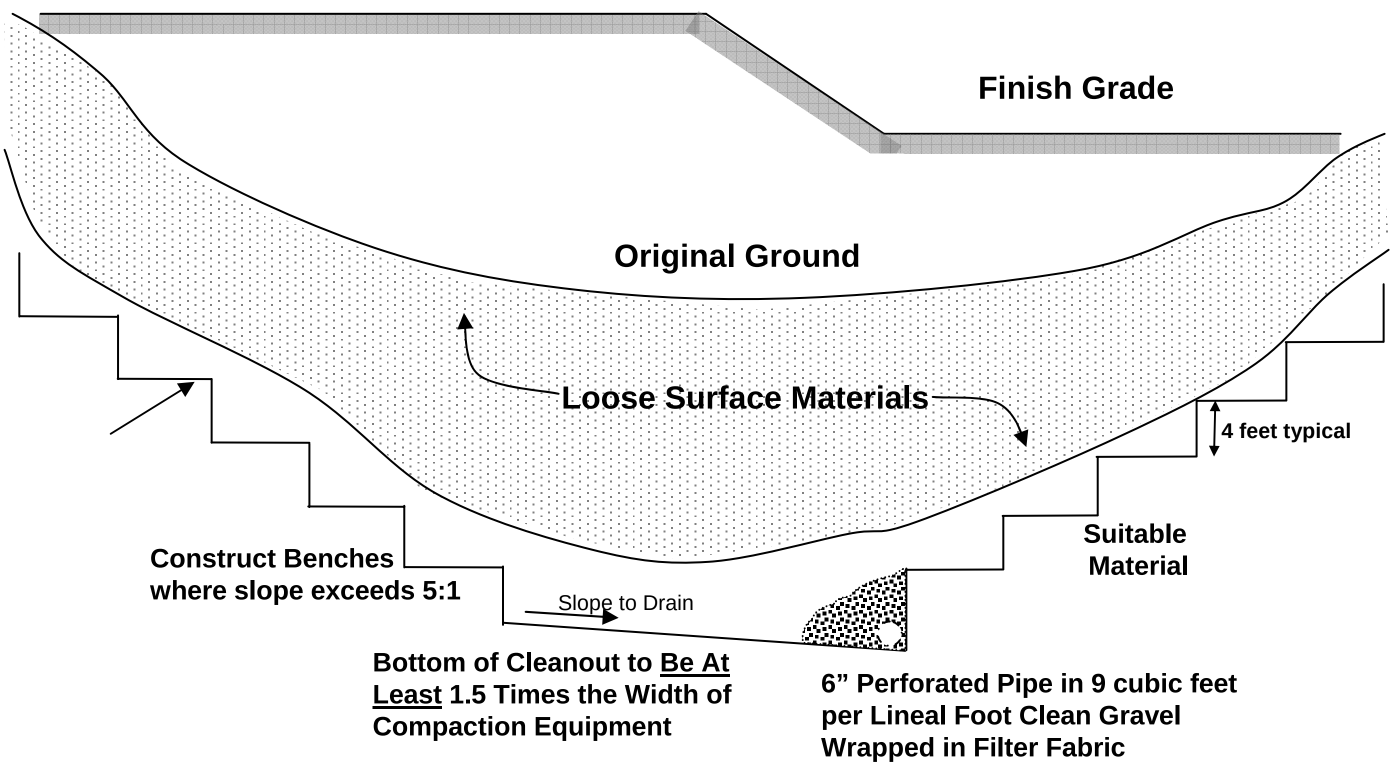
In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to technicians attention and notify our project manager or office. Effective communication and coordination between the contractors' representative and 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.

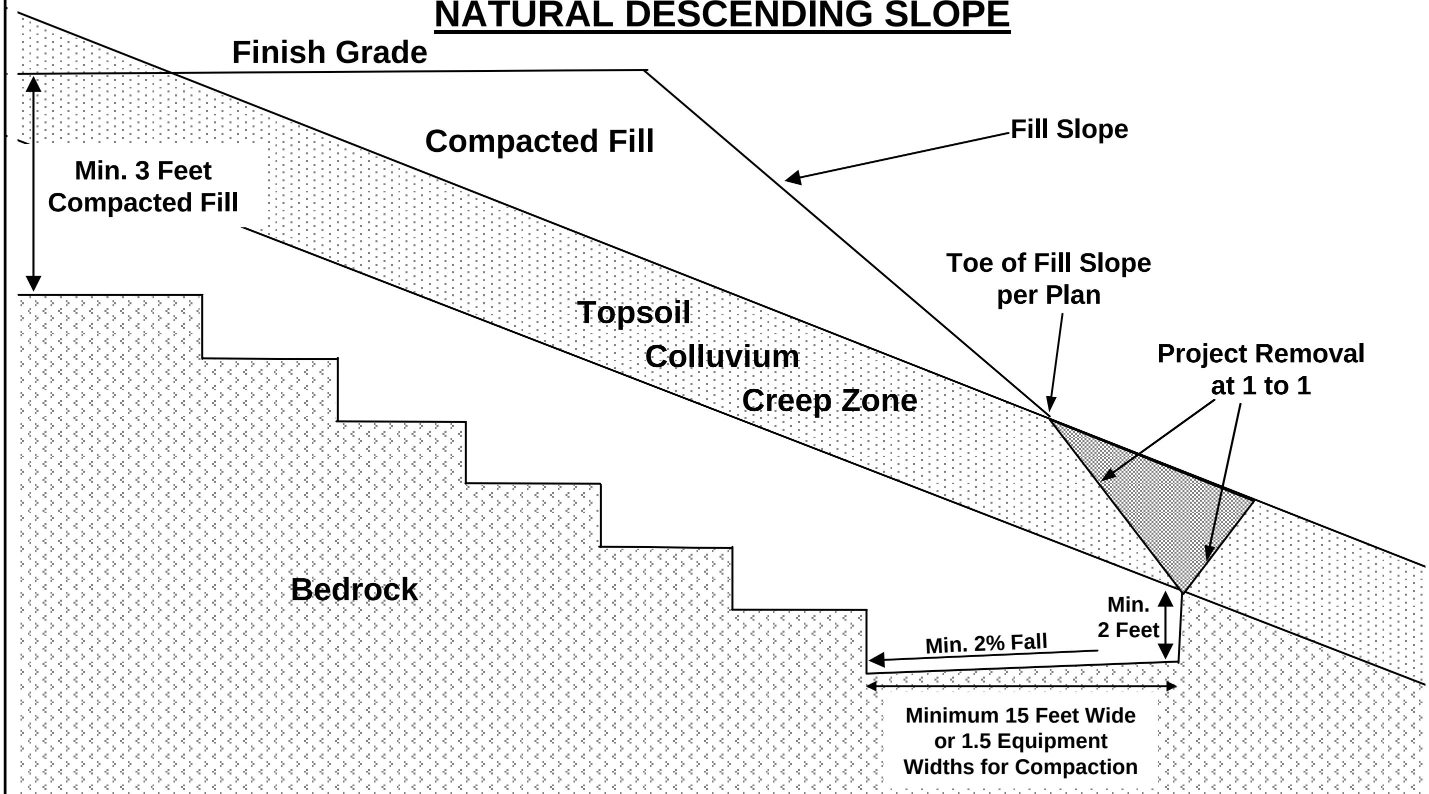
ALTERNATES



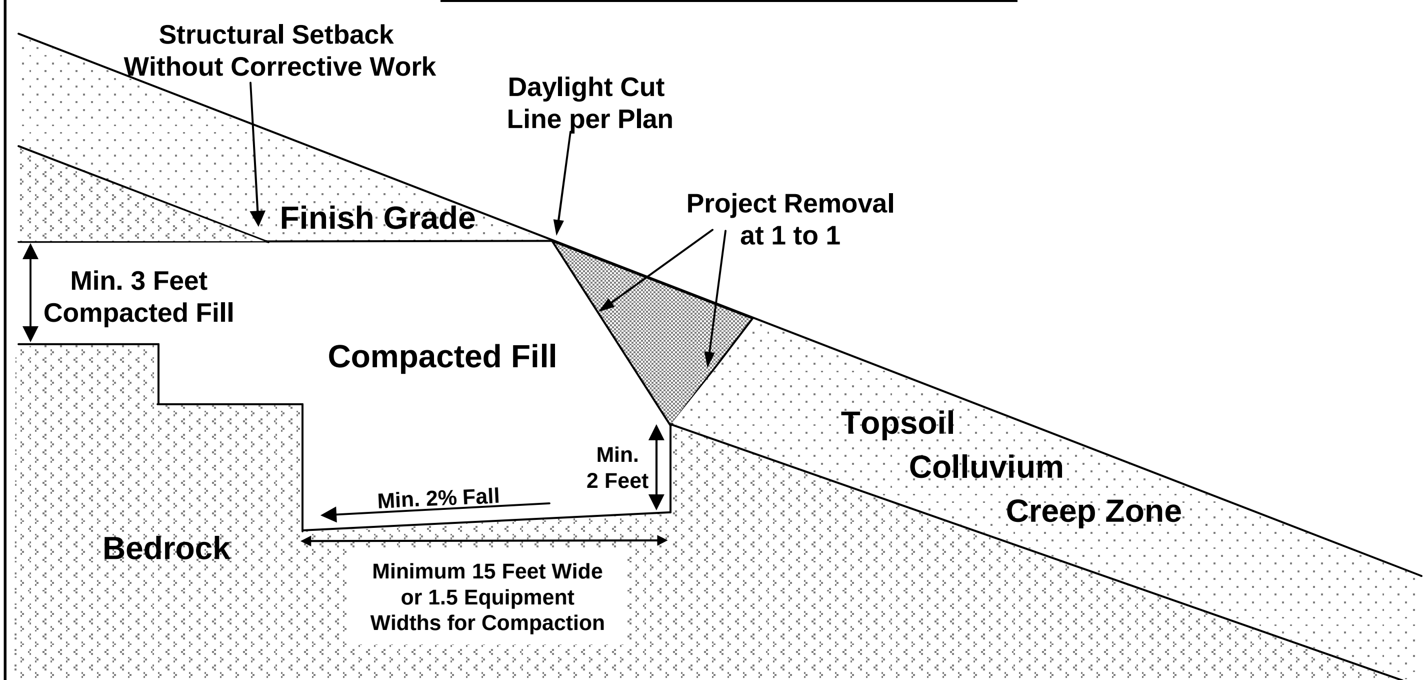


GEOTEK	1548 North Maple Street Corona, California 92878	TYPICAL CANYON CLEANOUT	STANDARD GRADING GUIDELINES PLATE E-1

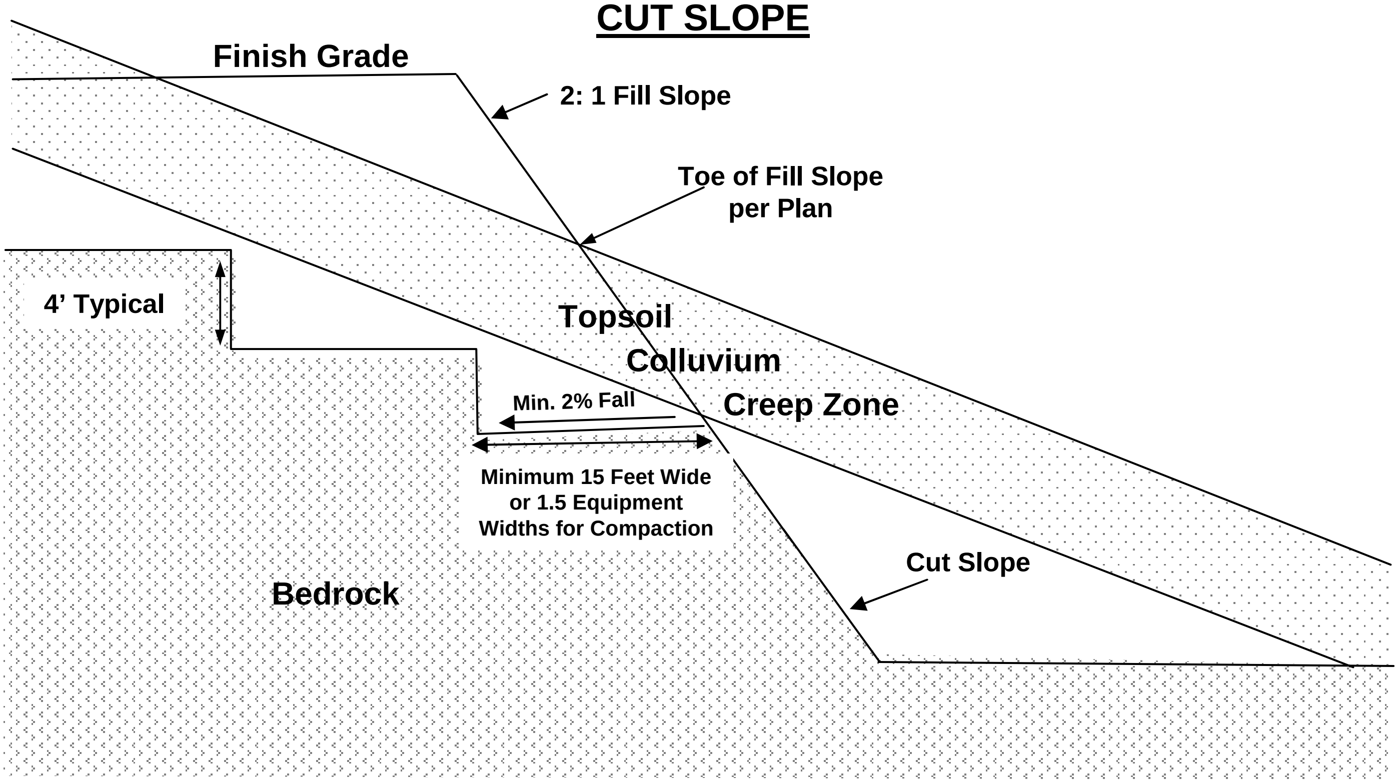
TYPICAL FILL SLOPE OVER NATURAL DESCENDING SLOPE



DAYLIGHT CUT AREA OVER NATURAL DESCENDING SLOPE



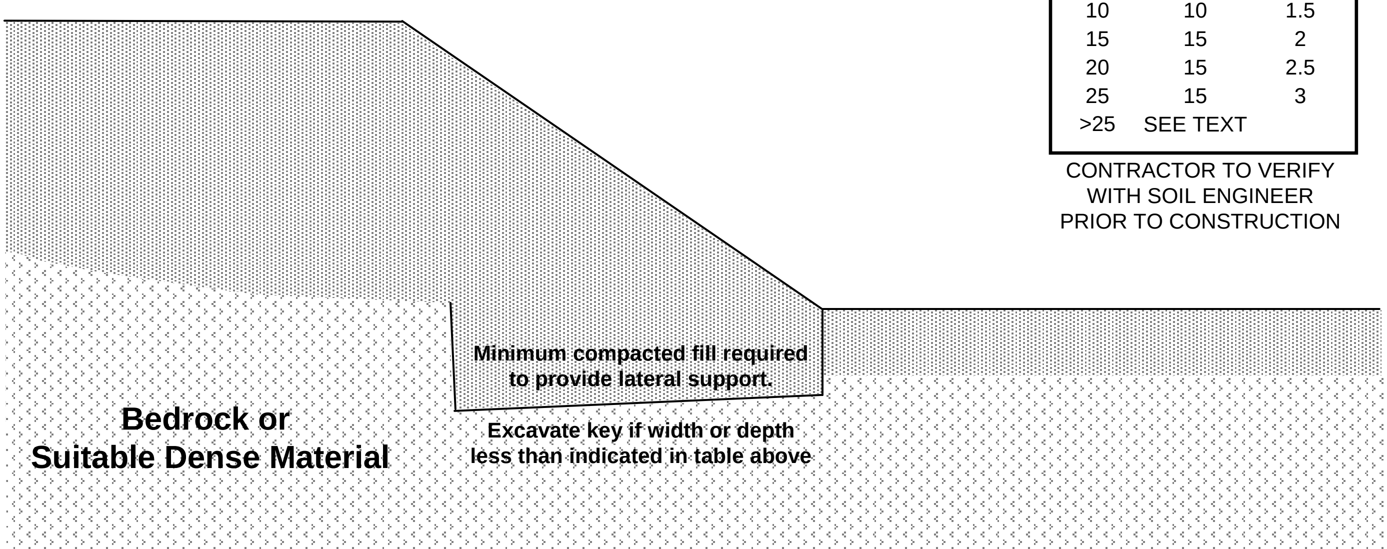
TYPICAL FILL SLOPE OVER CUT SLOPE



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



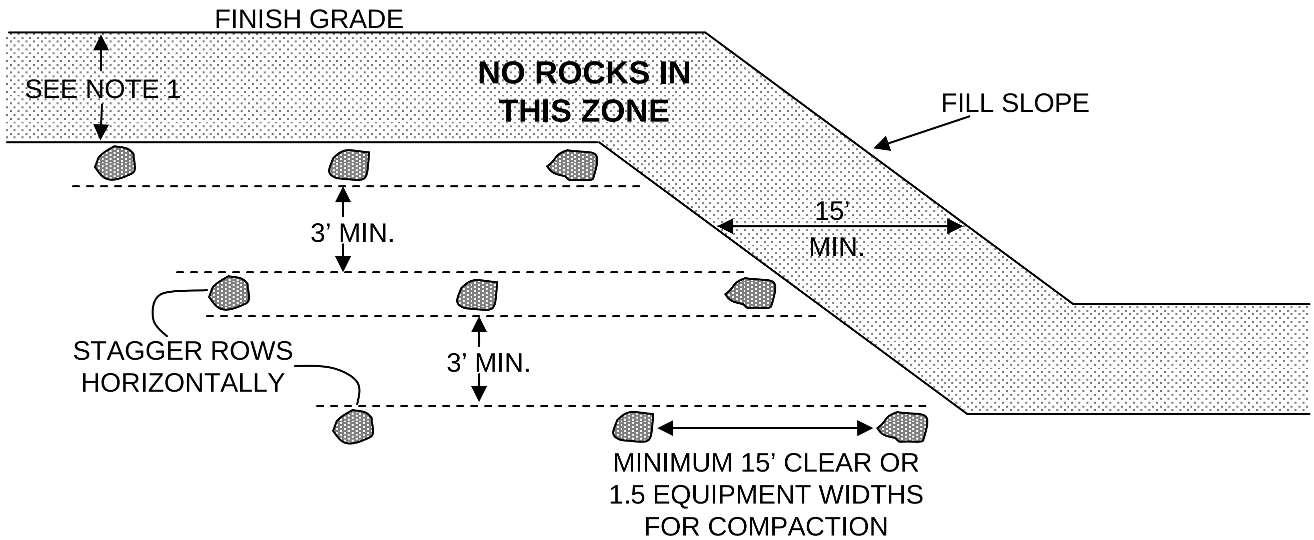
GEOTEK
1548 North Maple Street
Corona, California 92878

COMMON FILL SLOPE KEYS

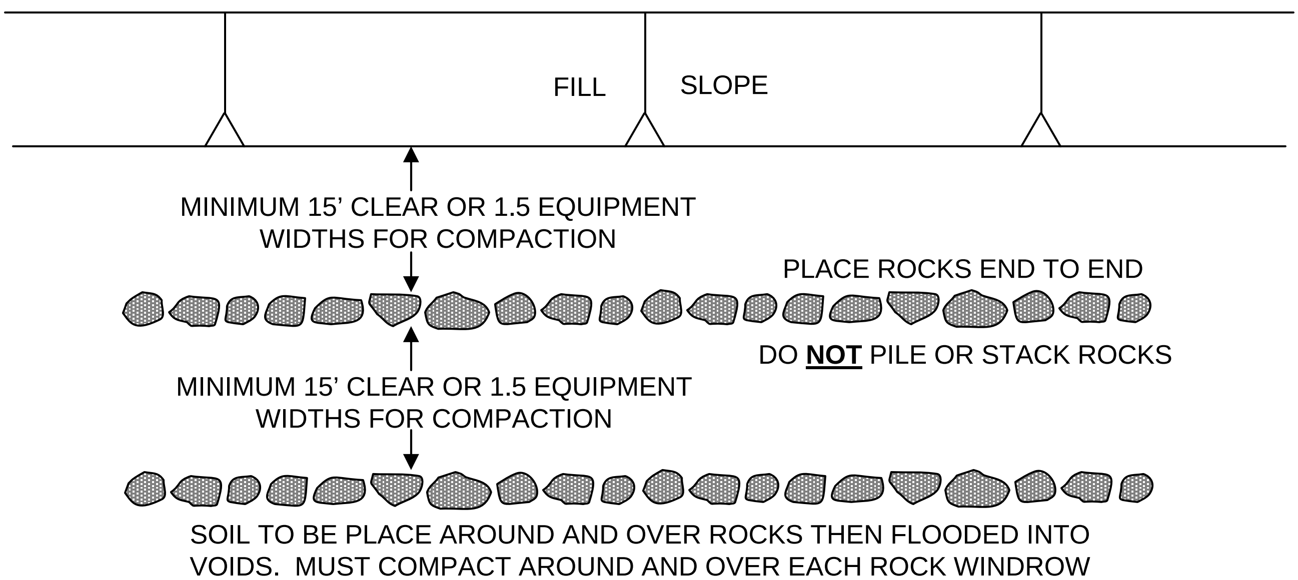
STANDARD GRADING GUIDELINES

PLATE E-3

CROSS SECTIONAL VIEW

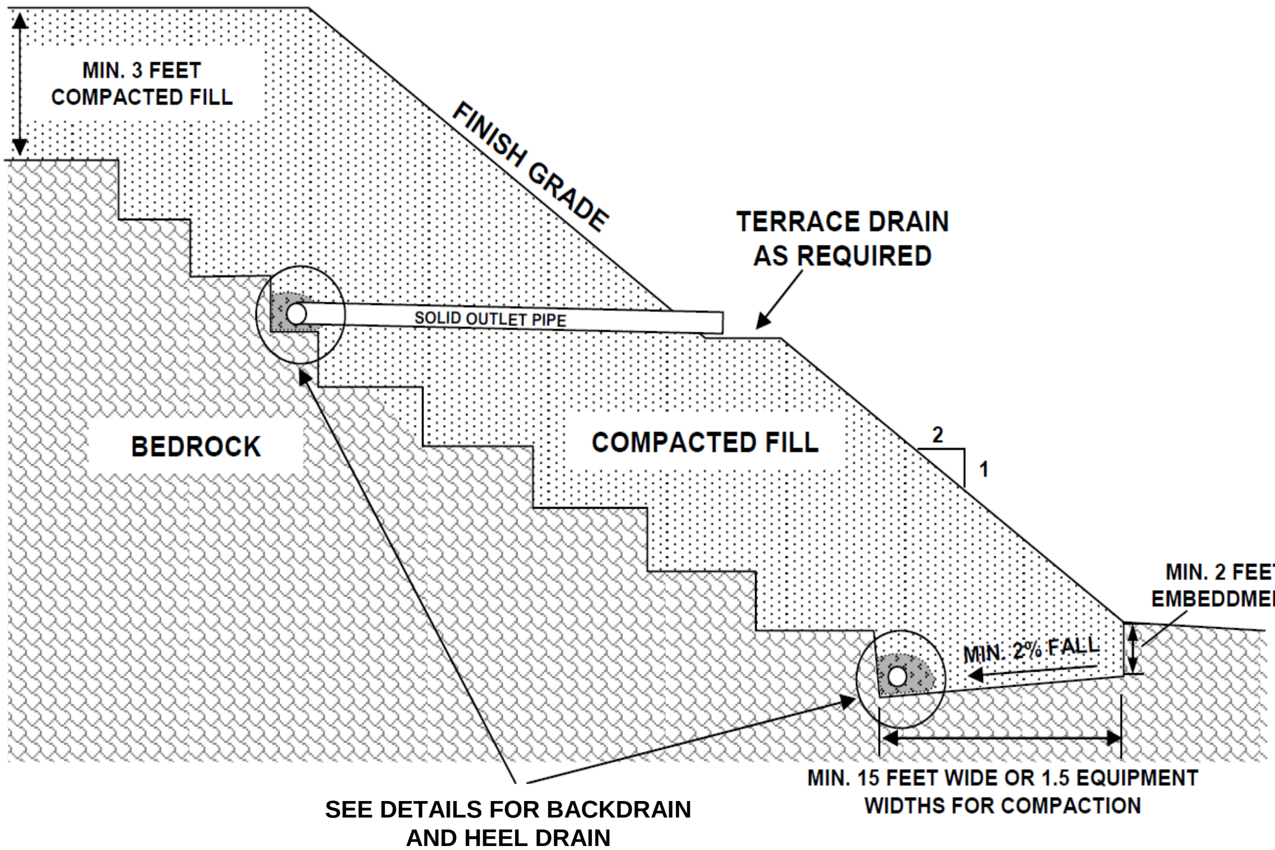


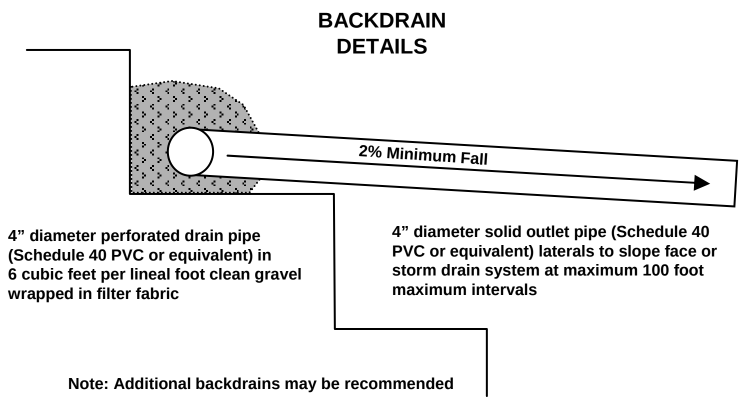
PLAN VIEW

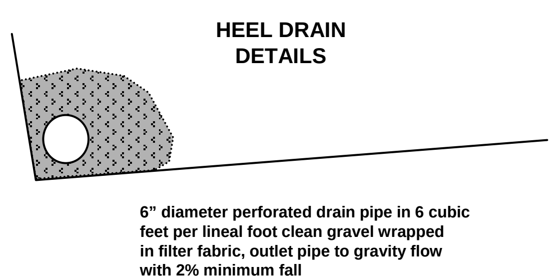


NOTES:

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







GEOTEK	1548 North Maple Street Corona, California 92878	TYPICAL BUTTRESS AND STABILIZATION FILL	STANDARD GRADING GUIDELINES PLATE E-5