# GEOTECHNICAL UPDATE & PERCOLATION TEST RESULTS

## HEMET 30
## SOUTH OF HIGHWAY 74 AND EAST OF JOEL DRIVE
## HEMET AREA OF RIVERSIDE COUNTY, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

**GLOBAL INVESTMENT AND DEVELOPMENT**
**LOS ANGELES, CALIFORNIA**

**DECEMBER 10, 2020**
**PROJECT NO. T2214-22-02**

Project No. T2214-22-02
December 10, 2020

Global Investment and Development
3470 Wilshire Boulevard, Suite 1020
Los Angeles, California 90010

Attention: Mr. Joseph Rivani

Subject: GEOTECHNICAL UPDATE & PERCOLATION TEST RESULTS
HEMET 30 SOUTH OF HIGHWAY 74 & WEST OF JOEL DRIVE
HEMETAREA OF RIVERSIDE COUNTY, CALIFORNIA

Dear Mr. Rivani:

In accordance with your authorization of Geocon Proposal IE-2666 dated October 23, 2020, Geocon West, Inc. (Geocon) herein submits the results of our geotechnical update and percolation test results for the proposed residential development. The accompanying report presents the results of our study, conclusions, and recommendations pertaining to the geotechnical aspects of the proposed development. The site is considered suitable for the proposed development provided the recommendations of this report are followed.

Should you have questions regarding this report, or if we may be of further service, please contact the undersigned at your convenience.

Very truly yours,

**GEOCON WEST, INC.**

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# GEOTECHNICAL UPDATE & PERCOLATION TEST RESULTS

## 1. PURPOSE AND SCOPE

This report presents our geotechnical update and the test results of recent percolation testing for the proposed residential development located south of Highway 74 and west of Joel Drive in the Hemet area of Riverside County, California (see *Vicinity Map*, Figure 1). The purpose of the study was to review existing geotechnical information for the site with respect to the latest grading plan prepared by Anderson Consulting Engineers, Inc. and, based on the site conditions, provide updated recommendations pertaining to the geotechnical aspects of developing the property based on 2019 California Building Code (CBC). We also performed percolation testing in the proposed basin and temporary channel in the northern area of the property for stormwater mitigation design.

The scope of our work included review of our subsurface exploration performed in 2004, a site reconnaissance, Underground Service Alert site mark out, excavation and testing of nine percolation test locations, engineering analyses with respect to the 2019 CBC, and the preparation of this report. A summary of the information reviewed for this study is presented in the *List of References*.

Geocon performed a reconnaissance of the site on November 3, 2020 to observe current site conditions with respect to the previous site conditions in 2004 at the time at our previous geotechnical report. Geocon also performed a percolation testing in the proposed basin and temporary channel in the northern portion of the site on November 9 through 11, 2020. The approximate locations of the excavations are depicted on the *Geologic Map (*Figure 2). A detailed discussion of the field investigation, excavation logs of the geotechnical and percolation excavations, and the percolation test data are presented in *Appendix A.*

We incorporated the 2004 seismic refraction traverses and trench data into this report and considered that data in our updated analyses and recommendations for the site.

Grain size analyses was performed on samples collected from the bottom of the percolation test excavations. The test results are presented in *Appendix B.*

## 2. SITE AND PROJECT DESCRIPTION

Based on the referenced *Hemet 30 TTM Grading Exhibit* prepared by Anderson Consulting Engineers, the residential development proposed by Global Investment and Development will include the construction of 147 single family homes, two parks, intra-tract trails, a basin and temporary channel along the northern area of the property, and associated infrastructure improvements.

The approximately 28-acre project site is located at the latitude of 33.7420 and longitude of -117.0559. The site is bounded on the north by Highway 74, on the south by Lynn Avenue (if extended west), on the east by Joel Drive, and on the west by an open field. Open fields were observed along most of the eastern boundary, southwest and west of the property. A bedrock hill in the south-central area of the property has been quarried from prior to 1949 to the present. A gently northeast sloping alluvial fan occupies the northern portion of the property. The site elevations currently range from approximately 1,527 feet to 1,571 feet above mean sea level (MSL). The site slopes downward to the northeast. Access is currently gained along the western side of the site from Highway 74. At the time of our field exploration, vegetation consisted of a moderate growth of grasses and occasional trees with multiple piles of undocumented fill consisting of sand and cobbles.

Based on the *Hemet 30 TTM Grading Exhibit* site plan (Anderson Consulting Engineers (ACE)), cuts (southern portion of the site) and fills (northern portion of the site) on the order of 17 and 11 feet, respectively, are planned for the site. Fills will be deepened to approximately 26 feet after remedial removals have been accomplished. Fill slopes are expected to be a maximum of 20 feet in height, and the maximum cut slope height is approximately 22 feet. ACE has also indicated the deepest utilities will be approximately 25 feet below grade at the site.

Although structural plans and loading information is unavailable at this time, we expect that the proposed buildings will be one to two stories and consist of a light-frame wood and/or metal-stud framing construction supported on conventional concrete shallow foundations with slab-on-grade floors. We expect column loads will be up to 150 kips and wall loads will be up to 10 kips per linear foot. Preliminary geotechnical recommendations for design of the structure are based on these assumptions and provided herein.

References to elevations presented in this report are based on the referenced project documents and Google Earth. Geocon does not practice in the field of land surveying and is not responsible for the accuracy of such topographic information.

The locations and descriptions provided herein are based on a site reconnaissance, our field exploration, review of our subsurface exploration performed in 2004, and project information provided by the client. If project details differ significantly from those described, Geocon should be contacted for review and possible revision to this report.

## 3. BACKGROUND

Geocon performed a geotechnical investigation of the site in 2004. The investigation included two seismic refraction traverses and excavation of 16 trenches. Laboratory test results indicated that site soils were not expansive and contained negligible sulfate. Groundwater was not encountered on the site but was encountered at depths of 32 to 42 feet north of the site in 1995 and 2002. Based on elevations, these depths would equate to an approximate depth of 47 feet at the subject site. The seismic refraction data indicated that the bedrock at the site is expected to be marginally rippable to depths of 25 to 38 feet during rough grading at the locations of the traverses. Faulting, liquefaction, and landsliding do not pose geologic hazards at the site. We recommended remedial removals of 1 to 5 feet with localized deeper remedial removals of 12 feet in the northern portion of the proposed development. The previous test pits should also be over-excavated and replaced with engineered fill during grading. The site is currently plowed and essentially unchanged from the time of our investigation in 2004.

## 4. GEOLOGIC SETTING

The site is located within the Lakeview Mountains area between the San Jacinto and Elsinore/Temecula Valleys within the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are bounded on the north by the Transverse Ranges and the Cucamonga/Sierra Madre faults, the east by the San Jacinto fault, the west by the Elsinore fault and the Santa Ana Mountains. The Peninsular Ranges extend southward into Mexico. The Peninsular Ranges are characterized by granitic highlands of low to moderate relief surrounded by alluvial plains and valleys. Locally, the site is located on the southeastern margin of the Lakeview Mountains at the San Jacinto Valley. The Lake View Mountains are an erosional remnant of a granitic pluton emplaced in the area during the Cretaceous Period. Geologic mapping by Morton (2003) identifies the bedrock unit at the site as Green Acres Gabbro with a lesser occurrence of granodiorite possible.

# 5. GEOLOGIC MATERIALS

## 5.1 General

The geologic units at the site include undocumented fill generated during the quarry operations, young alluvial fan deposits in the northern half of the site, older alluvial fan deposits in the southern and southeastern areas of the site, and Green Acres Gabbro underlies the alluvium on the site. Granodiorite may also be present in localized areas of the site. The geologic nomenclature follows that of Morton (2003). The descriptions of the geologic conditions are presented on the logs in *Appendix A* and discussed below. The extent of the surficial geologic units is depicted on the *Geologic Map*, Figure 2.

## 5.2 Undocumented Artificial Fill (afu)

Undocumented fill was observed in the majority of the site in end dumped piles and spread across the site in proximity to the end dumped piles overlying cut bedrock and alluvial fan deposits. The fill consists of silty sand with cobbles and occasional boulders.

## 5.2 Young Alluvial Fan Deposits (Qyf)

Young alluvial fan deposits were encountered in the northern and western areas of the site. The younger alluvium consists primarily of orange to red brown silty sand which was found to be loose, damp, and porous.

## 5.3 Old Alluvial Fan Deposits (Qof)

Old alluvial fan deposits were encountered in the southern and eastern areas of the site overlying the bedrock. The older alluvium was found to be generally red brown, damp, medium dense, cemented and moderately porous in the upper 1 to 3 feet, and dense below the cemented zone.

## 5.4 Green Acres Gabbro (Kgab)

Green Acres Gabbro was encountered at depth throughout most of the site and forms the soil and rock in the end dumped piles. The Green Acres Gabbro is a heterogeneous gabbroic rock that is often intruded by quartz diorite and tonalite. It is generally massive with core stones common and is expected to be marginally rippable based on seismic refraction data to depths of approximately 25 to 38 feet.

## 5.5 Granodiorite (Khg)

Granodiorite is geologically mapped near the site (Morton, 2003) and may be encountered during grading. Typically, granodiorite is a lighter, stronger rock than the gabbro and difficult ripping should be expected.

## 5.6 Geologic Structure

The geologic structure consists of generally massive gabbroic bedrock with occasional northwest trending leucocratic intrusions and a regional joint pattern that trends northwest and dips gently to moderately to the northeast (Morton, 2003). The older alluvial fan was generated from the previous bedrock hill on the site with the resent alluvium deposited through the San Jacinto Valley.

# 6. GEOLOGIC HAZARDS

## 6.1 Surface Fault Rupture

The numerous faults in southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS), formerly known as California Division of Mines and Geology (CDMG), for the Alquist-Priolo Earthquake Fault Zone Program (Byrant and Hart, 2007). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,000 years). A potentially active fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years) but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive. The site is not within a currently established State or County Fault Zone for surface fault rupture hazards.

The site is located in the seismically active southern California region, and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active southern California faults. The faults in the vicinity of the site are listed below.

The closest surface trace of an active fault to the site is the Casa Loma branch of the San Jacinto fault located approximately 5 miles northeast of the site. Other nearby active faults are listed in Table 6.1, below.

**TABLE 6.1**
**ACTIVE FAULTS WITHIN 50 MILES OF THE SITE**

| Fault Name | Maximum Magnitude (Mw) | Geometry (Slip Character) | Slip Rate (mm/yr) | Information Source | Distance from Site (mi) | Direction from Site |
|---|---|---|---|---|---|---|

| | | | | | | |
|---|---|---|---|---|---|---|
| San Jacinto (Casa Loma) | 6.9 | RL-SS | 12 | a | 5 | NE |
| San Jacinto (Claremont) | 6.7 | RL-SS | 12 | a | 8 | NE |
| San Jacinto (Clark) | 7.2 | RL-SS | 12 | a | 8 | SE |
| Elsinore (Wildomar) | 6.8 | RL-SS | 5 | a | 16 | W |
| San Gorgonio Pass | n/a | THRUST | n/a | a | 17 | NE |
| Elsinore Fault (Glen Ivy) | 6.8 | RL-SS | 5 | a | 19 | W |
| San Jacinto (Coyote Creek) | 6.8 | RL-SS | 4.0 | a | 21 | SE |
| San Andreas (San Bernardino) | 7.5 | RL-SS | 24 | a | 25 | N |
| Pinto Mountain | 7.2 | LL-SS | 2.5 | a | 29 | NE |
| San Jacinto (Glen Helen) | 6.7 | RL-SS | 12 | a | 31 | N |
| Morongo Valley | 7.2 | LL-SS | 2.5 | a | 32 | NE |
| Chino | 6.7 | RL-R-O | 1 | a | 35 | NW |
| Cucamonga | 6.9 | R | 5 | a | 38 | N |

Geometry: BT = blind thrust, LL = left lateral, N = normal, O = oblique, R = reverse, RL = right lateral, SS = strike slip.

Information Sources: a = Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Wills, C.J., 2003, The Revised 2002 California Probabilistic Seismic Hazard Maps, including Appendices A, B, and C, dated June; b = online Fault Activity Map of California website, maps.conservation.ca.gov/cgs/fam/, as of 1/2017.

n/a = data not available

## 6.2 Seismicity

As with southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 4.0 within a radius of 60 miles of the site are depicted on Figure 4, Regional Seismicity Map. A number of earthquakes of moderate to major magnitude have occurred in the southern California area within the last 100 years. A partial list of these earthquakes is included in the following table.

**TABLE 6.2**
**LIST OF HISTORIC EARTHQUAKES**

| Earthquake (Oldest to Youngest) | Date of Earthquake | Magnitude | Distance to Epicenter (Miles) | Direction to Epicenter |
|---|---|---|---|---|
| Near Redlands | July 23, 1923 | 6.3 | 21 | NW |
| Long Beach | March 10, 1933 | 6.4 | 53 | W |
| Tehachapi | July 21, 1952 | 7.5 | 141 | NW |
| San Fernando | February 9, 1971 | 6.6 | 90 | WNW |
| Whittier Narrows | October 1, 1987 | 5.9 | 63 | WNW |
| Sierra Madre | June 28, 1991 | 5.8 | 65 | WNW |
| Landers | June 28, 1992 | 7.3 | 48 | NE |
| Big Bear | June 28, 1992 | 6.4 | 34 | NNE |
| Northridge | January 17, 1994 | 6.7 | 91 | WNW |
| Hector Mine | October 16, 1999 | 7.1 | 74 | NE |
| Ridgecrest China Lake Fault | July 5, 2019 | 7.1 | 143 | NNW |

## 6.3 Liquefaction

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Additionally, seismically induced “dry-sand” settlement may occur whether the potential for liquefaction exists or not.

The current standard of practice as outlined in the *Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California* (SCEC, 1999) requires a liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be enough to induce liquefaction.

According to the *Map My County* GIS system (RCIT, 2020), the site northern portion of the site is located in an area of HIGH liquefaction potential, the older alluvial area surrounding the hill is considered to have a MODERATE liquefaction potential and the area that was the hill and is now shallow bedrock does not have liquefaction potential.

Based on the recommended remedial grading proposed herein (see Section 8.3) and due to the site geology generally consisting of shallow gabbroic bedrock, neither liquefaction nor seismic “dry-sand” settlement is a design consideration for the site.

## 6.4 Expansive Soil

The on-site surficial soils generally consist of sands and silty sands. Laboratory test results from 2004 indicate site soils have a low expansion potential with an expansion index test result of 0.

## 6.5 Hydrocompression

Hydrocompression is the tendency of unsaturated soil structure to collapse upon wetting resulting in the overall settlement of the affected soil and overlying foundations or improvements supported thereon. Potentially compressible soils underlying the site are typically removed and recompacted during remedial site grading. However, if compressible soil is left in-place, a potential for settlement due to hydrocompression of the soil exists.

Laboratory testing was performed on select soil samples obtained during our 2004 investigation of the site in order to evaluate the hydrocompression characteristic of the alluvial soils overlying the bedrock. The soil samples tested exhibited a collapse potential ranging between 3.0 and 12.4 percent under high-pressure loading, in accordance with ASTM D2435-96 (outdated). The recommended remedial grading discussed herein (see Section 8.3) and the presence of shallow gabbroic bedrock should reduce the potential for hydrocompression.

## 6.6 Slope Stability

Proposed cut and fill slopes will be graded up to heights of approximately 20 feet high at slope inclinations of 2:1 (h:v) or flatter. Based on our understanding of site materials and the proposed grading, permanent graded fill slopes with gradients of 2:1 (horizontal:vertical) or flatter and vertical heights of 12 feet or less will possess Factors of Safety of 1.5 or greater under static conditions and 1.1 or greater under pseudo-static conditions. Grading of cut and fill slopes should be designed in accordance with the grading ordinances of the County of Riverside and the 2019 CBC.

# 7. PERCOLATION TESTING

Percolation testing was conducted in accordance with the procedures in the *Riverside County Flood Control and Water Conservation District LID BMP, Appendix A* (Handbook), within the proposed basin (6 tests) and within the temporary channel (3 locations) at -1 foot from proposed finished grade elevations. The percolation test locations are depicted on the *Geologic Map* (see Figure 2).

The percolation test holes were excavated to a depths of 1 to 3 feet below existing grades (-1 ft below proposed finished grades) to a diameter of approximately 12 inches. Approximately two inches of gravel was placed at the bottom of each test hole and a perforated pipe was placed atop the gravel to keep the test hole open. Gravel was placed around the bottom of the test hole to support the test pipe. The test locations were pre-saturated prior to testing. Infiltration test results are included as Figures A-26 through A-34. Results of the converted percolation test rates to infiltration test rates are presented in Table 7.0 below. The *Handbook* requires a factor of safety of 3 be applied to the values below based on the test method used.

The in-situ field percolation tests performed provide short-term infiltration rates, which apply mainly to the initiation of the infiltration process due to the short time of the test (hours instead of days) and the amount of water used. Where appropriate the short-term infiltration rates should be converted to long-term infiltration rates using reduction factors depending upon the degree of infiltrate quality, maintenance access and frequency, site variability, subsurface stratigraphy variation, and other factors. The small-scale percolation testing cannot model the complexity of the effect of interbedded layers of different soil composition, and our test results should be considered only as index values of infiltration rates.

**TABLE 7.0**
**INFILTRATION TEST RATES**

| Parameter | P-1 | P-2 | P-3 | P-4 | P-5 | P-6 | P-7 | P-8 | P-9 |
|---|---|---|---|---|---|---|---|---|---|
| **Depth (inches)** | 36 | 19.1 | 19.8 | 19.3 | 19.8 | 19.9 | 19.1 | 18.7 | 19.0 |
| **Test Type** | Normal | Normal | Normal | Normal | Normal | Normal | Normal | Normal | Normal |
| **Change in head over time: ΔH (inches)** | 3.0 | 3.1 | 5.6 | 3.5 | 4.0 | 5.2 | 11.2 | 3.5 | 5.8 |
| **Average head: Havg (in)** | 14.0 | 12.4 | 12.7 | 12.3 | 10.1 | 12.8 | 3.7 | 11.5 | 13.4 |
| **Time Interval (minutes): Δt (minutes)** | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 | 30 |
| **Radius of test hole: r (inches)** | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 | 4 |
| **Tested Infiltration Rate: It (inches/hour)** | 0.8 | 0.9 | 1.5 | 1.0 | 1.4 | 1.4 | 1.1 | 1.0 | 1.5 |

If the basin design considers infiltration, any fill placed within the basin should consist of open graded rock. The basin bottom should not be compacted during excavation or during placement of the open graded fill. The contractor should not track on or compact the basin bottom. If it is necessary for equipment to track in the basin bottom, the bottom should be scarified as the equipment moves out of the basin leaving loose, uncompacted soil in the basin bottom.

The basin will require maintenance consisting of periodic removal of fine soils and scarification of the bottom after equipment traffic.

# 8. CONCLUSIONS AND RECOMMENDATIONS

## 8.1 General

8.1.1 Soil or geologic conditions were not encountered during this or the 2004 studies that would preclude the proposed development of the project, provided the recommendations presented herein are followed and implemented during design and construction.

8.1.2 Potential geologic hazards at the site include seismic shaking and compressible near surface soils.

8.1.3 Based on our investigation and available geologic information, active, potentially active, or inactive faults are not present on or trending toward the site.

8.1.4 The undocumented fill, young alluvium, and upper portion of the older alluvium are not considered suitable for the support of engineered compacted fill or settlement-sensitive improvements. Remedial grading of the near surface soil will be required as discussed herein. The site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed.

8.1.5 Groundwater was not encountered during our work on the site. Although grading is not expected to extend to the depth of groundwater, seepage and perched groundwater conditions may be encountered during the grading operations along the bedrock/soil contact during the rainy seasons.

8.1.6 Cobbles and boulders were observed in the fill piles and oversized rock may be generated during grading of the site. Grading recommendations addressing oversize rock are provided herein.

8.1.7 Over excavation of cut and cut/fill lots will be required during grading.

8.1.8 Bedrock is expected to be marginally rippable to depths of 25 to 38 feet below existing grade at the seismic traverses depicted on Figure 2. Bedrock rippability should be evaluated in deeper cut areas and where deep utility excavations are planned. Over excavation of utility corridors during grading is recommended.

8.1.9 Proper surface drainage should be maintained to prevent ponding and saturation of the fill in pad and slope areas. Recommendations for site drainage are provided herein.

8.1.10 Once design or civil grading plans are made available, the recommendations within this report should be reviewed and revised, as necessary. Additionally, as the project design progresses toward a final design, changes in the design, location, or elevation of the proposed improvement should be reviewed by this office. Geocon should be contacted to evaluate the necessity for review and possible revision of this report.

## 8.2 Soil Characteristics

8.2.1 Based on the soil encountered in the field investigation in 2004, the site soils are expected to be "non-expansive" (Expansion Index [EI] less than 21) as defined by 2019 California Building Code (CBC) Section 1803.5.3. Table 8.2.1 presents soil classifications based on the expansion index.

**TABLE 8.2.1**
**SOIL CLASSIFICATION BASED ON EXPANSION INDEX**

| Expansion Index (EI) | Expansion Classification | 2019 CBC Expansion Classification |
|---|---|---|
| **0 – 20** | **Very Low** | **Non-Expansive** |
| 21 – 50 | Low | Expansive |
| 51 – 90 | Medium | |
| 91 – 130 | High | |
| Greater Than 130 | Very High | |

8.2.2 Additional testing for expansion potential should be performed during finish grading along with plasticity index testing on soils with expansion indices of more than 20.

8.2.3 Laboratory tests performed on samples of the site materials in 2004 indicate that the on-site materials possess a sulfate content of less than 0.000 percent equating to a S0 sulfate exposure to concrete structures as defined by 2019 CBC Section 1904.3 and ACI 318. Table 8.2.3 presents a summary of concrete requirements set forth by 2019 CBC Section 1904.3 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

**TABLE 8.2.3**
**REQUIREMENTS FOR CONCRETE**
**EXPOSED TO SULFATE-CONTAINING SOLUTIONS**

| Exposure Class | Water-Soluble Sulfate Percent by Weight | Cement Type | Maximum Water to Cement Ratio by Weight | Minimum Compressive Strength (psi) |
|---|---|---|---|---|
| **S0** | **0.00-0.10** | **--** | **--** | **2,500** |
| S1 | 0.10-0.20 | II | 0.50 | 4,000 |
| S2 | 0.20-2.00 | V | 0.45 | 4,500 |
| S3 | > 2.00 | V+ Pozzolan or Slag | 0.45 | 4,500 |

8.2.4 Geocon does not practice in the field of corrosion engineering. Therefore, further evaluation by a corrosion engineer may be performed if improvements that could be susceptible to corrosion are planned.

## 8.3 Grading

8.3.1 Grading should be performed in accordance with the *Recommended Grading Specifications* of *Appendix C* and the grading ordinances of the County of Riverside.

8.3.2 Prior to commencing grading, a preconstruction conference should be held at the site with the County Inspector, Owner or Developer, Grading Contractor, Civil Engineer, and Geotechnical Engineer in attendance. Special soil handling and/or the grading plans can be discussed at that time.

8.3.3 Site preparation should begin with the removal of deleterious material, debris and vegetation. The depth of removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site.

8.3.4 The upper approximately 3 to 11 feet of surficial soils should be removed to expose competent older alluvium or bedrock. Expected remedial removal depths are depicted on the *Geologic Map,* Figure 2. The actual depth of remedial grading should be evaluated by the Engineering Geologist during grading operations. The bottom of the excavations should be scarified to a depth of at least 1 foot, moisture conditioned at or slightly above optimum moisture content, and compacted to 90 percent of the maximum dry density (as determined by ASTM D1557), prior to fill placement.

8.3.5 In cut and cut/fill areas, the bedrock will require over-excavation. The bedrock over-excavation should result in a differential fill condition of H/3 or less, where H is the deepest fill depth within a 1:1 projection of the structure or a minimum depth of 3 feet, whichever is greater.

8.3.6 The site should be brought to finish grade elevations with engineered fill compacted in layers. Layers of fill should be no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density at or slightly above optimum moisture content (as determined by ASTM D1557). Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill.

8.3.7 The fill placed within 4 feet of proposed finish grade should possess a “low” expansion potential (EI of 50 or less), where practical.

8.3.8 Oversized rock (i.e. rock greater than 12-inches in maximum dimension) will be generated during grading operations. The oversize rock will require special handling and placement. Rocks greater than 3 inches in maximum dimensions should not be placed within utility trench backfill. Rocks greater than 6 inches in maximum dimension should not be placed in soil fill within the upper 3 feet of finish grade. Rocks 6 to 12 inches in maximum dimension should be placed deeper than 3 feet below finished grade elevations. Rocks 12 inches or larger in maximum dimension should be exported from the site or placed at least 10 feet below finished grade elevations, in accordance with the *Recommended Grading Specifications* of *Appendix D*.

8.3.9 Import fill (if necessary) should consist of granular materials with a “low” expansion potential (EI of 50 or less), generally free of deleterious material and rock fragments larger than 6 inches and should be compacted as recommended herein. Geocon should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to evaluate its suitability as fill material.

8.3.10 The outer 15 feet (or a distance equal to the height of the slope, whichever is less) of fill slopes should be composed of properly compacted granular “soil” fill to reduce the potential for surficial sloughing. In general, soil with an expansion index of 50 or less or at least 35 percent sand-size particles should be acceptable as “soil” fill. Soil of questionable strength to satisfy surficial stability should be tested in the laboratory for acceptable drained shear strength. The use of cohesionless soil in the outer portion of fill slopes should be avoided. Fill slopes should be overbuilt 2 feet and cut back or be compacted by backrolling with a loaded sheepsfoot roller at vertical intervals not to exceed 4 feet and should be track-walked at the completion of each slope such that the fill is compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content to the face of the finished sloped.

8.3.11 Finished slopes should be landscaped with drought-tolerant vegetation having variable root depths and requiring minimal landscape irrigation. In addition, the slopes should be drained and properly maintained to reduce erosion.

## 8.4 Earthwork Grading Factors

8.4.1 Estimates of shrinkage factors are based on empirical judgments comparing the material in its existing or natural state as encountered in the exploratory excavations to a compacted state. Variations in natural soil density and in compacted fill density render shrinkage value estimates very approximate. As an example, the contractor can compact the fill to a dry density of 90 percent or higher of the laboratory maximum dry density. Thus, the contractor has an approximately 10 percent range of control over the fill volume. Based on our experience and in-situ density test results with respect to maximum density/optimum moisture test results for the fill are expected to be 10 to 15 percent, the young alluvium is expected to shrink 10 to 15 percent, the older alluvium is expected to shrink 5 to 10 percent and the bedrock is expected to bulk 0 to 5 percent. This estimate is for preliminary quantity estimates only. Due to the variations in the actual shrinkage/bulking factors, a balance area should be provided to accommodate variations.

## 8.5 Utility Trench Backfill

8.5.1 Utility trenches should be properly backfilled in accordance with the requirements of the County of Riverside and the latest edition of the *Standard Specifications for Public Works Construction* (Greenbook). The pipes should be bedded with well-graded crushed rock or clean sand (Sand Equivalent greater than 30) to a depth of at least one foot over the pipe. If open graded rock is used it should be wrapped in filter fabric to prevent soil piping into the voids between the rock. The remainder of the trench backfill may be derived from onsite soil or approved import soil. Backfill of utility trenches should not contain rocks greater than 3 inches in diameter. The use of 2-sack slurry and controlled low strength material (CLSM) are also acceptable as backfill. However, consideration should be given to the possibility of differential settlement where the slurry ends and earthen backfill begins. These transitions should be minimized, and additional stabilization should be considered at these transitions.

8.5.2 Utility trench backfill should be placed in layers no thicker than will allow for adequate bonding and compaction. Utility backfill should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density and moisture conditioned at or slightly above optimum moisture content as determined by ASTM D1557. Backfill at the finish subgrade elevation of new pavements should be compacted to at least 95 percent of the maximum dry density. Backfill materials placed below the recommended moisture content may require additional moisture conditioning prior to placing additional fill.

## 8.6 Seismic Design Criteria

8.6.1 The following table summarizes site-specific design criteria obtained from the 2019 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. We used the computer program *Seismic Design Maps*, provided by the Structural Engineers Association (SEA) to calculate the seismic design parameters. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16. The values presented herein are for the risk-targeted maximum considered earthquake ($MCE_R$).

**TABLE 8.6.1A**
**2019 CBC SEISMIC DESIGN PARAMETERS FOR LOTS OVER BEDROCK**
**(EXPECTED WITHIN THE INTERIOR LOTS AT THE SITE)**

| Parameter | Value | 2019 CBC Reference |
|---|---|---|
| Site Class | C | Section 1613.2.2 |
| $MCE_R$ Ground Motion Spectral Response Acceleration – Class B (short), $S_S$ | 1.601g | Figure 1613.2.1(1) |
| $MCE_R$ Ground Motion Spectral Response Acceleration – Class B (1 sec), $S_1$ | 0.599g | Figure 1613.2.1(2) |
| Site Coefficient, $F_A$ | 1.2 | Table 1613.2.3(1) |
| Site Coefficient, $F_V$ | 1.401 | Table 1613.2.3(2) |
| Site Class Modified $MCE_R$ Spectral Response Acceleration (short), $S_{MS}$ | 1.921g | Section 1613.2.3 (Eqn 16-36) |
| Site Class Modified $MCE_R$ Spectral Response Acceleration – (1 sec), $S_{M1}$ | 0.839g | Section 1613.2.3 (Eqn 16-37) |
| 5% Damped Design Spectral Response Acceleration (short), $S_{DS}$ | 1.281g | Section 1613.2.4 (Eqn 16-38) |
| 5% Damped Design Spectral Response Acceleration (1 sec), $S_{D1}$ | 0.599g | Section 1613.2.4 (Eqn 16-39) |
| 33.7420 (latitude) / -117.0559 (longitude) | | |

**TABLE 8.6.1B**
**2019 CBC SEISMIC DESIGN PARAMETERS FOR LOTS OVER OLDER ALLUVIUM (EXPECTED ALONG THE EASTERN AND WESTERN SIDES OF THE SITE AND NORTH OF STREET 'B')**

| Parameter | Value | 2019 CBC Reference |
|---|---|---|
| Site Class | D | Section 1613.2.2 |
| $MCE_R$ Ground Motion Spectral Response Acceleration – Class B (short), $S_S$ | 1.5g | Figure 1613.2.1(1) |
| $MCE_R$ Ground Motion Spectral Response Acceleration – Class B (1 sec), $S_1$ | 0.6g | Figure 1613.2.1(2) |
| Site Coefficient, $F_A$ | 1.2 | Table 1613.2.3(1) |
| Site Coefficient, $F_V$ | 1.7* | Table 1613.2.3(2) |
| Site Class Modified $MCE_R$ Spectral Response Acceleration (short), $S_{MS}$ | 1.8g | Section 1613.2.3 (Eqn 16-36) |
| Site Class Modified $MCE_R$ Spectral Response Acceleration – (1 sec), $S_{M1}$ | 1.02g* | Section 1613.2.3 (Eqn 16-37) |
| 5% Damped Design Spectral Response Acceleration (short), $S_{DS}$ | 1.2g | Section 1613.2.4 (Eqn 16-38) |
| 5% Damped Design Spectral Response Acceleration (1 sec), $S_{D1}$ | 0.68g* | Section 1613.2.4 (Eqn 16-39) |
| 33.7420 (latitude) / -117.0559 (longitude) | | |

*See ASCE 7-16 Section 11.4.7

8.6.2 The table below presents the mapped maximum considered geometric mean ($MCE_G$) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

**TABLE 8.6.2**
**ASCE 7-16 PEAK GROUND ACCELERATION**

| Parameter | Value | ASCE 7-10 Reference |
|---|---|---|
| Mapped $MCE_G$ Peak Ground Acceleration, PGA | 0.614 | Figure 22-7 |
| Site Coefficient, $F_{PGA}$ | 1.2g | Table 11.8-1 |
| Site Class Modified $MCE_G$ Peak Ground Acceleration, $PGA_M$ | 0.737g | Section 11.8.3 (Eqn 11.8-1) |

8.6.3 The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2019 California Building Code and ASCE 7-16, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building code is to maintain "Life Safety" during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

8.6.4 Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 8.1 magnitude event occurring at a hypo central distance of 9.52 kilometers from the site.

8.6.5 Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 8.1 magnitude occurring at a hypocentral distance of 9.52 kilometers from the site.

8.6.6 Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

## 8.7 Foundation and Concrete Slabs-On-Grade Recommendations

8.7.1 The foundation recommendations presented herein are for proposed one- to two-story buildings subsequent to the recommended grading. Future buildings are expected to be supported on conventional shallow foundations with concrete slabs-on-grade deriving support in newly placed engineered fill.

8.7.2 The foundation recommendations presented herein are for the proposed structures following remedial grading. We separated the foundation recommendations into three categories based on either the maximum and differential fill thickness or Expansion Index. We expect the over-excavated bedrock lots will be Category I, and the fill over alluvium and cut/fill transition lots will be Category II. However, the category may be increased to II or III where expansion potential or fill geometry dictates. The foundation category criteria for the expected conditions are presented in Table 8.7.2. Final foundation categories will be evaluated once site grading has been completed.

**TABLE 8.7.2**
**FOUNDATION CATEGORY CRITERIA**

| Foundation Category | Maximum Fill Thickness, T (Feet) | Differential Fill Thickness, D (Feet) | Expansion Index (EI) |
|---|---|---|---|
| **I – Over Bedrock** | **T<20** | **D<10** | **EI≤50** |
| **II – Over Alluvium & Cut Fill Transition Lots** | **20≤T<50** | **10≤D<20** | **50<EI≤90** |
| III | T≥50 | D≥20 | EI>90 |

8.7.3 Table 8.7.3 presents minimum foundation and interior concrete slab design criteria for conventional shallow foundation systems.

**TABLE 8.7.3**
**CONVENTIONAL FOUNDATION RECOMMENDATIONS BY CATEGORY**

| Foundation Category | Minimum Footing Embedment Depth (inches) | Continuous Footing Reinforcement | Interior Slab Reinforcement |
|---|---|---|---|
| **I** | **12** | **Two No. 4 bars, one top and one bottom** | **No. 3 bars at 24 inches on center, both directions** |
| **II** | **18** | **Four No. 4 bars, two top and two bottom** | |
| III | 24 | Four No. 5 bars, two top and two bottom | No. 3 bars at 18 inches on center, both directions |

8.7.4 The embedment depths presented in Table 8.7.3 should be measured from the lowest adjacent pad grade for both interior and exterior footings. A wall/column footing dimension detail depicting the depth to lowest adjacent grade is provided on Figure 3. The conventional foundations should have a minimum width of 12 inches and 24 inches for continuous and isolated footings, respectively.

8.7.5 The concrete slab-on-grade should be a minimum of 4 inches thick for Foundation Categories I and II and 5 inches thick for Foundation Category III.

8.7.6 Slabs-on-grade that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06) and should be installed in general conformance with ASTM E1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, the concrete slab-on-grade may be underlain by a vapor retarder over 4 inches of clean sand (sand equivalent greater than 30), since the sand will serve as a capillary break and will minimize the potential for punctures and damage to the vapor barrier.

8.7.7 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. However, we should be contacted to provide recommendations if the bedding sand is thicker than 4 inches. Placement of 3 inches and 4 inches of sand is common practice in southern California for 5-inch and 4-inch thick slabs, respectively. The foundation engineer should provide appropriate concrete mix design criteria and curing measures that may be utilized to assure proper curing of the slab to reduce the potential for rapid moisture loss and subsequent cracking and/or slab curl.

8.7.8 As an alternative to the conventional foundation recommendations, consideration should be given to the use of post-tensioned concrete slab and foundation systems for the support of the proposed structures. The post-tensioned systems should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI) DC 10.5-12 *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils* or *WRI/CRSI Design of Slab-on-Ground Foundations*, as required by the 2019 California Building Code (CBC Section 1808.6.2). Although this procedure was developed for expansive soil conditions, it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented in Table 8.7.8 for the Foundation Category designated. The parameters presented in Table 8.7.8 are based on the guidelines presented in the PTI DC 10.5 design manual.

**TABLE 8.7.8**
**POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS**

| Post-Tensioning Institute (PTI) DC 10.5-12 Design Parameters | Foundation Category | | |
|---|---|---|---|
| | I | II | III |
| 1. Thornthwaite Index | -20 | -20 | -20 |
| 2. Equilibrium Suction | 3.9 | 3.9 | 3.9 |
| 3. Edge Lift Moisture Variation Distance, $e_M$ (Feet) | 5.3 | 5.1 | 4.9 |
| 4. Edge Lift, $y_M$ (Inches) | 0.61 | 1.10 | 1.58 |
| 5. Center Lift Moisture Variation Distance, $e_M$ (Feet) | 9.0 | 9.0 | 9.0 |
| 6. Center Lift, $y_M$ (Inches) | 0.30 | 0.47 | 0.66 |

8.7.9 The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer. If a post-tensioned mat foundation system is planned, the slab should possess a thickened edge with a minimum width of 12 inches and extend below the clean sand or crushed rock layer.

8.7.10 If the structural engineer proposes a post-tensioned foundation design method other than the 2019 CBC:

- The deflection criteria presented in Table 8.7.8 are still applicable.
- Interior stiffener beams should be used for Foundation Categories II and III.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 12 inches, 18 inches and 24 inches for foundation categories I, II, and III, respectively. The embedment depths should be measured from the lowest adjacent pad grade.

8.7.11 Our experience indicates post-tensioned slabs may be susceptible to excessive edge lift, regardless of the underlying soil conditions. Placing reinforcing steel at the bottom of the perimeter footings and the interior stiffener beams may mitigate this potential. The structural engineer should design the foundation system to reduce the potential of edge lift occurring for the proposed structures.

8.7.12 During the construction of the post-tension foundation system, the concrete should be placed monolithically. Under no circumstances should cold joints form between the footings/grade beams and the slab during the construction of the post-tension foundation system unless designed by the structural engineer.

8.7.13 Category I, II, or III foundations may be designed for an allowable soil bearing pressure of 2,000 pounds per square foot (psf) (dead plus live load) for the site. This bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.

8.7.14 The maximum expected static settlement for the planned structures, supported on conventional foundation systems with the above allowable bearing pressures, and deriving support in engineered fill is estimated to be 1 inch and to occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement is not expected to exceed ½ inch over a horizontal distance of 40 feet.

8.7.15 Isolated footings outside of the slab area, if present, should have the minimum embedment depth and width recommended for conventional foundations for a Foundation Category. The use of isolated footings, which are located beyond the perimeter of the building and support structural elements connected to the building, are not recommended for Category III. Where this condition cannot be avoided, the isolated footings should be connected to the building foundation system with grade beams. In addition, consideration should be given to connecting patio slabs that exceed 5 feet in width to the building foundation, to reduce the potential for future separation to occur.

8.7.16 Interior stiffening beams should be incorporated into the design of the foundation system in accordance with the PTI design procedures.

8.7.17 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisture conditioned at or slightly above optimum moisture content.

8.7.18 Where buildings or other improvements are planned near the top of a slope 3:1 (horizontal:vertical) or steeper, special foundation and/or design considerations are recommended due to the tendency for lateral soil movement to occur.

- When located next to a descending 3:1 (horizontal:vertical) or steeper fill slope or cut slope, the foundations should be extended to a depth where the minimum horizontal distance is equal to H/3 (where H equals the vertical distance from the top of the fill slope to the base of the fill soil) with a minimum of 7 feet, but need not exceed 40 feet. The horizontal distance is measured from the outer, deepest edge of the footing to the face of the slope. A post-tensioned slab and foundation system or mat foundation system can be used to reduce the potential for distress in the structures associated with strain softening and lateral fill extension. Specific design parameters or recommendations for either of these alternatives can be provided once the building location and fill slope geometry have been determined.

- Swimming pools located within 7 feet of the top of cut or fill slopes are not recommended. Where such a condition cannot be avoided, the portion of the swimming pool wall within 7 feet of the slope face be designed assuming that the adjacent soil provides no lateral support. This recommendation applies to fill slopes up to 30 feet in height and cut slopes regardless of height. For swimming pools located near the top of fill slopes greater than 30 feet in height, additional recommendations may be required and Geocon should be contacted for a review of specific site conditions.

- Although other improvements, which are relatively rigid or brittle, such as concrete flatwork or masonry walls, may experience some distress if located near the top of a slope, it is generally not economical to mitigate this potential. It may be possible, however, to incorporate design measures which would permit some lateral soil movement without causing extensive distress. Geocon should be consulted for specific recommendations.

8.7.19 The recommendations of this report are intended to reduce the potential for cracking of slabs and foundations due to expansive soil (if present), differential settlement of fill soil with varying thicknesses. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

8.7.20 Concrete slabs should be provided with adequate crack-control joints, construction joints and/or expansion joints to reduce unsightly shrinkage cracking. The design of joints should consider criteria of the American Concrete Institute when establishing crack-control spacing. Additional steel reinforcing, concrete admixtures and/or closer crack control joint spacing should be considered where concrete-exposed finished floors are planned.

8.7.21 Geocon should be consulted to provide additional design parameters as required by the structural engineer.

## 8.8 Exterior Concrete Flatwork

8.8.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein assuming the subgrade materials possess an Expansion Index of 50 or less. Subgrade soils should be compacted to 90 percent relative compaction at or slightly above optimum moisture content. Slab panels should be a minimum of 4 inches thick and when in excess of 8 feet square should be reinforced with No. 3 reinforcing bars spaced 18 inches center-to-center in both directions to reduce the potential for cracking. In addition, concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the *Grading* section prior to concrete placement. Subgrade soil should be properly compacted, and the moisture content of subgrade soil should be verified prior to placing concrete. Base materials will not be required below concrete improvements.

8.8.2 Even with the incorporation of the recommendations of this report, the exterior concrete flatwork has a potential to experience some uplift due to expansive soil beneath grade. The steel reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

8.8.3 Where exterior flatwork abuts the structure at entrant or exit points, the exterior slab should be dowelled into the structure's foundation stem wall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer.

8.8.4 The recommendations presented herein are intended to reduce the potential for cracking of exterior slabs as a result of differential movement. However, even with the incorporation of the recommendations presented herein, slabs-on-grade will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Crack control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

## 8.9 Conventional Retaining Walls

8.9.1 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 10 feet. In the event that walls higher than 10 feet or other types of walls are planned, Geocon should be consulted for additional recommendations.

8.9.2 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 30 pounds per cubic foot (pcf). Where the backfill will be inclined at no steeper than 2:1 (horizontal to vertical), an active soil pressure of 55 pcf is recommended. These soil pressures expect that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall possess an EI of 50 or less. For walls where backfill materials do not conform to the criteria herein, Geocon should be consulted for additional recommendations.

8.9.3 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls with a level backfill surface should be designed for a soil pressure equivalent to the pressure exerted by a fluid density of 50 pcf.

8.9.4 The structural engineer should determine the seismic design category for the project in accordance with 2019 CBC. If the project possesses a seismic design category of D, E, or F, proposed retaining walls in excess of 6 feet in height should be designed with seismic lateral pressure (2019 CBC).

8.9.5 An incremental seismic load of 10 pcf should be used for design of walls with level backfill in accordance with 2019 CBC. The pressure should be taken as an inverted triangular distribution with the zero-pressure point at the toe of the wall and 20 H (psf where H in feet) at the top of the wall, where H is the wall height in feet. The point of application of the dynamic thrust may be taken at 0.6H above the toe of the wall. This seismic load should be applied in addition to the active earth pressure. The earth pressure is based on half of two-thirds of $PGA_M$ calculated from ASCE 7-16.

8.9.6 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.

8.9.7 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and waterproofed as required by the project architect. The soil immediately adjacent to the backfilled retaining wall should be composed of free draining material completely wrapped in Mirafi 140N (or equivalent) filter fabric for a lateral distance of 1 foot for the bottom two-thirds of the height of the retaining wall. The upper one-third should be backfilled with less permeable compacted fill to reduce water infiltration. Alternatively, a drainage panel, such as a Miradrain 6000 or equivalent, can be placed along the back of the wall. A typical drain detail for each option is shown on Figure 4. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein expect a properly compacted backfill (EI of 20 or less) with no hydrostatic forces or imposed surcharge load. If conditions different than those described are expected or if specific drainage details are desired, Geocon should be contacted for additional recommendations.

8.9.8 Wall foundations should be designed in accordance with the above foundation recommendations.

## 8.10 Lateral Loading

8.10.1 To resist lateral loads, a passive pressure exerted by an equivalent fluid weight of 350 pounds per cubic foot (pcf) should be used for the design of footings or shear keys poured neat against compacted fill. The allowable passive pressure expects a horizontal surface extending at least 5 feet or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.

8.10.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.40 should be used for design.

## 8.11 Swimming Pool/Spa

8.11.1 If swimming pools or spas are planned, the proposed swimming pool shell bottom should be designed as a free-standing structure and may derive support in newly placed engineered fill or competent gabbroic bedrock. We recommend that uniformity be maintained beneath the proposed swimming pools where possible to reduce the potential for differential settlement; however, swimming pool foundations may derive support in both engineered fill and gabbroic bedrock, as necessary

8.11.2 Swimming pool foundations and walls may be designed in accordance with the *Foundation* (see Section 8.7) and *Retaining Wall* (see Section 8.9) sections of this report.

8.11.3 Surface drainage around the pool/spa should be designed to prevent water from ponding and seeping into the ground. Surface water should be collected and conducted through non-erosive devices to the street, storm drain or other approved water course or disposal area. Leakage from the proposed pool/spa could create an artificial groundwater condition that will likely create instability problems. Therefore, all plumbing and the pool/spa should be leak free.

8.11.4 The deck for the swimming pool/spa should be cast separately of the swimming pool/spa, and water stops should be provided between the bond beam and the deck. Jointing for concrete flatwork should be provided in accordance with the recommendations of the American Concrete Institute. The joints should be sealed with an approved flexible sealant to reduce the potential for introduction of surface water into the underlying soil.

8.11.5 Consideration should be given to installing a subdrain system for the pool area. The subgrade surface should be graded to slope a minimum of 1 percent away from the pool. An impermeable liner (e.g. High-density polyethylene, HDPE, with a thickness of about 30 mil or equivalent PVC liner) could be placed over the subgrade soil. The liner, if installed, should overlap by at least 12 inches and sealed in accordance with manufacturer's recommendations.

8.11.6 To mitigate the potential for moisture infiltration into the subgrade soils beneath the pool deck, we recommend the construction of a deepened footing along the outside edge of the pool deck flatwork.

8.11.7 A subdrain consisting of 4-inch diameter perforated PVC pipe should be installed inside the deepened footing and sloped to drain into an approved outlet. The pipe should be surrounded by ¾ inch open-graded gravel and wrapped with filter fabric.

8.11.8 If the proposed pool is in proximity to a proposed building, consideration should be given to construction sequence. If the proposed pool is constructed after building foundation construction, the excavation required for pool construction could remove a component of lateral support from the foundations and would therefore require shoring. Once information regarding the pool location and depth becomes available, this information should be provided to Geocon for review and possible revision of these recommendations.

## 8.12 Preliminary Pavement Recommendations

8.12.1 The final pavement design should be based on R-value testing of soils at road subgrade elevation. Roadways should be designed in accordance with the County of Riverside Transportation Department *Road Improvement Standards & Specifications, Ordinance No. 461* (2007), when final Traffic Indices (TI) and R-Value test results of subgrade soils are completed. For preliminary design purposes, we used an expectd R-value of 30. A value of 78 was considered for aggregate base materials for the purposes of this preliminary analysis. Pavements should meet the minimum requirement for pavement thickness per County of Riverside *Ordinance No. 461* (2007). Preliminary flexible pavement sections are presented in Table 8.12.1. Geocon should be contacted if other roadway classifications and traffic indices are appropriate for the project.

**TABLE 8.12.1**
**PRELIMINARY FLEXIBLE PAVEMENT SECTIONS**

| Road Classification | Expectd Traffic Index | Expectd Subgrade R-Value | Asphalt Concrete (inches) | Crushed Aggregate Base (inches) |
|---|---|---|---|---|
| Local Street/Access Road | 5.5 | 30 | 3.5 | 6.0 |
| Enhanced Local Street at School or Park | 6.5 | 30 | 4.0 | 8.0 |
| Collector | 7.0 | 30 | 4.0 | 9.5 |
| Industrial Collector | 8.0 | 30 | 5.0 | 10.5 |
| Secondary Highway | 8.5 | 30 | 5.5 | 11.5 |

8.12.2 The upper 12 inches of the roadway subgrade soil should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density at or slightly above optimum moisture content (as determined by ASTM D1557).

8.12.3 The crushed aggregated base and asphalt concrete materials should conform to Section 200-2.2 and Section 203-6, respectively, of the latest edition of the California *Greenbook* and County of Riverside *Ordinance No. 461* (2007). Base materials should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near or slightly above optimum moisture content (as determined by ASTM D1557). Asphalt concrete should be compacted to a density of 95 percent of the laboratory Hveem density (as determined by ASTM D1561).

8.12.4 A rigid Portland cement concrete (PCC) pavement section should be placed in driveway aprons and cross gutters. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 *Guide for Design and Construction of Concrete Parking Lots* using the parameters presented in Table 8.12.4.

**TABLE 8.12.4**
**RIGID PAVEMENT DESIGN PARAMETERS**

| Design Parameter | Design Value |
|---|---|
| Modulus of subgrade reaction, k | 100 pci |
| Modulus of rupture for concrete, $M_R$ | 500 psi |
| Traffic Category, TC | B and C |
| Average daily truck traffic, ADTT | 300 |

8.12.5 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 8.12.5.

**TABLE 8.12.5**
**RIGID PAVEMENT RECOMMENDATIONS**

| Location | Portland Cement Concrete (inches) |
|---|---|
| Roadways (TC = C) | 7.0 |
| Bus Stops (TC = D) | 7.5 |

8.12.6 The PCC pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near or slightly above optimum moisture content (as determined by ASTM D1557). This pavement section is based on a minimum concrete compressive strength of approximately 3,500 psi (pounds per square inch). Base material will not be required beneath concrete improvements.

8.12.7 A thickened edge or integral curb should be constructed on the outside of concrete slabs subjected to wheel loads. The thickened edge should be 1.2 times the slab thickness or a minimum thickness of 2 inches, whichever results in a thicker edge, and taper back to the recommended slab thickness 4 feet behind the face of the slab (e.g., a 9-inch-thick slab would have an 11-inch-thick edge). Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

8.12.8 In order to control the location and spread of concrete shrinkage cracks, crack-control joints (weakened plane joints) should be included in the design of the concrete pavement slab in accordance with the referenced ACI report.

8.12.9 Performance of the pavements is highly dependent on providing positive surface drainage away from the edge of the pavement. Ponding of water on or adjacent to the pavement surfaces will likely result in pavement distress and subgrade failure. Drainage from landscaped areas should be directed to controlled drainage structures. Landscape areas adjacent to the edge of asphalt pavements are not recommended due to the potential for surface or irrigation water to infiltrate the underlying permeable aggregate base and cause distress. Where such a condition cannot be avoided, consideration should be given to incorporating measures that will significantly reduce the potential for subsurface water migration into the aggregate base. If planter islands are planned, a perimeter curb or the use of an impermeable geosynthetic should be considered and extend at least 6 inches below the bottom level of the base materials.

## 8.13 Temporary Excavations

8.13.1 Excavations of up to 25 feet below the existing ground surface are expected for construction of the proposed utility improvements; and we expect that the proposed utilities will be installed with conventional trench cut-and-cover methods.

8.13.2 The excavations are expected to expose newly placed engineered fill, alluvium and/or gabbroic bedrock which are suitable for vertical excavations up to 5 feet where loose soils or caving sands are not present and where not surcharged by adjacent traffic or structures.

8.13.3 Vertical excavations greater than 5 feet will require sloping measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments should be designed by the contractor's competent person in accordance with OSHA regulations.

8.13.4 Where there is insufficient space for sloped excavations, shoring or trench shields should be used to support excavations. Shoring may also be necessary where sloped excavation could remove vertical or lateral support of existing improvements, including existing utilities and adjacent structures. Recommendations for temporary shoring are provided in the following section.

8.13.5 Where sloped embankments are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The contractor's competent person should inspect the soils exposed in the cut slopes during excavation in accordance with OSHA regulations so that modifications of the slopes can be made if variations in the soil conditions occur.

## 8.14 Shoring

8.14.1 Where there is insufficient space to safely perform sloped excavations, shoring should be implemented. We expect that braced shoring, such as conventionally braced shields or cross-braced hydraulic shoring, will be utilized; however, the selection of the shoring system is the responsibility of the contractor. Shoring systems should be designed by a California licensed civil or structural engineer with experience in designing shoring systems.

8.14.2 We recommend that an equivalent fluid pressure based on the table below, be utilized for design of shoring. These pressures are based on the assumption that there are no hydrostatic pressures above the bottom of the excavation.

**TABLE 8.14.2**
**RECOMMENDED SHORING PRESURES**

| HEIGHT OF SHORED EXCAVATION (FEET) | EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (Active Pressure) | EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (Active Pressure with 2:1 Slope) | EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot) (AT-REST PRESSURE) |
|---|---|---|---|
| Up to 15 | 25 | 50 | 45 |

8.14.3 Active pressures can only be achieved when movement in the soil (earth wall) occurs. If movement in the soil is not acceptable, such as adjacent to an existing structure or where braced shoring will be utilized the at-rest pressure should be considered for design purposes.

8.14.4 Additional active pressure should be added for a surcharge condition due to sloping ground, construction equipment, vehicular traffic, or adjacent structures and should be designed for each condition as the project progresses.

8.14.5 In addition to the recommended earth pressure, the upper 15 feet of the shoring adjacent to roadways or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an expectd 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 15 feet from the shoring, the traffic surcharge may be neglected. Higher surcharge loads may be required to account for construction equipment.

8.14.6 It is difficult to accurately predict the amount of deflection of a shored embankment, but some deflection will occur. We recommend that the deflection be minimized to prevent damage to existing structures and adjacent improvements. Where public right-of-ways are present or adjacent offsite structures do not surcharge the shoring excavation, the shoring deflection should be limited to less than 1 inch at the top of the shored embankment. Where offsite structures are within the shoring surcharge area it is recommended that the beam deflection be limited to less than ½ inch at the elevation of the adjacent offsite foundation, and no deflection at all if deflections will damage existing structures. The allowable deflection is dependent on many factors, such as the presence of structures and utilities near the top of the embankment and will be assessed and designed by the project shoring engineer.

## 8.15 Site Drainage and Moisture Protection

8.15.1 Adequate site drainage is critical to reduce the potential for differential soil movement, erosion and subsurface seepage. Under no circumstances should water be allowed to pond adjacent to footings. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2019 CBC 1804.4 or other applicable standards. In addition, surface drainage should be directed away from the top of slopes into swales or other controlled drainage devices. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

8.15.2 Storm water infiltration systems should be offset a minimum of 20 feet from the outside edge of structural foundations.

8.15.3 Underground utilities should be leak free. Utility and irrigation lines should be checked periodically for leaks, and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

8.15.4 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. We recommend that area drains to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes be used. In addition, where landscaping is planned adjacent to the pavement, we recommend construction of a cutoff wall or the use of an impermeable geosynthetic along the edge of the pavement that extends at least 6 inches below the bottom of the base material.

8.15.5 If not properly constructed, there is a potential for distress to improvements and properties located hydrologically down gradient or adjacent to infiltration areas. Factors such as the amount of water to be detained, its residence time, and soil permeability have an important effect on seepage transmission and the potential adverse impacts that may occur if the storm water management features are not properly designed and constructed. We have not performed a hydrogeology study at the site. Downgradient and adjacent structures may be subjected to seeps, movement of foundations and slabs, or other impacts as a result of water infiltration.

## 8.16 Plan Review

8.16.1 Grading and foundation plans should be reviewed by the Geotechnical Engineer of Record prior to finalization of design to verify that the plans have been prepared in substantial conformance with the recommendations of this report, and to provide additional analyses or recommendations, if necessary.

## LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in this and Geocon's 2004 investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that expected herein, Geocon West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous materials was not part of the scope of services provided by Geocon West, Inc.

This report is issued with the understanding that it is the responsibility of the owner, or of their representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to expect the responsibilities of project Geotechnical Engineer of Record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to expect the role of Geotechnical Engineer of Record.

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16. Office of Statewide Health Planning and Development (OSHPD), *Seismic Design Maps* website: https://seismicmaps.org/; accessed October 2020.

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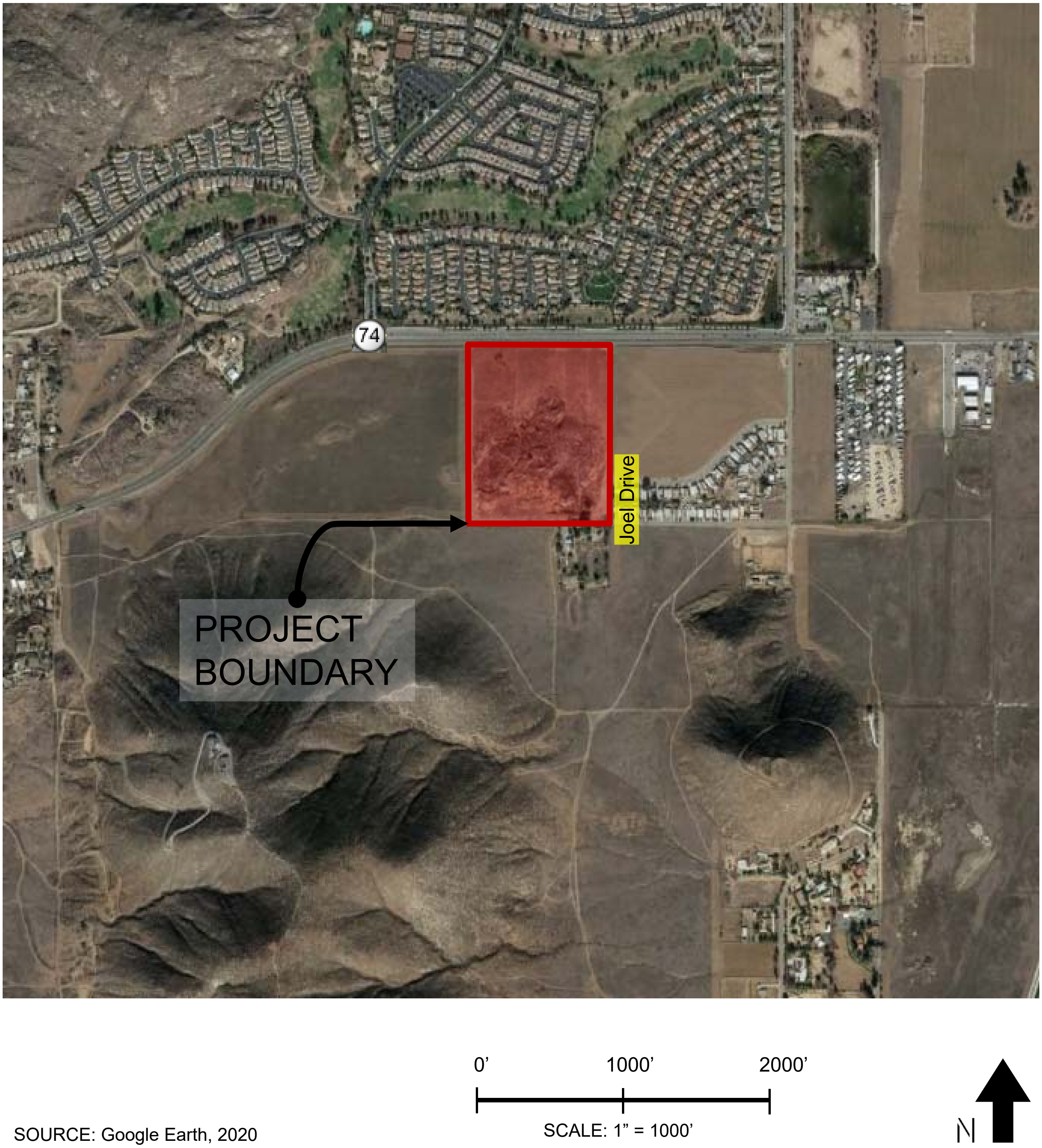


SOURCE: Google Earth, 2020

VICINITY MAP

GEOCON
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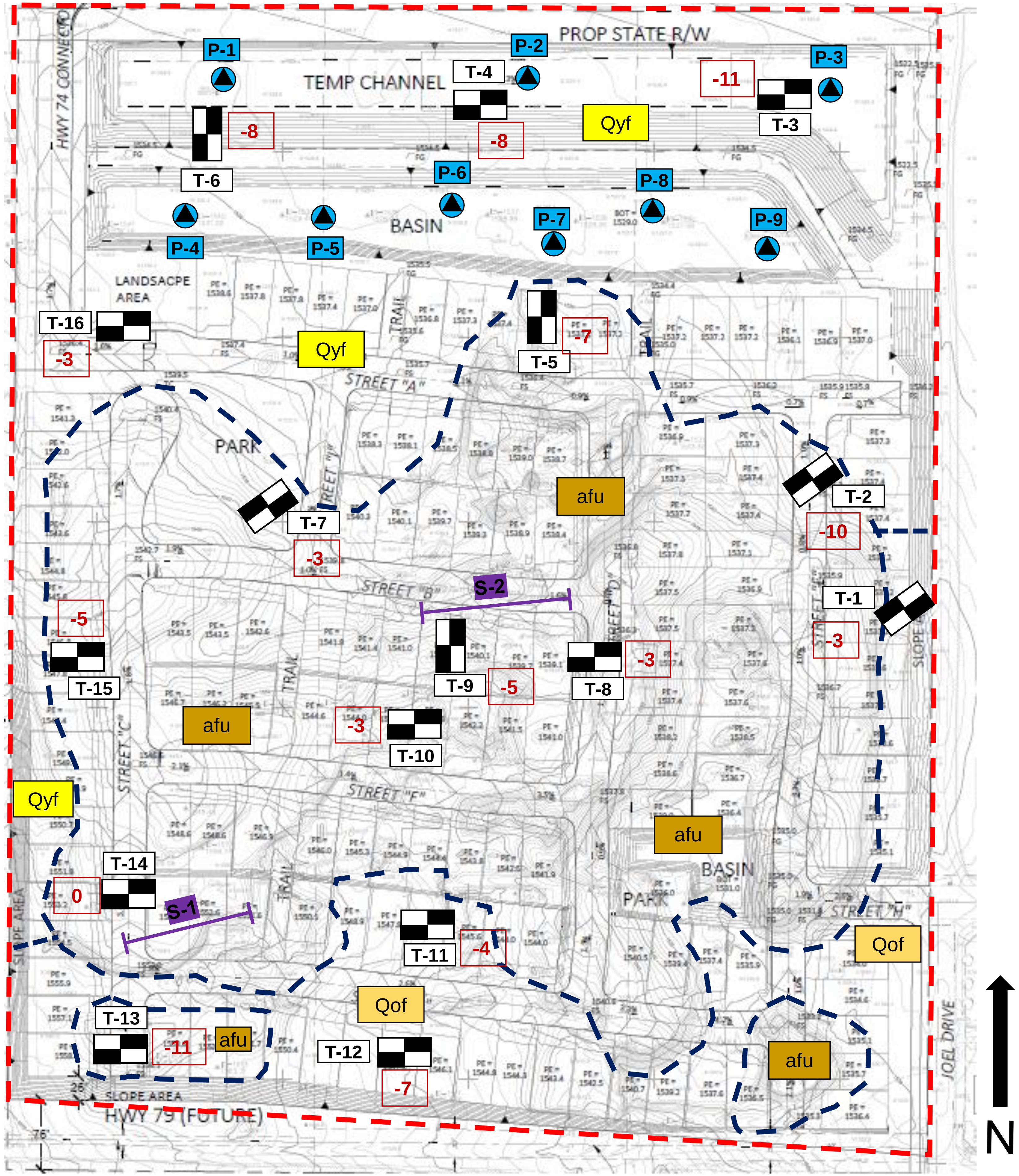
RIVANI HEMET 30
SOUTH OF HIGHWAY 74 & WEST OF JOEL DRIVE
HEMET AREA
RIVERSIDE COUNTY, CALIFORNIA

| | | | | | |
|---|---|---|---|---|---|
| LCW | | | DECEMBER 2020 | PROJECT NO. T2214-22-02 | FIG. 1 |




## GEOCON LEGEND

Locations are approximate

....... PROJECT EXTENTS

P-9 ....... PERCOLATION TEST LOCATION

T-12 .......TEST PIT LOCATION

S-2 ....... SEISMIC REFRACTIONTRANVERSE

afu ....... UNDOCUMENTED FILL

Qyf ....... YOUNG ALLUVIAL FAN DEPOSITS

Qof ....... OLD ALLUVIAL FAN DEPOSITS

Khg ....... GRANODIORITE

Kgab ....... GREEN ACRES GABBRO

....... GEOLOGIC CONTACT

-11 ....... ESTIMATED REMEDIAL REMOVAL DEPTH

Source: Anderson Consulting Engineers, Inc., TTM Grading Exhibit, undated.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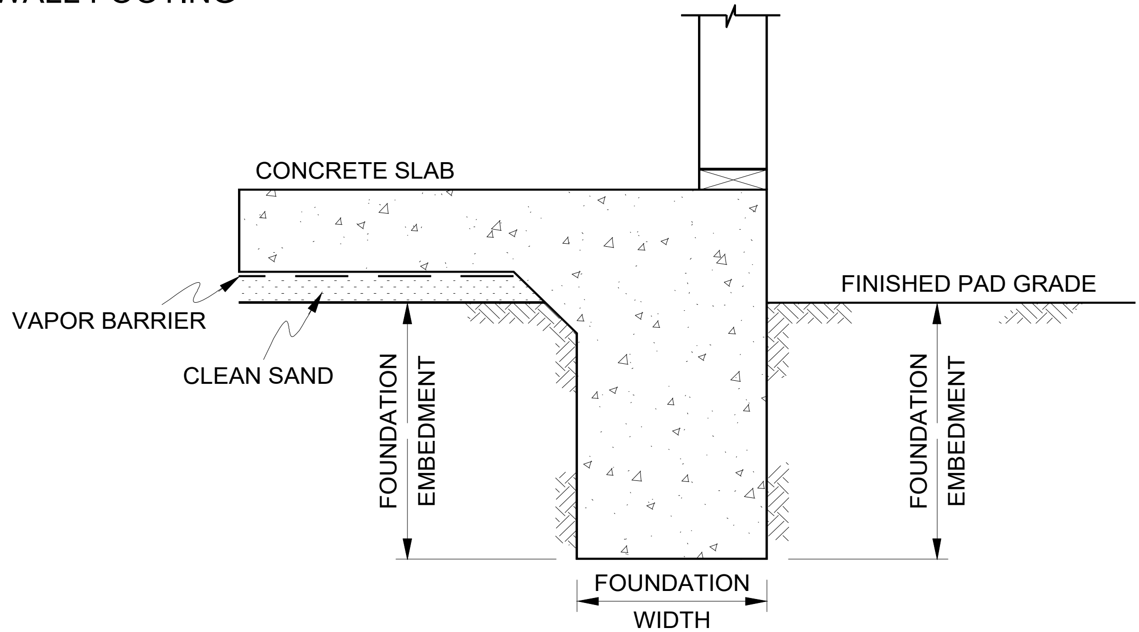
GEOTECHNICAL MAP

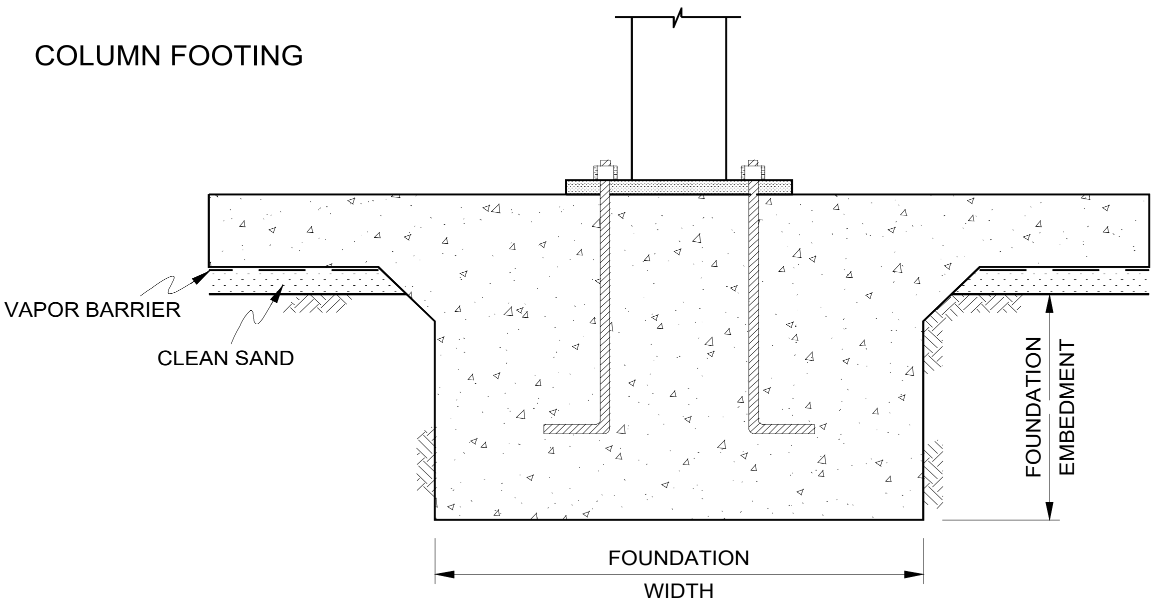
RIVANI HEMET 30
SOUTH OF HIGHWAY 74 & WEST OF JOEL DRIVE
HEMET AREA
RIVERSIDE COUNTY, CALIFORNIA

| LCW | | | DECEMBER 2020 | PROJECT NO. T2214-22-02 | FIG. 2 |
|---|---|---|---|---|---|

WALL FOOTING




COLUMN FOOTING




NOTE: SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE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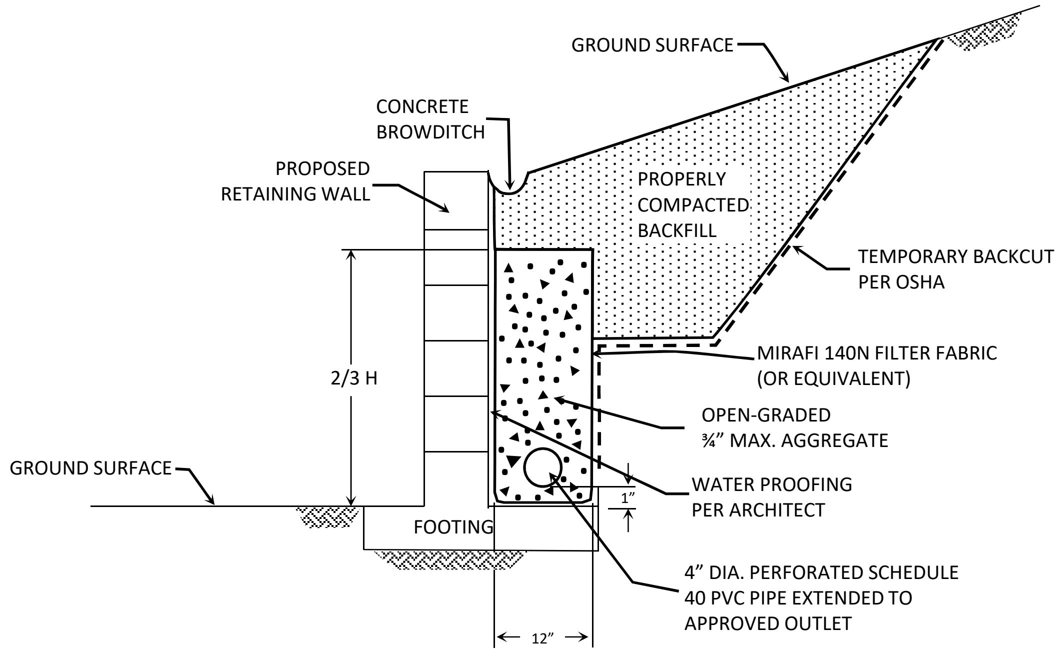
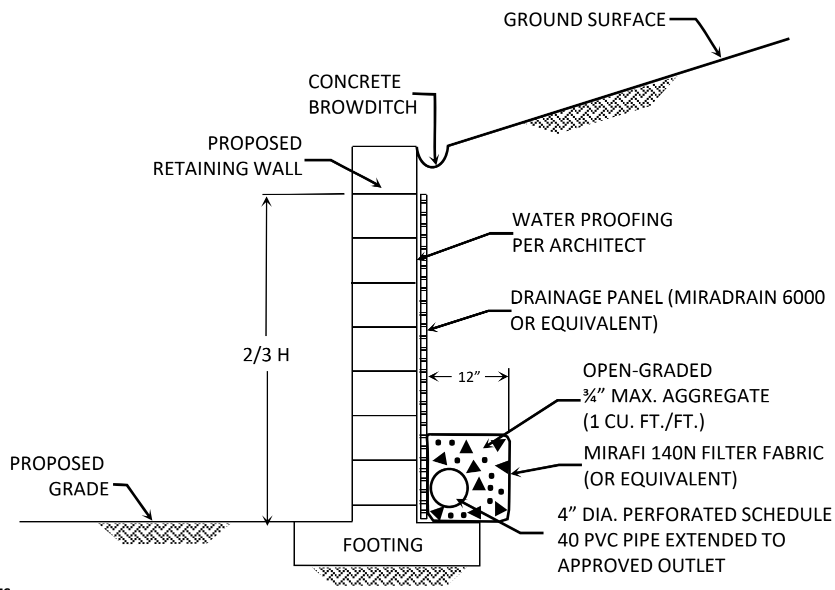
| LCW | | |
|---|---|---|

# WALL / COLUMN FOOTING DETAIL

RIVANI HEMET 30
SOUTH OF HIGHWAY 74 & WEST OF JOEL DRIVE
HEMET AREA
RIVERSIDE COUNTY, CALIFORNIA

| DECEMBER 2020 | PROJECT NO. T2214-22-02 | FIG. 3 |
|---|---|---|




**NOTES:**

DRAIN SHOULD BE UNFORMLY SLOPED TO GRAVITY OUTLET OR TO A SUMP WHERE WATER CAN BE REMOVED BY PUMPIMG

CONCRETE BROW DITCH RECOMMENDED FOR SLOPE HEIGHTS GREATER THAN 6 FEET

NO SCALE

# TYPICAL RETAINING WALL DRAIN DETAIL

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HEMET AREA
RIVERSIDE COUNTY, CALIFORNIA

| | | | | | |
|---|---|---|---|---|---|
| LCW | | | DECEMBER 2020 | PROJECT NO. T2214-22-02 | FIG. 4 |

# APPENDIX A

# APPENDIX A

## EXPLORATORY EXCAVATIONS

Geocon hand excavated percolation test holes and performed percolation testing on the site on November 9 through 11, 2020. The percolation test borings were excavated to depths of approximately 1 to 3 feet deep within the proposed basin and temporary channel areas in the northern area of the property. Percolation testing was performed in accordance with Riverside County Flood Control and Water Conservation Districts LID BMP Handbook. We collected soil samples at the depth of percolation testing in each test hole. Percolation test locations were manually backfilled with soil cuttings when the tests were complete. The percolation test pit logs are presented on Figures A-17 through A-25. The percolation test data is presented on Figures A-26 through A-34. The locations of the percolation tests are presented on *Figure 2*.

The test pit and seismic refraction logs from Geocon's 2004 investigation are also presented herein for ease of reference and their locations are depicted on *Figure 2*.

**TABLE A-I**
**SEISMIC TRAVERSE RESULTS**

| Seismic Traverse No. | V1 (fps) | V2 (fps) | V3 (fps) | D1 (feet) | D2 (feet) | D3 (feet) | Anticipated Maximum Excavation in feet |
|---|---|---|---|---|---|---|---|
| S-1 | 1600 | 3305 | 10,365 | 2-5 | 38-53 | 38+ | 20 |
| S-2 | 1520 | 4260 | 12,975 | 2-5 | 25-33 | 25+ | 20 |

$V_1$ = Seismic compression wave velocity in feet per second of first layer of materials
$V_2$ = Second layer velocities
$V_3$ = Third layer velocities
$D_1$ = Depth in feet to base of first layer
$D_2$ = Depth to base of second layer
$D_3$ = Depth to base of third layer
fps = Feet per second

NOTE:

For mass grading, materials with velocities of less than approximately 5000 fps are generally rippable with a D9 Caterpillar Tractor equipped with a single shank hydraulic ripper. Seismic compression wave velocities of 5000 to 6000 fps indicate marginal ripping and blasting. Velocities greater than 6000 fps generally require pre-blasting. For trenching, materials with velocities less than 3800 fps are generally rippable depending upon the degree of fracturing and the presence or absence of boulders. Velocities between 3800 and 4300 fps generally indicate marginal ripping, and velocities greater than 4300 fps generally indicate non-rippable conditions. The above velocities are based on a Kohring 505. The reported velocities represent average velocities over the length of each traverse, and should not generally be used for subsurface interpretation greater than 100 feet from a traverse.

PROJECT NO. T2214-12-01

# TRENCH T 1

ELEV. (MSL.) 1524 DATE COMPLETED 03-24-2004

EQUIPMENT CASE 580 BACKHOE

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | **UNDOCUMENTED FILL** Loose, dry, gray brown, fine to coarse SAND with gravel, cobbles and boulders | | | |
| | T1-1 | | | SP | | | | |
| 2 | | | | | | | | |
| | T1-2 | | | | **GABBROIC BEDROCK** Moderately hard, dry, dark gray, slightly jointed with iron oxide in joints | | | |
| 4 | | | | | | | | |
| 6 | | | | | TRENCH TERMINATED AT 6 FEET<br>No groundwater encountered<br>Backfilled | | | |

T2214-12-01.GPJ

**Figure A-1,**
**Log of Trench T 1, Page 1 of 1**

| SAMPLE SYMBOLS | | | |
|---|---|---|---|
| | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-12-01

# TRENCH T 2

ELEV. (MSL.) 1524 DATE COMPLETED 03-24-2004

EQUIPMENT CASE 580 BACKHOE

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | SP | **UNDOCUMENTED FILL** Loose, dry, gray brown, fine to coarse SAND with cobbles | | | |
| | T2-1 | | | | **ALLUVIUM** Loose, damp, orange brown, fine to coarse Silty SAND, abundant porosity | | | |
| 2 | | | | | | | | |
| 4 | | | | | | | | |
| | T2-2 | | | SM | | | 95.3 | 7.4 |
| 6 | | | | | | | | |
| 8 | | | | | | | | |
| | T2-3 | | | SM | **OLDER ALLUVIUM** Medium dense, damp, orange brown, Silty SAND, slightly porous, slightly cemented with carbonate | | 110.3 | 4.6 |
| 10 | | | | | **GABBROIC BEDROCK** Hard, damp, moderately weathered | | | |
| | | | | | TRENCH TERMINATED AT 11 FEET<br>No groundwater encountered<br>Backfilled | | | |

T2214-12-01.GPJ

**Figure A-2,**
**Log of Trench T 2, Page 1 of 1**

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-12-01

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | TRENCH T 3<br>ELEV. (MSL.) 1522 DATE COMPLETED 03-24-2004<br>EQUIPMENT CASE 580 BACKHOE | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| | | | | | MATERIAL DESCRIPTION | | | |
| 0 | | | | SM | **UNDOCUMENTED FILL**<br>Loose, dry, brown, coarse Silty SAND with gravel and cobbles | | | |
| 2 – 8 | T3-1 | | | SM | **ALLUVIUM**<br>Loose, damp to moist, red brown, medium to coarse, Silty SAND, moderate amount of porosity | | | |
| 10 – 14 | T3-2<br>T3-3<br>T3-4<br>T3-5 | | | SM | -Carbonate cemented zone, blocky and abundant porosity at 9 feet<br>**OLDER ALLUVIUM**<br>Medium dense, moist, orange brown and gray, fine to medium, Clayey/Silty SAND, slightly blocky, moderately porous in upper 3 feet, slightly porous below | | 108.6 | 9.5 |
| | | | | | TRENCH TERMINATED AT 15 FEET<br>No groundwater encountered<br>Backfilled | | | |

**Figure A-3,**
**Log of Trench T 3, Page 1 of 1**

T2214-12-01.GPJ

| SAMPLE SYMBOLS | | | |
|---|---|---|---|
| | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-12-01

# TRENCH T 4

ELEV. (MSL.) 1524 DATE COMPLETED 03-24-2004

EQUIPMENT CASE 580 BACKHOE

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | SM | **UNDOCUMENTED FILL** Loose, dry, gray brown, coarse Silty SAND | | | |
| 2 – 6 | | | | SM | **ALLUVIUM** Loose, damp, orange brown, coarse Silty SAND, with cobble to 12" diameter | | | |
| 8 | T4-1 | | | SM | **OLDER ALLUVIUM** Medium dense, damp, orange brown, coarse Silty SAND, slightly blocky, upper 1 foot cemented with carbonate, moderately porous | | 130.3 | 8.6 |
| 10 – 12 | | | | | | | | |
| 13 | T4-2 | | | | -Slightly porous, operator reports difficult digging at 13 feet | | 122.7 | 11.6 |
| 14 | | | | | TRENCH TERMINATED AT 14 FEET<br>No groundwater encountered<br>Backfilled | | | |

T2214-12-01.GPJ

**Figure A-4,**
**Log of Trench T 4, Page 1 of 1**

| SAMPLE SYMBOLS | | | |
|---|---|---|---|
| | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-12-01

# TRENCH T 5

ELEV. (MSL.) **1528** DATE COMPLETED **03-24-2004**

EQUIPMENT **CASE 580 BACKHOE**

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 – 4 | | | | SM | **UNDOCUMENTED FILL** Loose, dry, light brown, coarse Silty SAND with gravel, cobbles and small boulders | | | |
| 4 – 7 | | | | SM | **OLDER ALLUVIUM** Medium dense, damp, red brown, Clayey/Silty SAND with some cobbles and small boulders | | | |
| 7 – 8 | | | | | **GABBROIC BEDROCK** Hard, damp, black, white and orange, moderately to highly weathered | | | |
| 8 | | | | | TRENCH TERMINATED AT 8 FEET<br>No groundwater<br>Backfilled | | | |

T2214-12-01.GPJ

**Figure A-5,**
**Log of Trench T 5, Page 1 of 1**

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-12-01

## TRENCH T 6

ELEV. (MSL.) 1526 DATE COMPLETED 03-24-2004

EQUIPMENT CASE 580 BACKHOE

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | **ALLUVIUM** Loose, dry-damp, brown to orange, Silty SAND | | | |
| 2 | | | | | | | | |
| | T6-1 | | | SM | | | 98.4 | 6.8 |
| 4 | | | | | | | | |
| 6 | | | | | Medium dense, damp, brown, very coarse, Silty SAND, abundant primary porosity | | | |
| | | | | SM | | | | |
| 8 | T6-2 | | | | **OLDER ALLUVIUM** Medium dense, damp, red brown, medium to coarse, Clayey/Silty SAND, blocky, trace porosity | | 117.5 | 9.8 |
| 10 | | | | SM | | | | |
| 12 | | | | | **GABBROIC BEDROCK** Moderately hard, damp, black-white and orange, highly weathered in upper 1 foot | | | |

TRENCH TERMINATED AT 13 FEET
No groundwater encountered
Backfilled

T2214-12-01.GPJ

**Figure A-6,**
**Log of Trench T 6, Page 1 of 1**

| SAMPLE SYMBOLS | | | |
|---|---|---|---|
| | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-12-01

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | TRENCH T 7<br>ELEV. (MSL.) 1528 DATE COMPLETED 03-24-2004<br>EQUIPMENT CASE 580 BACKHOE | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| | | | | | MATERIAL DESCRIPTION | | | |
| 0<br>2 | | | | SM | **UNDOCUMENTED FILL**<br>Loose, dry, light brown, coarse Silty SAND with some cobbles and small boulders | | | |
| | | | | | **GABBROIC BEDROCK**<br>Hard, dry, gray, slightly weathered, refusal at 3 feet | | | |
| | | | | | TRENCH TERMINATED AT 3 FEET<br>No groundwater encountered<br>Backfilled | | | |

T2214-12-01.GPJ

**Figure A-7,**
**Log of Trench T 7, Page 1 of 1**

| SAMPLE SYMBOLS | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
|---|---|---|---|
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-12-01

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | TRENCH T 8<br>ELEV. (MSL.) 1560 DATE COMPLETED 03-24-2004<br>EQUIPMENT CASE 580 BACKHOE | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| | | | | | MATERIAL DESCRIPTION | | | |
| 0 – 2 | | | | SM | **UNDOCUMENTED FILL**<br>Loose, dry, light brown, coarse Silty SAND with gravel, cobbles and boulders | | | |
| 4 | | | | | **GABBROIC BEDROCK**<br>Very hard, dry, black and white, fresh | | | |
| | | | | | TRENCH TERMINATED AT 4½ FEET<br>No groundwater encountered<br>Backfilled | | | |

T2214-12-01.GPJ

**Figure A-8,**
**Log of Trench T 8, Page 1 of 1**

| SAMPLE SYMBOLS | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
|---|---|---|---|
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-12-01

# TRENCH T 9

ELEV. (MSL.) **1552** DATE COMPLETED **03-24-2004**

EQUIPMENT **CASE 580 BACKHOE**

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 – 5 | | | | SM | **UNDOCUMENTED FILL**<br>Loose, dry, light brown, coarse Silty SAND with cobbles and boulders | | | |
| 5 – 6 | | | | | **GABBROIC BEDROCK**<br>Hard, dry, black and white, fresh | | | |
| 6 | | | | | TRENCH TERMINATED AT 6 FEET<br>No groundwater encountered<br>Backfilled | | | |

T2214-12-01.GPJ

**Figure A-9,**
**Log of Trench T 9, Page 1 of 1**

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-12-01

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | TRENCH T 10<br>ELEV. (MSL.) 1552 DATE COMPLETED 03-24-2004<br>EQUIPMENT CASE 580 BACKHOE | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| | | | | | MATERIAL DESCRIPTION | | | |
| 0 – 3 | | | | SM | **UNDOCUMENTED FILL**<br>Loose, dry, coarse Silty SAND with gravel and cobbles | | | |
| 3 – 4½ | | | | | **GABBROIC BEDROCK**<br>Hard, dry, dark gray, joints 1-3 inches apart with carbonate lining the joints, moderately weathered | | | |
| | | | | | TRENCH TERMINATED AT 4½ FEET<br>No groundwater encountered<br>Backfilled | | | |

T2214-12-01.GPJ

**Figure A-10,**
**Log of Trench T 10, Page 1 of 1**

| SAMPLE SYMBOLS | | | |
|---|---|---|---|
| | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-12-01

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | TRENCH T 11 ELEV. (MSL.) 1553 DATE COMPLETED 03-24-2004 EQUIPMENT CASE 580 BACKHOE | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| | | | | | MATERIAL DESCRIPTION | | | |
| 0 | T1-1 | | | SC | **UNDOCUMENTED FILL** Medium dense, dry, red brown, Clayey SAND, porous | | | |
| 2 | | | | | | | | |
| 4 | | | | SC | **OLDER ALLUVIUM** Dense, dry, red brown, coarse Clayey SAND, very difficult digging | | | |
| | | | | | TRENCH TERMINATED AT 5 FEET<br>No groundwater encountered<br>Backfilled | | | |

T2214-12-01.GPJ

**Figure A-11,**
**Log of Trench T 11, Page 1 of 1**

| SAMPLE SYMBOLS | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
|---|---|---|---|
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-12-01

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | TRENCH T 12<br>ELEV. (MSL.) 1552 DATE COMPLETED 03-24-2004<br>EQUIPMENT CASE 580 BACKHOE | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| | | | | | MATERIAL DESCRIPTION | | | |
| 0 | | | | | **OLDER ALLUVIUM**<br>Medium dense, dry, orange brown, Clayey Silty SAND, blocky, moderately porous | | | |
| 2 | | | | | | | | |
| 4 | | | | SM | | | | |
| 6 | T12-1 | | | | -Cemented with clay and carbonate, moderately porous with some large diameter pores at 5 feet | | 108.2 | 7.8 |
| 8 | T12-2 | | | | -Porosity decreasing but still some large diameter pores (1/2 cm) at 8 feet | | 122.9 | 7.7 |
| | | | | | TRENCH TERMINATED AT 9½ FEET<br>No groundwater encountered<br>Backfilled | | | |

T2214-12-01.GPJ

**Figure A-12,**
**Log of Trench T 12, Page 1 of 1**

| SAMPLE SYMBOLS | | | |
|---|---|---|---|
| | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-12-01

## TRENCH T 13

ELEV. (MSL.) 1575 DATE COMPLETED 03-24-2004

EQUIPMENT CASE 580 BACKHOE

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 – 8 | | | | SM | **UNDOCUMENTED FILL** Loose to medium dense, dry to damp, red brown, coarse Silty SAND with some cobbles and chunks of soil | | | |
| 8 – 11 | T13-1 | | | SM | **ALLUVIUM** Medium dense, moist, orange brown, medium Silty SAND, slightly blocky, some large diameter pores (1/2 cm) | | 102.2 | 10.1 |
| 11 – 12 | | | | SM | **OLDER ALLUVIUM** Dense, damp, red brown, Silty SAND, slightly porous | | | |
| 12 | | | | | TRENCH TERMINATED AT 12 FEET<br>No groundwater encountered<br>Backfilled | | | |

T2214-12-01.GPJ

**Figure A-13,**
**Log of Trench T 13, Page 1 of 1**

| SAMPLE SYMBOLS | | | |
|---|---|---|---|
| | ☐ ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ■ ... DRIVE SAMPLE (UNDISTURBED) |
| | ⊠ ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ▼ ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-12-01

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | TRENCH T 14<br>ELEV. (MSL.) 1552 DATE COMPLETED 03-24-2004<br>EQUIPMENT CASE 580 BACKHOE | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| | | | | | MATERIAL DESCRIPTION | | | |
| 0<br>2 | | | | | **GABBROIC BEDROCK**<br>Soft to moderately hard, dry, black and white, highly weathered | | | |
| 4 | | | | | TRENCH TERMINATED AT 4 FEET<br>No groundwater encountered<br>Backfilled | | | |

T2214-12-01.GPJ

**Figure A-14,**
**Log of Trench T 14, Page 1 of 1**

| SAMPLE SYMBOLS | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
|---|---|---|---|
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-12-01

## TRENCH T 15

ELEV. (MSL.) **1546** DATE COMPLETED **03-24-2004**

EQUIPMENT **CASE 580 BACKHOE**

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 – 5 | | | | SM | **UNDOCUMENTED FILL** Loose, damp, light gray, coarse Silty SAND with boulders | | | |
| 5 – 6 | | | | | **GABBROIC BEDROCK** Moderately hard, damp, gray, highly weathered | | | |
| 6 | | | | | TRENCH TERMINATED AT 6 FEET<br>No groundwater encountered<br>Backfilled | | | |

T2214-12-01.GPJ

**Figure A-15,**
**Log of Trench T 15, Page 1 of 1**

| SAMPLE SYMBOLS | | | |
|---|---|---|---|
| | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-12-01

# TRENCH T 16

ELEV. (MSL.) 1529 DATE COMPLETED 03-24-2004

EQUIPMENT CASE 580 BACKHOE

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | | | | | **ALLUVIUM**<br>Loose, damp, brown, medium Silty SAND | | | |
| 2 | | | | SM | | | | |
| 4 | | | | | **GABBROIC BEDROCK**<br>Moderately hard, damp, black, white to orange, highly weathered | | | |
| | | | | | TRENCH TERMINATED AT 5 FEET<br>No groundwater encountered<br>Backfilled | | | |

T2214-12-01.GPJ

**Figure A-16,**
**Log of Trench T 16, Page 1 of 1**

SAMPLE SYMBOLS
- ... SAMPLING UNSUCCESSFUL
- ... DISTURBED OR BAG SAMPLE
- ... STANDARD PENETRATION TEST
- ... CHUNK SAMPLE
- ... DRIVE SAMPLE (UNDISTURBED)
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-22-02

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | HAND PIT P-1<br>ELEV. (MSL.) 1533 DATE COMPLETED 11/9/2020<br>EQUIPMENT HAND AUGER BY: Weidman | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| | | | | | MATERIAL DESCRIPTION | | | |
| 0 | | | | SM | **ALLUVIUM (Qa)**<br>Silty SAND, medium dense, dry, brown; fine to coarse sand; trace gravel at surface | | | |
| 2 | | | | | | | | |
| | P-1@3' | | | | | | | |
| | | | | | Total Depth = 3'<br>Groundwater not encountered<br>Backfilled with cuttings 11/10/2019 | | | |

**Figure A-17,**
**Log of Hand Pit P-1, Page 1 of 1**

BORING LOGS.GPJ

| SAMPLE SYMBOLS | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
|---|---|---|---|
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | HAND PIT P-2<br>ELEV. (MSL.) 1529 DATE COMPLETED 11/9/2020<br>EQUIPMENT HAND AUGER BY: Weidman | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| | | | | | MATERIAL DESCRIPTION | | | |
| 0 | P-2@20" | | | SM | **ALLUVIUM (Qa)**<br>Silty SAND, medium dense, dry, grayish brown; fine to coarse sand; few gravel | | | |
| | | | | | Total Depth = 1.59'<br>Groundwater not encountered<br>Backfilled with cuttings 11/11/2019 | | | |

**Figure A-18,**
**Log of Hand Pit P-2, Page 1 of 1**

BORING LOGS.GPJ

| SAMPLE SYMBOLS | | | |
|---|---|---|---|
| | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | HAND PIT P-3<br>ELEV. (MSL.) 1528 DATE COMPLETED 11/9/2020<br>EQUIPMENT HAND AUGER BY: Weidman | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| | | | | | MATERIAL DESCRIPTION | | | |
| 0 | P-3@20" | | | SM | **ALLUVIUM (Qa)**<br>Silty SAND, medium dense, dry, grayish brown; fine to coarse sand; few gravel | | | |
| | | | | | Total Depth = 1.65'<br>Groundwater not encountered<br>Backfilled with cuttings 11/11/2019 | | | |

**Figure A-19,**
**Log of Hand Pit P-3, Page 1 of 1**

BORING LOGS.GPJ

| SAMPLE SYMBOLS | | | |
|---|---|---|---|
| | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | **HAND PIT P-4**<br>ELEV. (MSL.) **1531** DATE COMPLETED **11/9/2020**<br>EQUIPMENT **HAND AUGER** BY: **Weidman** | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| | | | | | MATERIAL DESCRIPTION | | | |
| 0 | P-4@20" | | | SM | **ALLUVIUM (Qa)**<br>Silty SAND, medium dense, dry, brown; fine to coarse sand; trace gravel | | | |
| | | | | | Total Depth = 1.61'<br>Groundwater not encountered<br>Backfilled with cuttings 11/10/2019 | | | |

**Figure A-20,**
**Log of Hand Pit P-4, Page 1 of 1**

BORING LOGS.GPJ

| SAMPLE SYMBOLS | | | |
|---|---|---|---|
| | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | **HAND PIT P-5**<br>ELEV. (MSL.) **1530** DATE COMPLETED **11/9/2020**<br>EQUIPMENT **HAND AUGER** BY: **Weidman** | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| | | | | | MATERIAL DESCRIPTION | | | |
| 0 | | | | SM | **ALLUVIUM (Qa)**<br>Silty SAND, medium dense, dry, grayish brown; fine to coarse sand | | | |
| | P-5@20" | | | | | | | |
| | | | | | Total Depth = 1.65'<br>Groundwater not encountered<br>Backfilled with cuttings 11/10/2019 | | | |

**Figure A-21,**
**Log of Hand Pit P-5, Page 1 of 1**

BORING LOGS.GPJ

| SAMPLE SYMBOLS | | | |
|---|---|---|---|
| | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

PROJECT NO. T2214-22-02

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | HAND PIT P-6<br>ELEV. (MSL.) 1531 DATE COMPLETED 11/9/2020<br>EQUIPMENT HAND AUGER BY: Weidman | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| | | | | | MATERIAL DESCRIPTION | | | |
| 0 | P-6@20" | | | SM | **ALLUVIUM (Qa)**<br>Silty SAND, medium dense, dry, grayish brown; fine to coarse sand | | | |
| | | | | | Total Depth = 1.66'<br>Groundwater not encountered<br>Backfilled with cuttings 11/10/2019 | | | |

**Figure A-22,**
**Log of Hand Pit P-6, Page 1 of 1**

BORING LOGS.GPJ

| SAMPLE SYMBOLS | | | |
|---|---|---|---|
| | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

## HAND PIT P-7

ELEV. (MSL.) **1530** DATE COMPLETED **11/9/2020**

EQUIPMENT **HAND AUGER** BY: **Weidman**

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | MATERIAL DESCRIPTION | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| 0 | P-7@20" | | | SM | **ALLUVIUM (Qa)** Silty SAND, medium dense, dry, grayish brown; fine to coarse sand; trace gravel | | | |
| | | | | | Total Depth = 1.59'<br>Groundwater not encountered<br>Backfilled with cuttings 11/11/2019 | | | |

**Figure A-23,**
**Log of Hand Pit P-7, Page 1 of 1**

BORING LOGS.GPJ

SAMPLE SYMBOLS

- ... SAMPLING UNSUCCESSFUL
- ... STANDARD PENETRATION TEST
- ... DRIVE SAMPLE (UNDISTURBED)
- ... DISTURBED OR BAG SAMPLE
- ... CHUNK SAMPLE
- ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

PROJECT NO. T2214-22-02

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | HAND PIT P-8<br>ELEV. (MSL.) 1529 DATE COMPLETED 11/9/2020<br>EQUIPMENT HAND AUGER BY: Weidman | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| | | | | | MATERIAL DESCRIPTION | | | |
| 0 | P-8@20" | | | SM | **ALLUVIUM (Qa)**<br>Silty SAND, medium dense, dry, grayish brown; fine to coarse sand; trace gravel | | | |
| | | | | | Total Depth = 1.56'<br>Groundwater not encountered<br>Backfilled with cuttings 11/11/2019 | | | |

**Figure A-24,**
**Log of Hand Pit P-8, Page 1 of 1**

BORING LOGS.GPJ

| SAMPLE SYMBOLS | | | |
|---|---|---|---|
| | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

| DEPTH IN FEET | SAMPLE NO. | LITHOLOGY | GROUNDWATER | SOIL CLASS (USCS) | HAND PIT P-9<br>ELEV. (MSL.) 1529 DATE COMPLETED 11/9/2020<br>EQUIPMENT HAND AUGER BY: Weidman | PENETRATION RESISTANCE (BLOWS/FT.) | DRY DENSITY (P.C.F.) | MOISTURE CONTENT (%) |
|---|---|---|---|---|---|---|---|---|
| | | | | | MATERIAL DESCRIPTION | | | |
| 0 | P-9@20" | | | SM | **ALLUVIUM (Qa)**<br>Silty SAND, medium dense, dry, grayish brown; fine to coarse sand; trace gravel | | | |
| | | | | | Total Depth = 1.58'<br>Groundwater not encountered<br>Backfilled with cuttings 11/11/2019 | | | |

**Figure A-25,**
**Log of Hand Pit P-9, Page 1 of 1**

BORING LOGS.GPJ

| SAMPLE SYMBOLS | | | |
|---|---|---|---|
| | ... SAMPLING UNSUCCESSFUL | ... STANDARD PENETRATION TEST | ... DRIVE SAMPLE (UNDISTURBED) |
| | ... DISTURBED OR BAG SAMPLE | ... CHUNK SAMPLE | ... WATER TABLE OR SEEPAGE |

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED. IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

GEOCON

## PERCOLATION TEST REPORT

| | | | |
|---|---|---|---|
| **Project Name:** | Rivani Hemet | **Project No.:** | T2214-22-02 |
| **Test Hole No.:** | P-1 | **Date Excavated:** | 11/9/2020 |
| **Length of Test Pipe:** | 42.0 inches | **Soil Classification:** | SM |
| **Height of Pipe above Ground:** | 6.0 inches | **Presoak Date:** | 11/9/2020 |
| **Depth of Test Hole:** | 36.0 inches | **Perc Test Date:** | 11/10/2020 |
| **Check for Sandy Soil Criteria Tested by:** | Weidman | **Percolation Tested by:** | Weidman |

**Water level measured from BOTTOM of hole**

**Sandy Soil Criteria Test**

| Trial No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Level (in) | Final Water Level (in) | Δ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 9:30 AM<br>9:55 AM | 25 | 25 | 14.3 | 11.3 | 3.0 | 8.3 |
| 2 | 9:55 AM<br>10:20 AM | 25 | 50 | 16.4 | 12.0 | 4.4 | 5.6 |

**Soil Criteria: Normal**

**Percolation Test**

| Reading No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Head (in) | Final Water Head (in) | Δ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 10:20 AM<br>10:50 AM | 30 | 30 | 16.3 | 11.6 | 4.7 | 6.4 |
| 2 | 10:50 AM<br>11:20 AM | 30 | 60 | 15.7 | 11.8 | 4.0 | 7.6 |
| 3 | 11:20 AM<br>11:50 AM | 30 | 90 | 15.5 | 11.8 | 3.7 | 8.1 |
| 4 | 11:50 AM<br>12:20 PM | 30 | 120 | 15.6 | 12.1 | 3.5 | 8.6 |
| 5 | 12:20 PM<br>12:50 PM | 30 | 150 | 15.5 | 12.0 | 3.5 | 8.6 |
| 6 | 12:50 PM<br>1:20 PM | 30 | 180 | 15.5 | 12.0 | 3.5 | 8.6 |
| 7 | 1:20 PM<br>1:50 PM | 30 | 210 | 15.5 | 12.5 | 3.0 | 10.0 |
| 8 | 1:50 PM<br>2:20 PM | 30 | 240 | 15.4 | 12.1 | 3.2 | 9.3 |
| 9 | 2:20 PM<br>2:50 PM | 30 | 270 | 15.5 | 12.4 | 3.1 | 9.6 |
| 10 | 2:50 PM<br>3:20 PM | 30 | 300 | 15.5 | 12.6 | 2.9 | 10.4 |
| 11 | 3:20 PM<br>3:50 PM | 30 | 330 | 15.5 | 12.5 | 3.0 | 10.0 |
| 12 | 3:50 PM<br>4:20 PM | 30 | 360 | 15.5 | 12.5 | 3.0 | 10.0 |

| | |
|---|---|
| **Infiltration Rate (in/hr):** | 0.8 |
| **Radius of test hole (in):** | 4 |
| **Average Head (in):** | 14.0 |

**Figure A-26**

## PERCOLATION TEST REPORT

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Project Name:** | | Rivani Hemet | | | **Project No.:** | | T2214-22-02 |
| **Test Hole No.:** | | P-2 | | | **Date Excavated:** | | 11/9/2020 |
| **Length of Test Pipe:** | | | 21.6 | inches | **Soil Classification:** | | SM |
| **Height of Pipe above Ground:** | | | 2.5 | inches | **Presoak Date:** | | 11/10/2020 |
| **Depth of Test Hole:** | | | 19.1 | inches | **Perc Test Date:** | | 11/11/2020 |
| **Check for Sandy Soil Criteria Tested by:** | | | | Weidman | **Percolation Tested by:** | | Weidman |

**Water level measured from BOTTOM of hole**

### Sandy Soil Criteria Test

| Trial No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Level (in) | Final Water Level (in) | Δ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 9:42 AM<br>10:07 AM | 25 | 25 | 14.3 | 9.7 | 4.6 | 5.5 |
| 2 | 10:07 AM<br>10:32 AM | 25 | 50 | 13.9 | 10.6 | 3.4 | 7.4 |

**Soil Criteria: Normal**

### Percolation Test

| Reading No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Head (in) | Final Water Head (in) | Δ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 10:32 AM<br>11:02 AM | 30 | 30 | 13.9 | 9.8 | 4.1 | 7.4 |
| 2 | 11:02 AM<br>11:32 AM | 30 | 60 | 13.9 | 10.8 | 3.1 | 9.6 |
| 3 | 11:32 AM<br>12:02 PM | 30 | 90 | 13.9 | 10.4 | 3.5 | 8.6 |
| 4 | 12:02 PM<br>12:32 PM | 30 | 120 | 13.9 | 10.4 | 3.5 | 8.6 |
| 5 | 12:32 PM<br>1:02 PM | 30 | 150 | 13.9 | 10.6 | 3.4 | 8.9 |
| 6 | 1:02 PM<br>1:32 PM | 30 | 180 | 13.9 | 10.9 | 3.0 | 10.0 |
| 7 | 1:32 PM<br>2:02 PM | 30 | 210 | 13.9 | 10.9 | 3.0 | 10.0 |
| 8 | 2:02 PM<br>2:32 PM | 30 | 240 | 13.9 | 10.4 | 3.5 | 8.6 |
| 9 | 2:32 PM<br>3:02 PM | 30 | 270 | 13.9 | 10.8 | 3.1 | 9.6 |
| 10 | 3:02 PM<br>3:32 PM | 30 | 300 | 13.9 | 10.7 | 3.2 | 9.3 |
| 11 | 3:32 PM<br>4:02 PM | 30 | 330 | 13.9 | 10.9 | 3.0 | 10.0 |
| 12 | 4:02 PM<br>4:32 PM | 30 | 360 | 13.9 | 10.8 | 3.1 | 9.6 |

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Infiltration Rate (in/hr):** | | | 0.9 | | | | |
| **Radius of test hole (in):** | | | 4 | | | | **Figure A-27** |
| **Average Head (in):** | | | 12.4 | | | | |

| PERCOLATION TEST REPORT | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Project Name:** | | Rivani Hemet | | | **Project No.:** | | T2214-22-02 |
| **Test Hole No.:** | | P-3 | | | **Date Excavated:** | | 11/9/2020 |
| **Length of Test Pipe:** | | | 24.0 | inches | **Soil Classification:** | | SM |
| **Height of Pipe above Ground:** | | | 4.2 | inches | **Presoak Date:** | | 11/10/2020 |
| **Depth of Test Hole:** | | | 19.8 | inches | **Perc Test Date:** | | 11/11/2020 |
| **Check for Sandy Soil Criteria Tested by:** | | | | Weidman | **Percolation Tested by:** | | Weidman |
| **Water level measured from BOTTOM of hole** | | | | | | | |

**Sandy Soil Criteria Test**

| Trial No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Level (in) | Final Water Level (in) | Δ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 9:39 AM<br>10:04 AM | 25 | 25 | 15.5 | 8.9 | 6.6 | 3.8 |
| 2 | 10:04 AM<br>10:29 AM | 25 | 50 | 15.5 | 9.0 | 6.5 | 3.9 |

**Soil Criteria: Normal**

**Percolation Test**

| Reading No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Head (in) | Final Water Head (in) | Δ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 10:29 AM<br>10:59 AM | 30 | 30 | 15.2 | 8.5 | 6.7 | 4.5 |
| 2 | 10:59 AM<br>11:29 AM | 30 | 60 | 15.4 | 9.2 | 6.1 | 4.9 |
| 3 | 11:29 AM<br>11:59 AM | 30 | 90 | 15.4 | 9.1 | 6.2 | 4.8 |
| 4 | 11:59 AM<br>12:29 PM | 30 | 120 | 15.4 | 9.6 | 5.8 | 5.2 |
| 5 | 12:29 PM<br>12:59 PM | 30 | 150 | 15.5 | 9.7 | 5.8 | 5.2 |
| 6 | 12:59 PM<br>1:29 PM | 30 | 180 | 15.4 | 9.8 | 5.5 | 5.4 |
| 7 | 1:29 PM<br>1:59 PM | 30 | 210 | 15.5 | 10.1 | 5.4 | 5.6 |
| 8 | 1:59 PM<br>2:29 PM | 30 | 240 | 15.2 | 9.6 | 5.6 | 5.3 |
| 9 | 2:29 PM<br>2:59 PM | 30 | 270 | 15.4 | 10.0 | 5.4 | 5.6 |
| 10 | 2:59 PM<br>3:29 PM | 30 | 300 | 15.5 | 9.7 | 5.8 | 5.2 |
| 11 | 3:29 PM<br>3:59 PM | 30 | 330 | 15.4 | 9.7 | 5.6 | 5.3 |
| 12 | 3:59 PM<br>4:29 PM | 30 | 360 | 15.5 | 9.8 | 5.6 | 5.3 |

| | | | | | |
|---|---|---|---|---|---|
| **Infiltration Rate (in/hr):** | 1.5 | | | | |
| **Radius of test hole (in):** | 4 | | | | **Figure A-28** |
| **Average Head (in):** | 12.7 | | | | |

## PERCOLATION TEST REPORT

| | | | |
|---|---|---|---|
| **Project Name:** | Rivani Hemet | **Project No.:** | T2214-22-02 |
| **Test Hole No.:** | P-4 | **Date Excavated:** | 11/9/2020 |
| **Length of Test Pipe:** | 26.4 inches | **Soil Classification:** | SM |
| **Height of Pipe above Ground:** | 7.1 inches | **Presoak Date:** | 11/9/2020 |
| **Depth of Test Hole:** | 19.3 inches | **Perc Test Date:** | 11/10/2020 |
| **Check for Sandy Soil Criteria Tested by:** | Weidman | **Percolation Tested by:** | Weidman |

**Water level measured from BOTTOM of hole**

### Sandy Soil Criteria Test

| Trial No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Level (in) | Final Water Level (in) | ∆ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 9:33 AM<br>9:58 AM | 25 | 25 | 8.4 | 2.4 | 6.0 | 4.2 |
| 2 | 9:58 AM<br>10:23 AM | 25 | 50 | 9.2 | 5.3 | 4.0 | 6.3 |

**Soil Criteria: Normal**

### Percolation Test

| Reading No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Head (in) | Final Water Head (in) | ∆ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 10:23 AM<br>10:53 AM | 30 | 30 | 11.0 | 5.6 | 5.4 | 5.6 |
| 2 | 10:53 AM<br>11:23 AM | 30 | 60 | 11.0 | 6.5 | 4.6 | 6.6 |
| 3 | 11:23 AM<br>11:53 AM | 30 | 90 | 11.5 | 7.1 | 4.4 | 6.8 |
| 4 | 11:53 AM<br>12:23 PM | 30 | 120 | 11.6 | 7.3 | 4.3 | 6.9 |
| 5 | 12:23 PM<br>12:53 PM | 30 | 150 | 11.9 | 9.2 | 2.6 | 11.4 |
| 6 | 12:53 PM<br>1:23 PM | 30 | 180 | 14.0 | 10.2 | 3.8 | 7.8 |
| 7 | 1:23 PM<br>1:53 PM | 30 | 210 | 14.2 | 10.3 | 3.8 | 7.8 |
| 8 | 1:53 PM<br>2:23 PM | 30 | 240 | 13.9 | 10.3 | 3.6 | 8.3 |
| 9 | 2:23 PM<br>2:53 PM | 30 | 270 | 13.9 | 10.7 | 3.2 | 9.3 |
| 10 | 2:53 PM<br>3:23 PM | 30 | 300 | 14.2 | 10.4 | 3.7 | 8.1 |
| 11 | 3:23 PM<br>3:53 PM | 30 | 330 | 14.0 | 10.6 | 3.5 | 8.6 |
| 12 | 3:53 PM<br>4:23 PM | 30 | 360 | 14.0 | 10.6 | 3.5 | 8.6 |

| | |
|---|---|
| **Infiltration Rate (in/hr):** | 1.0 |
| **Radius of test hole (in):** | 4 |
| **Average Head (in):** | 12.3 |

**Figure A-29**

## PERCOLATION TEST REPORT

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Project Name:** | | Rivani Hemet | | | **Project No.:** | | T2214-22-02 |
| **Test Hole No.:** | | P-5 | | | **Date Excavated:** | | 11/9/2020 |
| **Length of Test Pipe:** | | | 26.4 | inches | **Soil Classification:** | | SM |
| **Height of Pipe above Ground:** | | | 6.6 | inches | **Presoak Date:** | | 11/9/2020 |
| **Depth of Test Hole:** | | | 19.8 | inches | **Perc Test Date:** | | 11/10/2020 |
| **Check for Sandy Soil Criteria Tested by:** | | | | Weidman | **Percolation Tested by:** | | Weidman |

**Water level measured from BOTTOM of hole**

### Sandy Soil Criteria Test

| Trial No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Level (in) | Final Water Level (in) | Δ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 9:36 AM<br>10:01 AM | 25 | 25 | 11.3 | 4.2 | 7.1 | 3.5 |
| 2 | 10:01 AM<br>10:26 AM | 25 | 50 | 11.2 | 6.2 | 4.9 | 5.1 |

**Soil Criteria: Normal**

### Percolation Test

| Reading No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Head (in) | Final Water Head (in) | Δ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 10:26 AM<br>10:56 AM | 30 | 30 | 11.8 | 7.6 | 4.2 | 7.1 |
| 2 | 10:56 AM<br>11:26 AM | 30 | 60 | 11.4 | 7.7 | 3.7 | 8.1 |
| 3 | 11:26 AM<br>11:56 AM | 30 | 90 | 11.9 | 7.7 | 4.2 | 7.1 |
| 4 | 11:56 AM<br>12:26 PM | 30 | 120 | 12.0 | 7.9 | 4.1 | 7.4 |
| 5 | 12:26 PM<br>12:56 PM | 30 | 150 | 11.3 | 6.7 | 4.6 | 6.6 |
| 6 | 12:56 PM<br>1:26 PM | 30 | 180 | 12.2 | 8.0 | 4.2 | 7.1 |
| 7 | 1:26 PM<br>1:56 PM | 30 | 210 | 12.2 | 7.8 | 4.4 | 6.8 |
| 8 | 1:56 PM<br>2:26 PM | 30 | 240 | 12.5 | 8.3 | 4.2 | 7.1 |
| 9 | 2:26 PM<br>2:56 PM | 30 | 270 | 12.4 | 8.2 | 4.2 | 7.1 |
| 10 | 2:56 PM<br>3:26 PM | 30 | 300 | 12.0 | 8.0 | 4.0 | 7.6 |
| 11 | 3:26 PM<br>3:56 PM | 30 | 330 | 12.1 | 8.3 | 3.8 | 7.8 |
| 12 | 3:56 PM<br>4:26 PM | 30 | 360 | 12.2 | 8.3 | 4.0 | 7.6 |

| | |
|---|---|
| **Infiltration Rate (in/hr):** | 1.4 |
| **Radius of test hole (in):** | 4 |
| **Average Head (in):** | 10.1 |

**Figure A-30**

## PERCOLATION TEST REPORT

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Project Name:** | | Rivani Hemet | | | **Project No.:** | | T2214-22-02 |
| **Test Hole No.:** | | P-6 | | | **Date Excavated:** | | 11/9/2020 |
| **Length of Test Pipe:** | | | 24.0 | inches | **Soil Classification:** | | SM |
| **Height of Pipe above Ground:** | | | 4.1 | inches | **Presoak Date:** | | 11/9/2020 |
| **Depth of Test Hole:** | | | 19.9 | inches | **Perc Test Date:** | | 11/10/2020 |
| **Check for Sandy Soil Criteria Tested by:** | | | | Weidman | **Percolation Tested by:** | | Weidman |

**Water level measured from BOTTOM of hole**

**Sandy Soil Criteria Test**

| Trial No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Level (in) | Final Water Level (in) | Δ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 9:39 AM<br>10:04 AM | 25 | 25 | 14.0 | 8.2 | 5.9 | 4.3 |
| 2 | 10:04 AM<br>10:29 AM | 25 | 50 | 14.8 | 9.2 | 5.5 | 4.5 |

**Soil Criteria: Normal**

**Percolation Test**

| Reading No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Head (in) | Final Water Head (in) | Δ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 10:29 AM<br>10:59 AM | 30 | 30 | 14.9 | 8.5 | 6.4 | 4.7 |
| 2 | 10:59 AM<br>11:29 AM | 30 | 60 | 14.4 | 9.1 | 5.3 | 5.7 |
| 3 | 11:29 AM<br>11:59 AM | 30 | 90 | 14.6 | 9.5 | 5.2 | 5.8 |
| 4 | 11:59 AM<br>12:29 PM | 30 | 120 | 15.2 | 9.7 | 5.5 | 5.4 |
| 5 | 12:29 PM<br>12:59 PM | 30 | 150 | 15.2 | 10.0 | 5.3 | 5.7 |
| 6 | 12:59 PM<br>1:29 PM | 30 | 180 | 15.5 | 10.2 | 5.3 | 5.7 |
| 7 | 1:29 PM<br>1:59 PM | 30 | 210 | 15.4 | 10.2 | 5.2 | 5.8 |
| 8 | 1:59 PM<br>2:29 PM | 30 | 240 | 15.4 | 10.2 | 5.2 | 5.8 |
| 9 | 2:29 PM<br>2:59 PM | 30 | 270 | 15.1 | 10.8 | 4.3 | 6.9 |
| 10 | 2:59 PM<br>3:29 PM | 30 | 300 | 15.4 | 10.1 | 5.3 | 5.7 |
| 11 | 3:29 PM<br>3:59 PM | 30 | 330 | 15.2 | 10.1 | 5.2 | 5.8 |
| 12 | 3:59 PM<br>4:29 PM | 30 | 360 | 15.4 | 10.2 | 5.2 | 5.8 |

| | | | |
|---|---|---|---|
| **Infiltration Rate (in/hr):** | | 1.4 | |
| **Radius of test hole (in):** | | 4 | **Figure A-31** |
| **Average Head (in):** | | 12.8 | |

## PERCOLATION TEST REPORT

| | | | |
|---|---|---|---|
| **Project Name:** | Rivani Hemet | **Project No.:** | T2214-22-02 |
| **Test Hole No.:** | P-7 | **Date Excavated:** | 11/9/2020 |
| **Length of Test Pipe:** | 24.0 inches | **Soil Classification:** | SM |
| **Height of Pipe above Ground:** | 4.9 inches | **Presoak Date:** | 11/10/2020 |
| **Depth of Test Hole:** | 19.1 inches | **Perc Test Date:** | 11/11/2020 |
| **Check for Sandy Soil Criteria Tested by:** | Weidman | **Percolation Tested by:** | Weidman |

**Water level measured from BOTTOM of hole**

### Sandy Soil Criteria Test

| Trial No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Level (in) | Final Water Level (in) | Δ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 9:30 AM<br>9:55 AM | 25 | 25 | 12.7 | 8.4 | 4.3 | 5.8 |
| 2 | 9:55 AM<br>10:20 AM | 25 | 50 | 13.1 | 8.5 | 4.6 | 5.5 |

**Soil Criteria: Normal**

### Percolation Test

| Reading No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Head (in) | Final Water Head (in) | Δ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 10:20 AM<br>10:50 AM | 30 | 30 | 13.0 | 8.2 | 4.8 | 6.3 |
| 2 | 10:50 AM<br>11:20 AM | 30 | 60 | 13.1 | 8.3 | 4.8 | 6.3 |
| 3 | 11:20 AM<br>11:50 AM | 30 | 90 | 13.0 | 8.4 | 4.6 | 6.6 |
| 4 | 11:50 AM<br>12:20 PM | 30 | 120 | 13.0 | 8.6 | 4.3 | 6.9 |
| 5 | 12:20 PM<br>12:50 PM | 30 | 150 | 13.0 | 8.9 | 4.1 | 7.4 |
| 6 | 12:50 PM<br>1:20 PM | 30 | 180 | 13.1 | 9.0 | 4.1 | 7.4 |
| 7 | 1:20 PM<br>1:50 PM | 30 | 210 | 13.0 | 9.2 | 3.7 | 8.1 |
| 8 | 1:50 PM<br>2:20 PM | 30 | 240 | 13.0 | 9.0 | 4.0 | 7.6 |
| 9 | 2:20 PM<br>2:50 PM | 30 | 270 | 13.3 | 9.4 | 4.0 | 7.6 |
| 10 | 2:50 PM<br>3:20 PM | 30 | 300 | 13.1 | 9.2 | 3.8 | 7.8 |
| 11 | 3:20 PM<br>3:50 PM | 30 | 330 | 13.2 | 9.5 | 3.7 | 8.1 |
| 12 | 3:50 PM<br>4:20 PM | 30 | 360 | 13.1 | 9.4 | 3.7 | 8.1 |

| | |
|---|---|
| **Infiltration Rate (in/hr):** | 1.1 |
| **Radius of test hole (in):** | 4 |
| **Average Head (in):** | 11.2 |

**Figure A-32**

## PERCOLATION TEST REPORT

| | | | |
|---|---|---|---|
| **Project Name:** | Rivani Hemet | **Project No.:** | T2214-22-02 |
| **Test Hole No.:** | P-8 | **Date Excavated:** | 11/9/2020 |
| **Length of Test Pipe:** | 24.0 inches | **Soil Classification:** | SM |
| **Height of Pipe above Ground:** | 5.3 inches | **Presoak Date:** | 11/10/2020 |
| **Depth of Test Hole:** | 18.7 inches | **Perc Test Date:** | 11/11/2020 |
| **Check for Sandy Soil Criteria Tested by:** | Weidman | **Percolation Tested by:** | Weidman |

**Water level measured from BOTTOM of hole**

### Sandy Soil Criteria Test

| Trial No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Level (in) | Final Water Level (in) | Δ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 9:33 AM<br>9:58 AM | 25 | 25 | 13.3 | 8.6 | 4.7 | 5.3 |
| 2 | 9:58 AM<br>10:23 AM | 25 | 50 | 13.0 | 9.2 | 3.7 | 6.7 |

**Soil Criteria: Normal**

### Percolation Test

| Reading No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Head (in) | Final Water Head (in) | Δ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 10:23 AM<br>10:53 AM | 30 | 30 | 13.2 | 9.1 | 4.1 | 7.4 |
| 2 | 10:53 AM<br>11:23 AM | 30 | 60 | 13.2 | 9.0 | 4.2 | 7.1 |
| 3 | 11:23 AM<br>11:53 AM | 30 | 90 | 13.4 | 9.2 | 4.2 | 7.1 |
| 4 | 11:53 AM<br>12:23 PM | 30 | 120 | 13.6 | 9.6 | 4.0 | 7.6 |
| 5 | 12:23 PM<br>12:53 PM | 30 | 150 | 13.3 | 9.6 | 3.7 | 8.1 |
| 6 | 12:53 PM<br>1:23 PM | 30 | 180 | 13.1 | 9.6 | 3.5 | 8.6 |
| 7 | 1:23 PM<br>1:53 PM | 30 | 210 | 13.2 | 9.6 | 3.6 | 8.3 |
| 8 | 1:53 PM<br>2:23 PM | 30 | 240 | 13.6 | 9.6 | 4.0 | 7.6 |
| 9 | 2:23 PM<br>2:53 PM | 30 | 270 | 13.2 | 9.6 | 3.6 | 8.3 |
| 10 | 2:53 PM<br>3:23 PM | 30 | 300 | 13.3 | 9.7 | 3.6 | 8.3 |
| 11 | 3:23 PM<br>3:53 PM | 30 | 330 | 13.3 | 9.7 | 3.6 | 8.3 |
| 12 | 3:53 PM<br>4:23 PM | 30 | 360 | 13.2 | 9.7 | 3.5 | 8.6 |

| | |
|---|---|
| **Infiltration Rate (in/hr):** | 1.0 |
| **Radius of test hole (in):** | 4 |
| **Average Head (in):** | 11.5 |

**Figure A-33**

## PERCOLATION TEST REPORT

| | | | |
|---|---|---|---|
| **Project Name:** | Rivani Hemet | **Project No.:** | T2214-22-02 |
| **Test Hole No.:** | P-9 | **Date Excavated:** | 11/9/2020 |
| **Length of Test Pipe:** | 21.6 inches | **Soil Classification:** | SM |
| **Height of Pipe above Ground:** | 2.6 inches | **Presoak Date:** | 11/10/2020 |
| **Depth of Test Hole:** | 19.0 inches | **Perc Test Date:** | 11/11/2020 |
| **Check for Sandy Soil Criteria Tested by:** | Weidman | **Percolation Tested by:** | Weidman |

**Water level measured from BOTTOM of hole**

### Sandy Soil Criteria Test

| Trial No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Level (in) | Final Water Level (in) | Δ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 9:36 AM<br>10:01 AM | 25 | 25 | 16.3 | 9.2 | 7.1 | 3.5 |
| 2 | 10:01 AM<br>10:26 AM | 25 | 50 | 16.4 | 10.4 | 6.0 | 4.2 |

**Soil Criteria: Normal**

### Percolation Test

| Reading No. | Time | Time Interval (min) | Total Elapsed Time (min) | Initial Water Head (in) | Final Water Head (in) | Δ in Water Level (in) | Percolation Rate (min/inch) |
|---|---|---|---|---|---|---|---|
| 1 | 10:26 AM<br>10:56 AM | 30 | 30 | 16.2 | 10.0 | 6.2 | 4.8 |
| 2 | 10:56 AM<br>11:26 AM | 30 | 60 | 16.6 | 10.2 | 6.4 | 4.7 |
| 3 | 11:26 AM<br>11:56 AM | 30 | 90 | 16.2 | 10.4 | 5.8 | 5.2 |
| 4 | 11:56 AM<br>12:26 PM | 30 | 120 | 16.6 | 10.6 | 6.0 | 5.0 |
| 5 | 12:26 PM<br>12:56 PM | 30 | 150 | 16.3 | 10.3 | 6.0 | 5.0 |
| 6 | 12:56 PM<br>1:26 PM | 30 | 180 | 16.3 | 10.4 | 5.9 | 5.1 |
| 7 | 1:26 PM<br>1:56 PM | 30 | 210 | 16.6 | 10.4 | 6.1 | 4.9 |
| 8 | 1:56 PM<br>2:26 PM | 30 | 240 | 16.3 | 10.4 | 5.9 | 5.1 |
| 9 | 2:26 PM<br>2:56 PM | 30 | 270 | 16.3 | 10.4 | 5.9 | 5.1 |
| 10 | 2:56 PM<br>3:26 PM | 30 | 300 | 16.4 | 10.6 | 5.9 | 5.1 |
| 11 | 3:26 PM<br>3:56 PM | 30 | 330 | 16.3 | 10.4 | 5.9 | 5.1 |
| 12 | 3:56 PM<br>4:26 PM | 30 | 360 | 16.3 | 10.6 | 5.8 | 5.2 |

| | |
|---|---|
| **Infiltration Rate (in/hr):** | 1.5 |
| **Radius of test hole (in):** | 4 |
| **Average Head (in):** | 13.4 |

**Figure A-34**

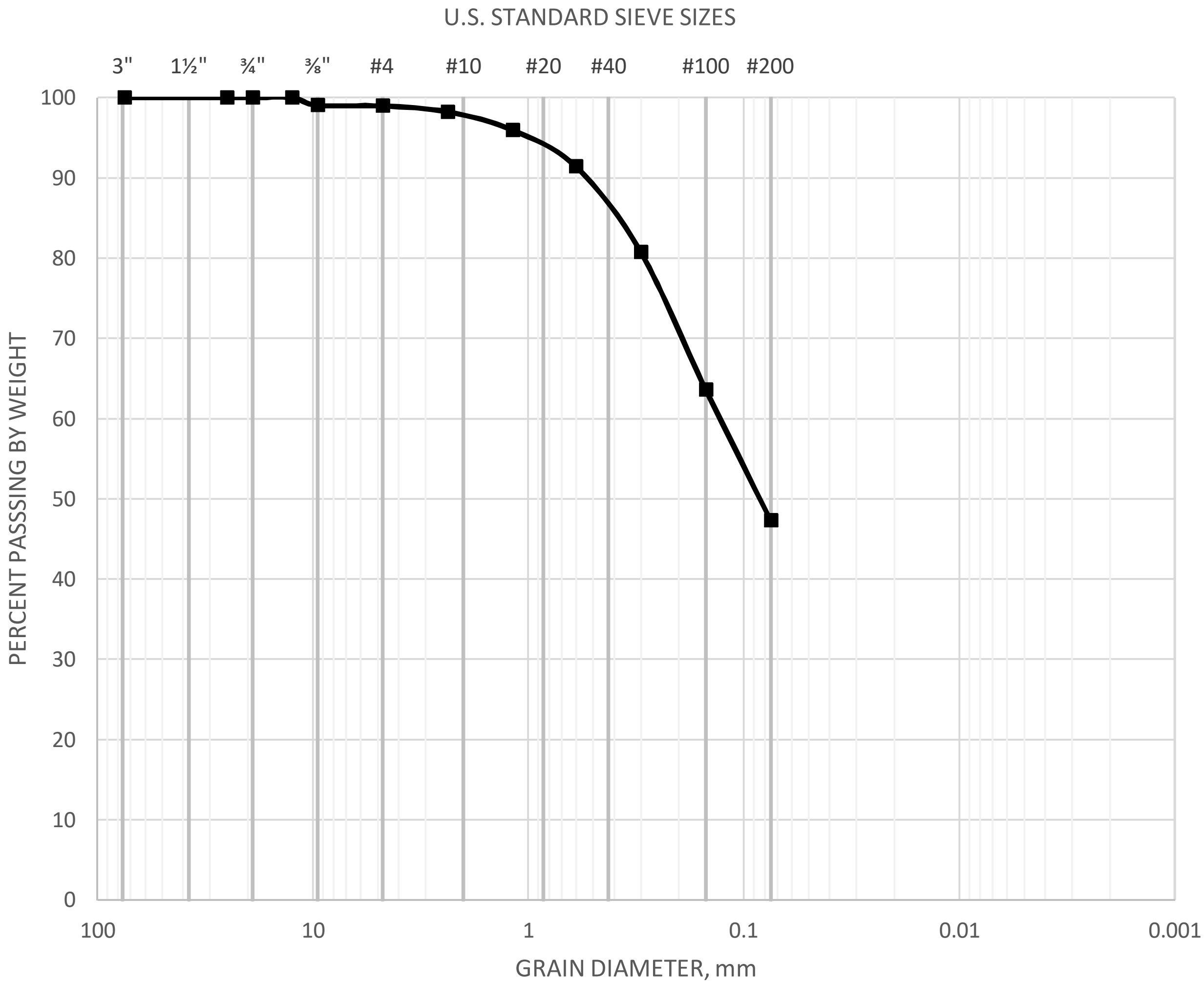
# APPENDIX B

# APPENDIX B

## LABORATORY TESTING

Laboratory tests were performed in general accordance with test methods of ASTM International (ASTM), California test (CT) methods or other suggested procedures. The results of the laboratory tests for this update are summarized in Figures B-1 through B-9. The laboratory test results from our 2004 investigation are also included in herein.

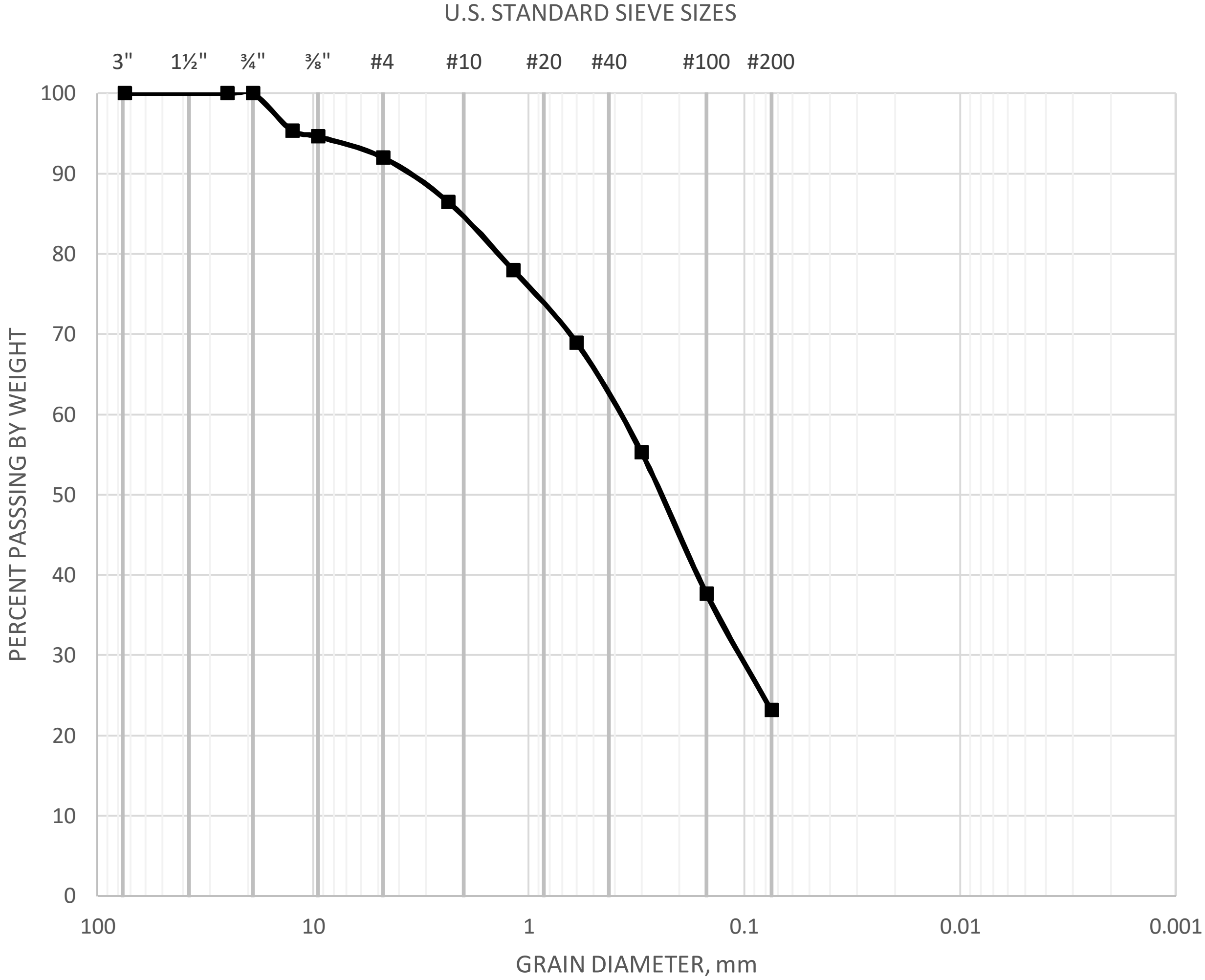
| GRAVEL | | SAND | | | SILT AND CLAY |
|---|---|---|---|---|---|
| COARSE | FINE | COARSE | MEDIUM | FINE | |




| SAMPLE | CLASSIFICATION | D60 | D30 | D10 |
|---|---|---|---|---|
| P-1@3' | silty SAND (SM), brown | 0 | 0 | 0 |

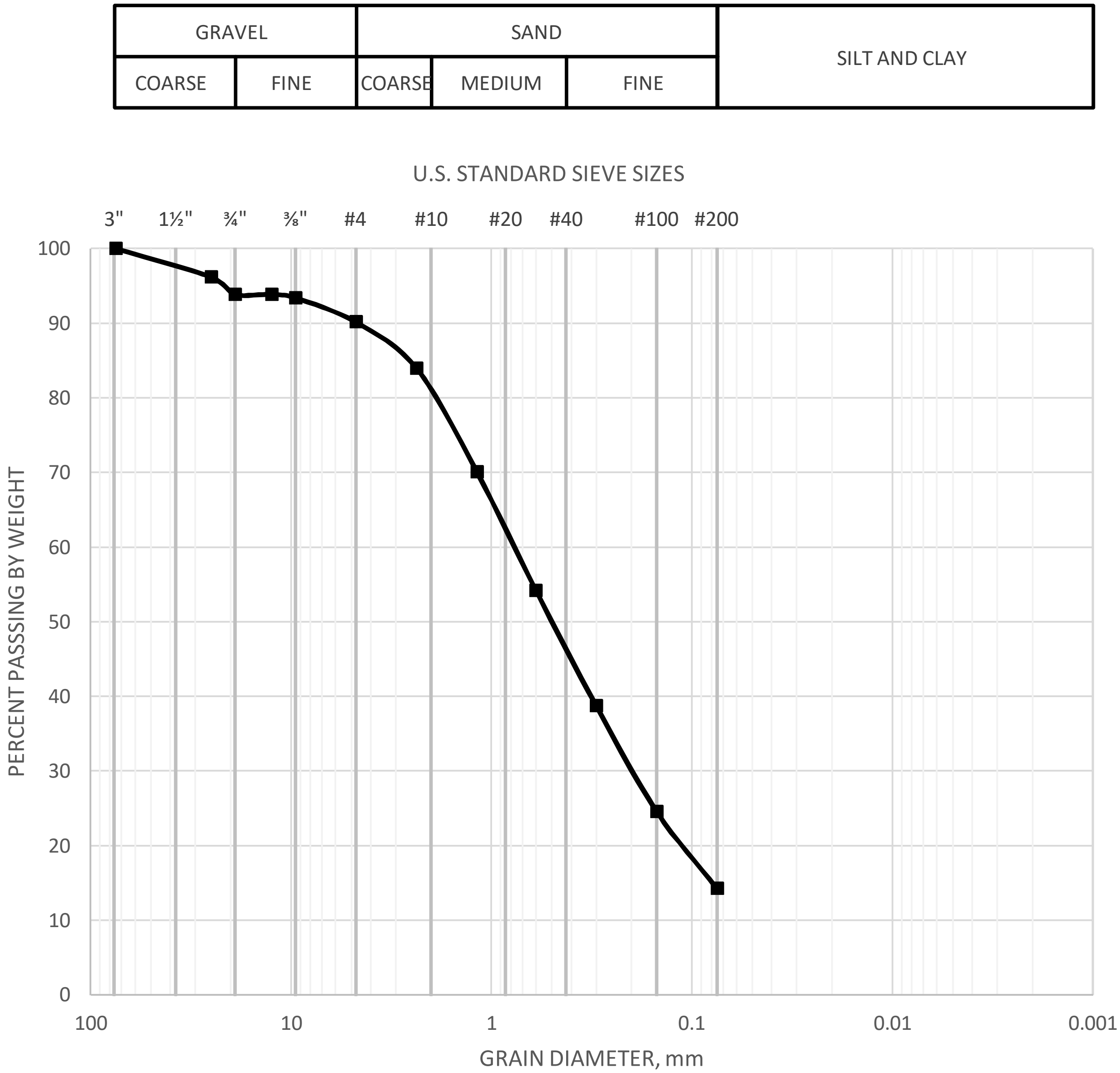
| | | |
|---|---|---|
| GEOCON | **GRAIN SIZE DISTRIBUTION**<br>ASTM D-422 | Project No.: T2214-22-02 |
| | | RIVANI HEMET 30<br>SOUTH OF HWY 74 & WEST OF JOEL DR<br>HEMET, RIVERSIDE COUNTY, CALIFORNIA |
| | Checked by: | Dec 20 Figure B-1 |

| GRAVEL | | SAND | | | SILT AND CLAY |
|---|---|---|---|---|---|
| COARSE | FINE | COARSE | MEDIUM | FINE | |




| SAMPLE | CLASSIFICATION | D60 | D30 | D10 |
|---|---|---|---|---|
| P-2@20" | silty SAND with few gravel (SM), grayish brown | 0.38 | 0.11 | 0 |

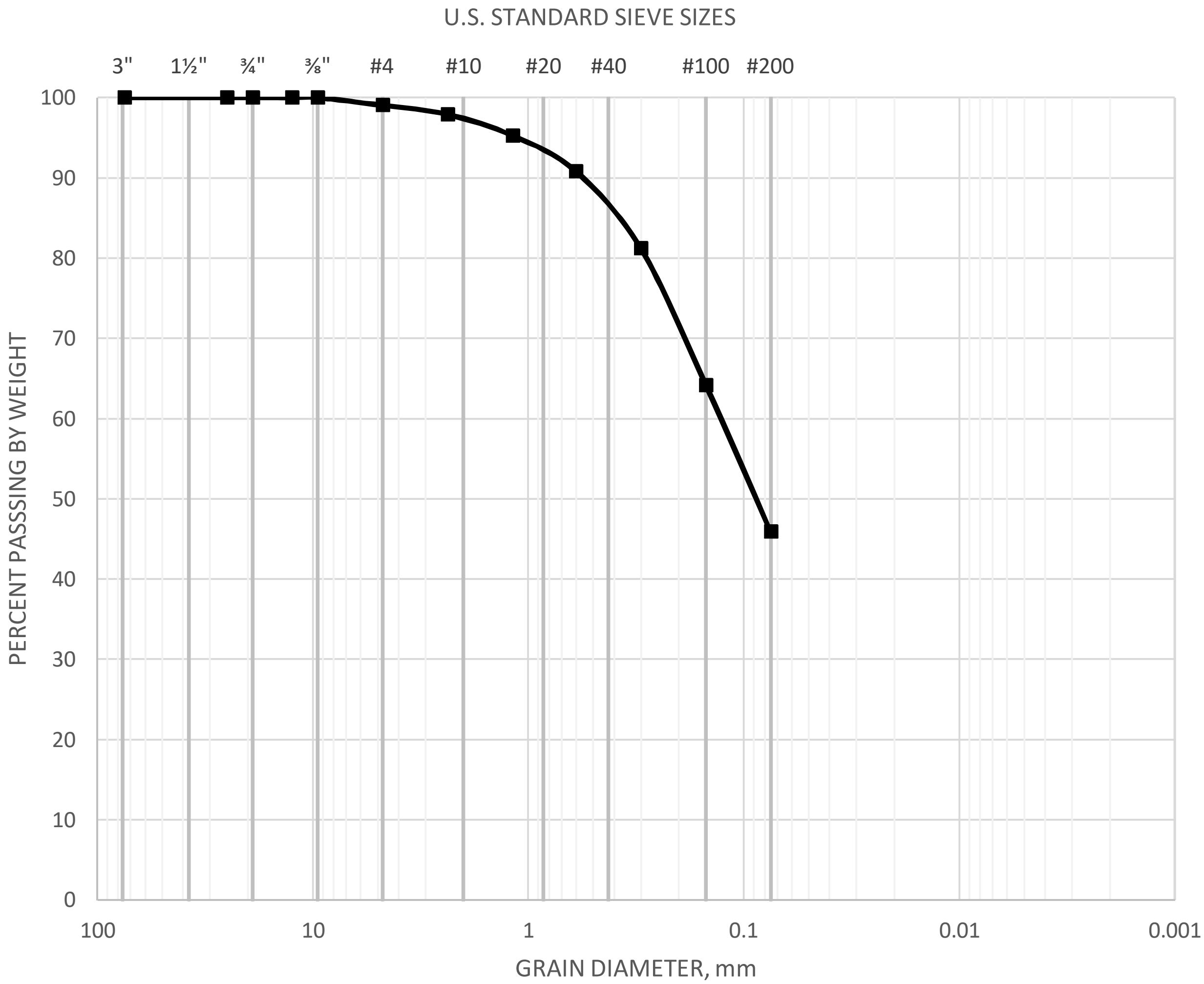
| GEOCON | **GRAIN SIZE DISTRIBUTION** | Project No.: T2214-22-02 |
|---|---|---|
| | ASTM D-422 | RIVANI HEMET 30<br>SOUTH OF HWY 74 & WEST OF JOEL DR<br>HEMET, RIVERSIDE COUNTY, CALIFORNIA |
| | Checked by: | Dec 20 Figure B-2 |




| SAMPLE | CLASSIFICATION | D60 | D30 | D10 |
|---|---|---|---|---|
| P-3@20" | silty SAND with few gravel (SM), grayish brown | 0.79 | 0.2 | 0.055 |

| | | |
|---|---|---|
| GEOCON | **GRAIN SIZE DISTRIBUTION**<br>ASTM D-422 | Project No.: T2214-22-02<br>RIVANI HEMET 30<br>SOUTH OF HWY 74 & WEST OF JOEL DR<br>HEMET, RIVERSIDE COUNTY, CALIFORNIA |
| | Checked by: | Dec 20 Figure B-3 |

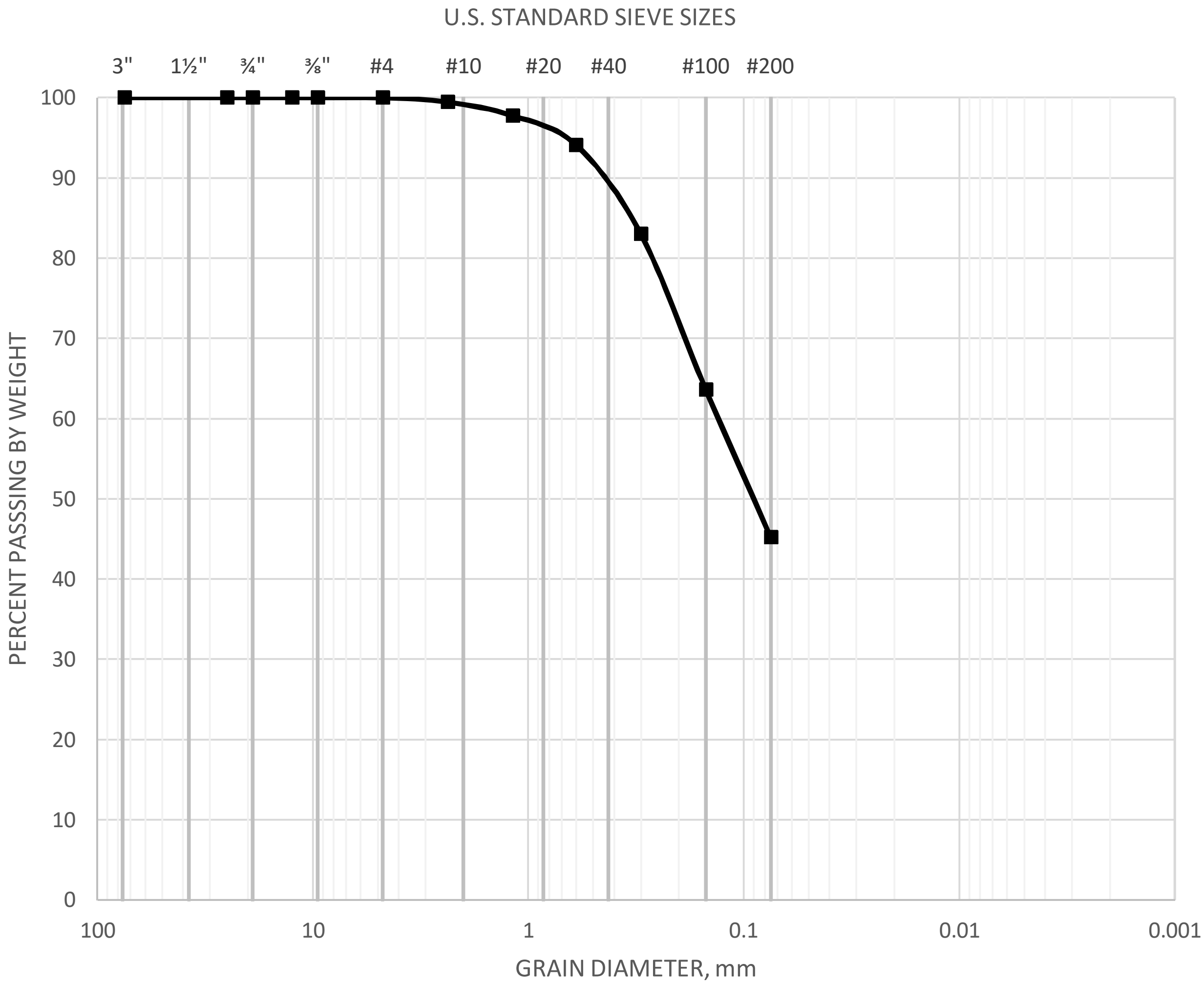
| GRAVEL | | SAND | | | SILT AND CLAY |
|---|---|---|---|---|---|
| COARSE | FINE | COARSE | MEDIUM | FINE | |




| SAMPLE | CLASSIFICATION | D60 | D30 | D10 |
|---|---|---|---|---|
| P-4@20" | silty SAND with trace gravel (SM), brown | 0.14 | 0 | 0 |

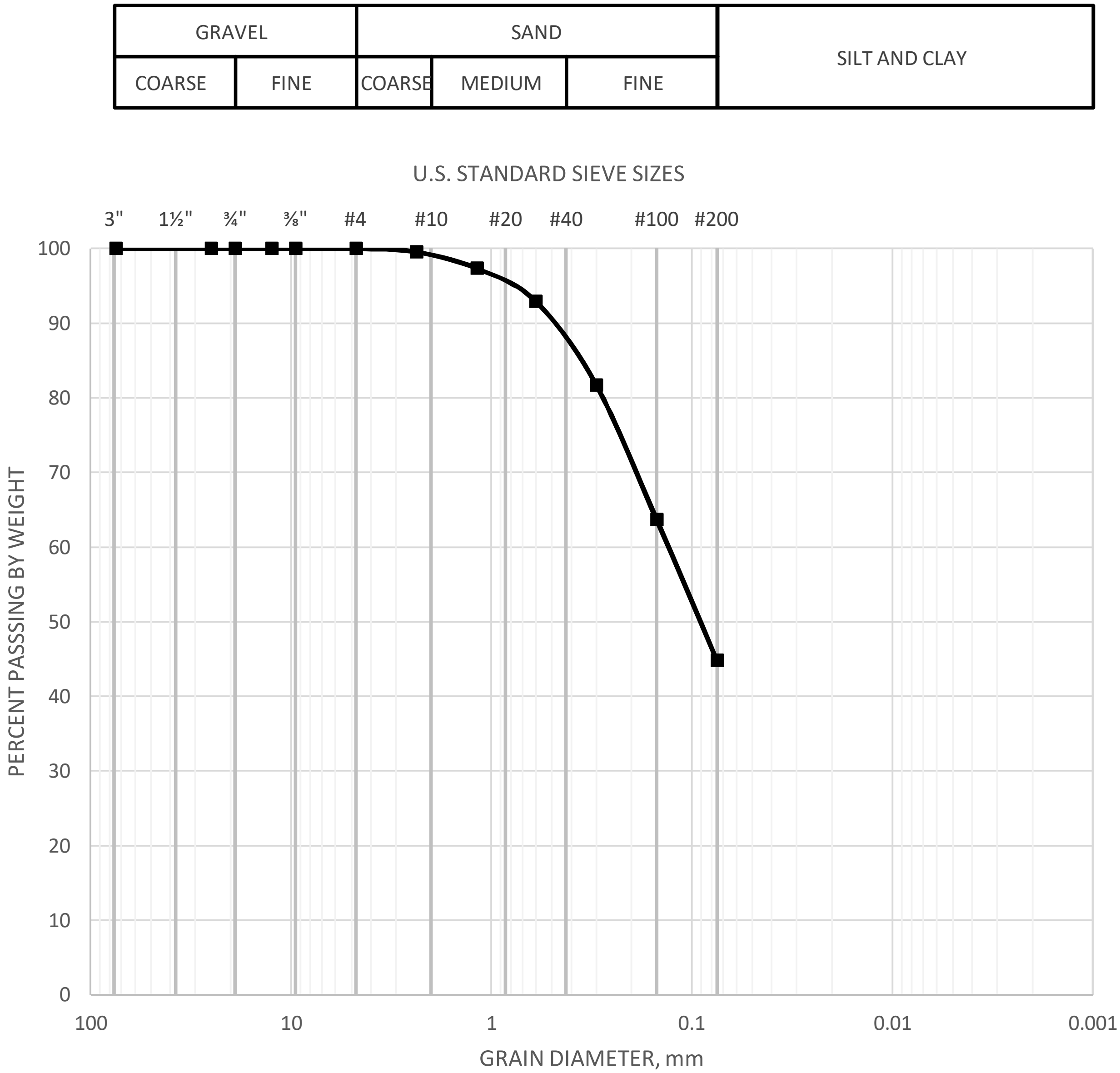
| | | |
|---|---|---|
| GEOCON | **GRAIN SIZE DISTRIBUTION**<br>ASTM D-422 | Project No.: T2214-22-02 |
| | | RIVANI HEMET 30<br>SOUTH OF HWY 74 & WEST OF JOEL DR<br>HEMET, RIVERSIDE COUNTY, CALIFORNIA |
| | Checked by: | Dec 20 Figure B-4 |

| GRAVEL | | SAND | | | SILT AND CLAY |
|---|---|---|---|---|---|
| COARSE | FINE | COARSE | MEDIUM | FINE | |




| SAMPLE | CLASSIFICATION | D60 | D30 | D10 |
|---|---|---|---|---|
| P-5@20" | silty SAND (SM), grayish brown | 0.14 | 0 | 0 |

| GEOCON | **GRAIN SIZE DISTRIBUTION**<br>ASTM D-422 | Project No.: T2214-22-02 |
|---|---|---|
| | | RIVANI HEMET 30<br>SOUTH OF HWY 74 & WEST OF JOEL DR<br>HEMET, RIVERSIDE COUNTY, CALIFORNIA |
| | Checked by: | Dec 20 — Figure B-5 |




| SAMPLE | CLASSIFICATION | D60 | D30 | D10 |
|---|---|---|---|---|
| P-6@20" | silty SAND (SM), grayish brown | 0.14 | 0 | 0 |

GEOCON

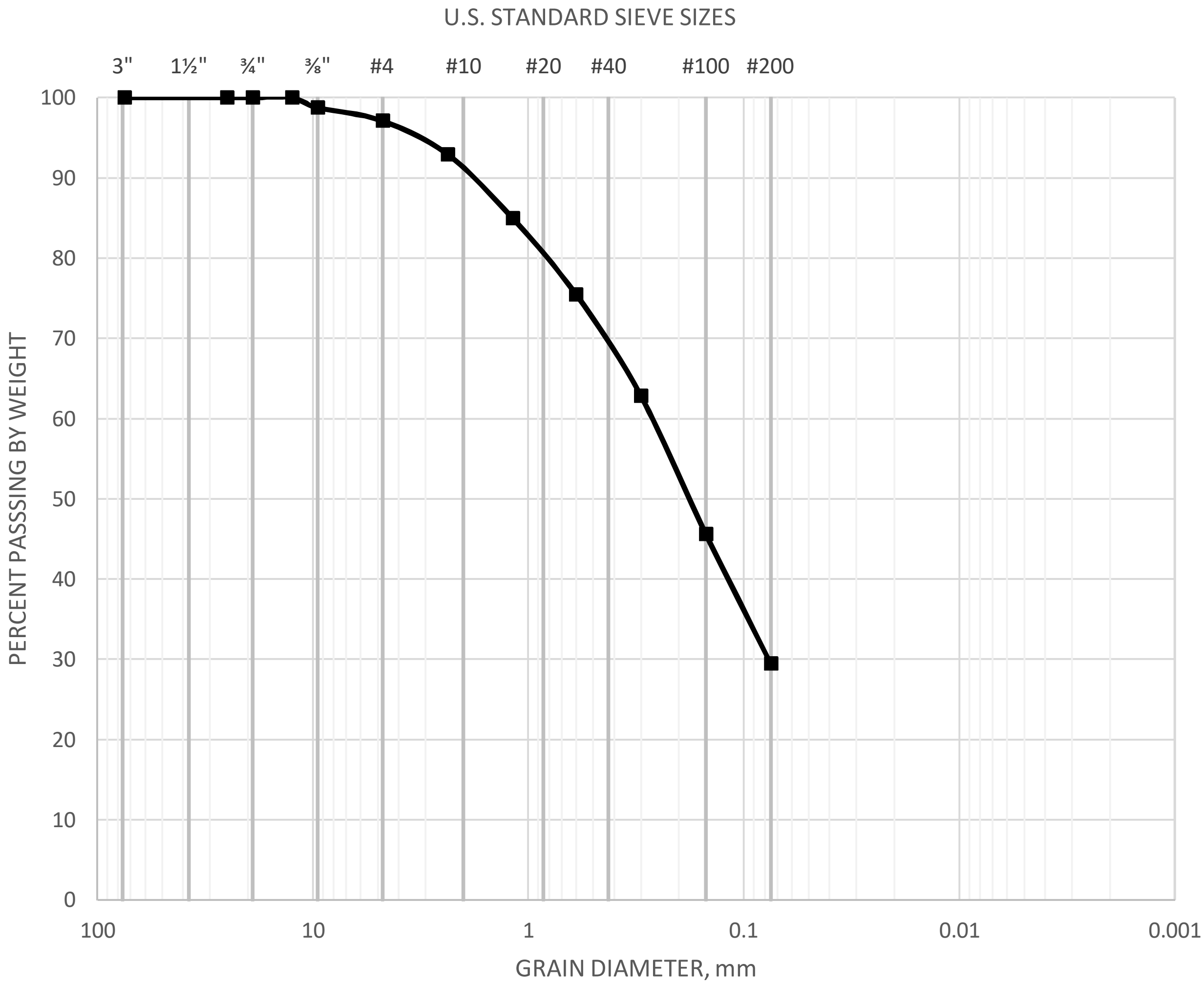
**GRAIN SIZE DISTRIBUTION**

ASTM D-422

Checked by:

| | |
|---|---|
| Project No.: | T2214-22-02 |
| RIVANI HEMET 30<br>SOUTH OF HWY 74 & WEST OF JOEL DR<br>HEMET, RIVERSIDE COUNTY, CALIFORNIA | |
| Dec 20 | Figure B-6 |

| GRAVEL | | SAND | | | SILT AND CLAY |
|---|---|---|---|---|---|
| COARSE | FINE | COARSE | MEDIUM | FINE | |




| SAMPLE | CLASSIFICATION | D60 | D30 | D10 |
|---|---|---|---|---|
| P-7@20" | silty SAND with trace gravel (SM), grayish brown | 0.27 | 0.08 | 0 |

GEOCON

**GRAIN SIZE DISTRIBUTION**

ASTM D-422

Checked by:

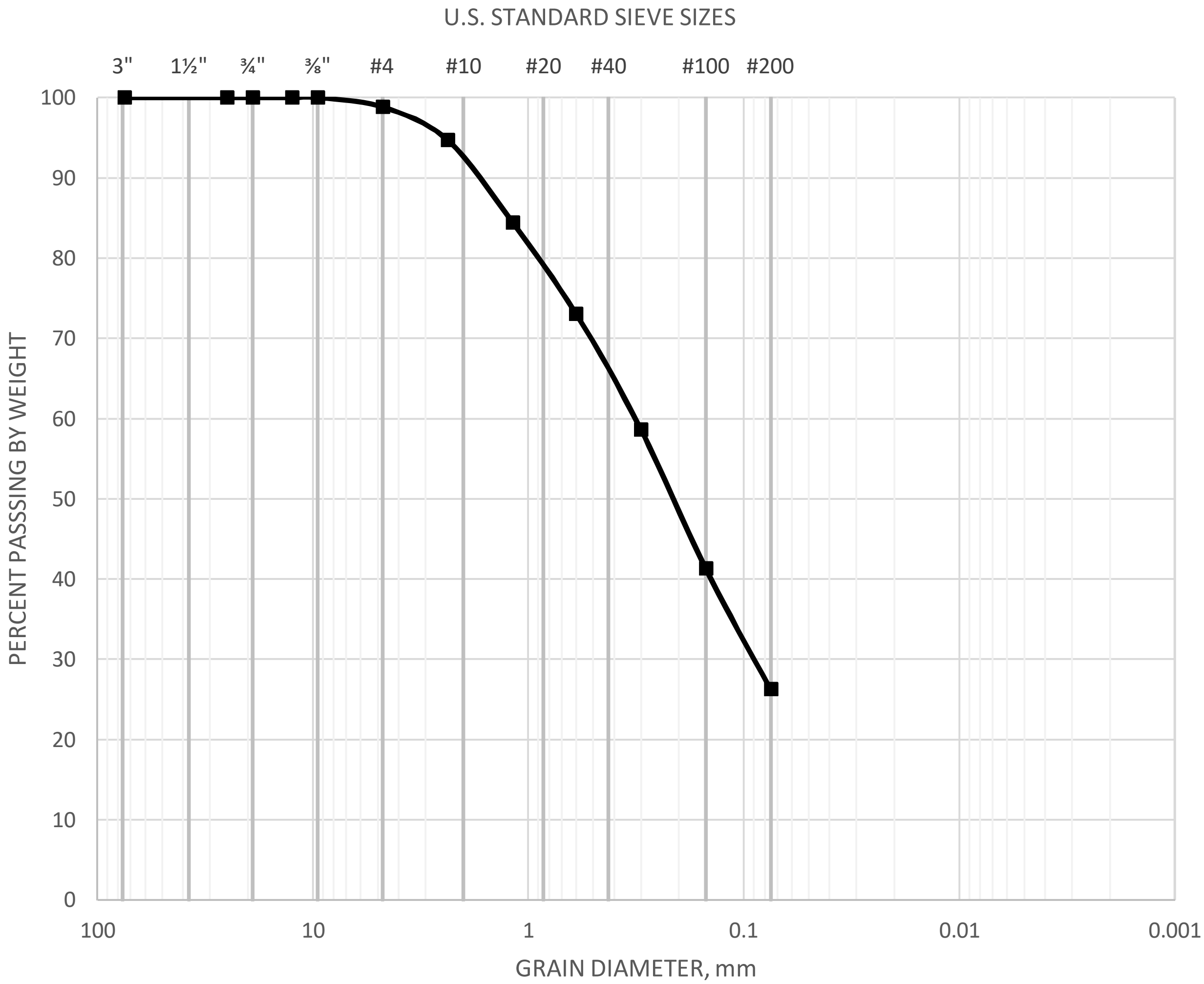
Project No.: T2214-22-02

RIVANI HEMET 30
SOUTH OF HWY 74 & WEST OF JOEL DR
HEMET, RIVERSIDE COUNTY, CALIFORNIA

Dec 20

Figure B-7

| GRAVEL | | SAND | | | SILT AND CLAY |
|---|---|---|---|---|---|
| COARSE | FINE | COARSE | MEDIUM | FINE | |




| SAMPLE | CLASSIFICATION | D60 | D30 | D10 |
|---|---|---|---|---|
| P-8@20" | silty SAND with trace gravel (SM), grayish brown | 0.31 | 0.09 | 0 |

GEOCON

**GRAIN SIZE DISTRIBUTION**

ASTM D-422

Checked by:

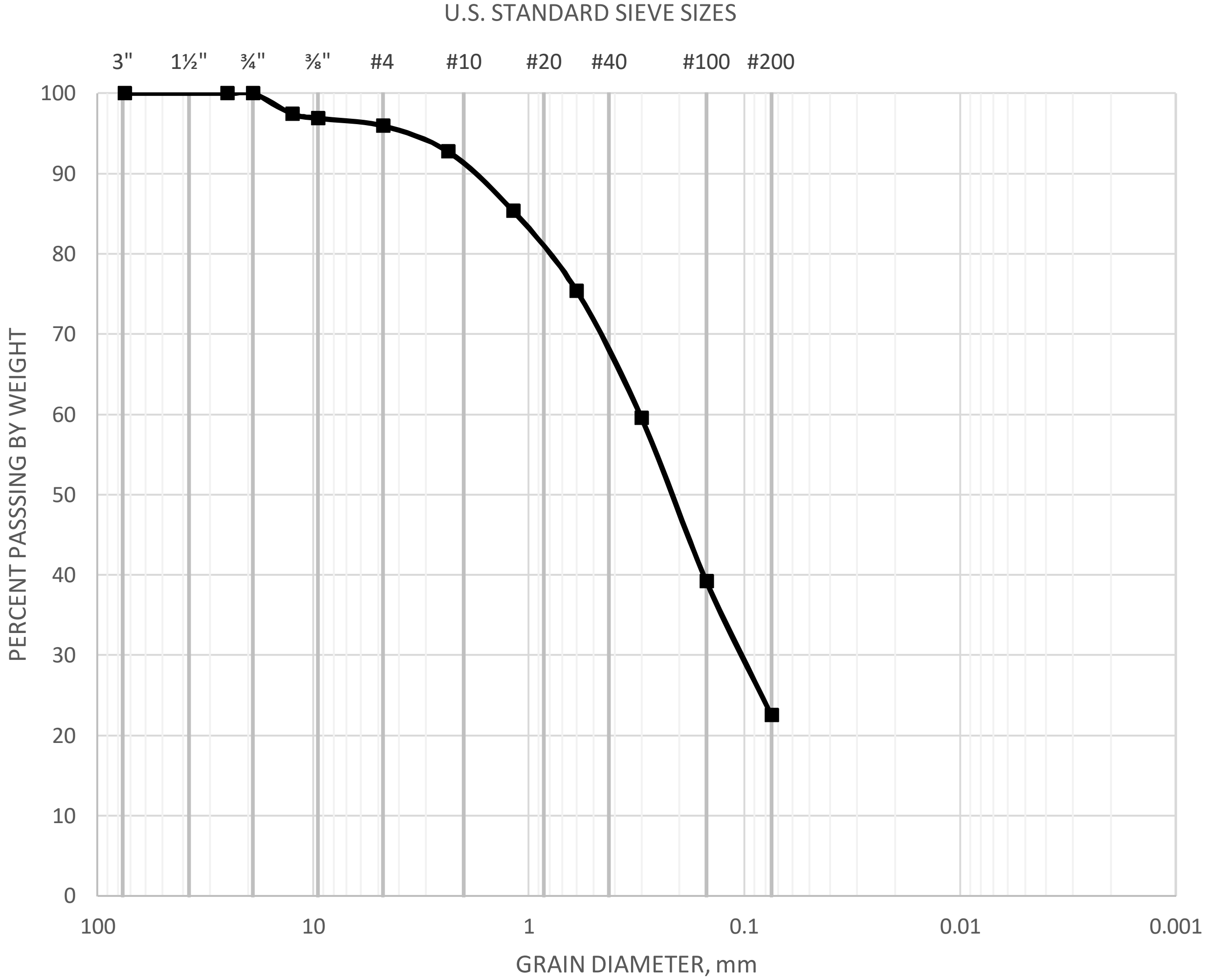
Project No.: T2214-22-02

RIVANI HEMET 30
SOUTH OF HWY 74 & WEST OF JOEL DR
HEMET, RIVERSIDE COUNTY, CALIFORNIA

Dec 20

Figure B-8

| GRAVEL | | SAND | | | SILT AND CLAY |
|---|---|---|---|---|---|
| COARSE | FINE | COARSE | MEDIUM | FINE | |




| SAMPLE | CLASSIFICATION | D60 | D30 | D10 |
|---|---|---|---|---|
| P-9@20" | silty SAND with trace gravel (SM), grayish brown | 0.3 | 0.1 | 0 |

GEOCON

**GRAIN SIZE DISTRIBUTION**

ASTM D-422

Checked by:

Project No.: T2214-22-02

RIVANI HEMET 30
SOUTH OF HWY 74 & WEST OF JOEL DR
HEMET, RIVERSIDE COUNTY, CALIFORNIA

Dec 20

Figure B-9

# APPENDIX B

## LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected chunk samples were tested for their in-place dry density and moisture content and collapse potential. Disturbed bulk samples were tested to determine maximum dry density and optimum moisture content, shear strength and expansion characteristics. Water-soluble sulfate tests were also performed. Results of the laboratory tests are presented in tabular form herewith.

### TABLE B-I
### SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS
### ASTM D 1557-00

| Sample No. | Description | Maximum Dry Density (pcf) | Optimum Moisture Content (% dry wt.) |
|---|---|---|---|
| T2-1 | Orange-Brown, Fine to Coarse Silty SAND | 137.0 | 7.9 |

### TABLE B-II
### SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
### ASTM D4829-95

| Sample No. | Moisture Content | | Dry Density (pcf) | Expansion Index |
|---|---|---|---|---|
| | Before Test (%) | After Test (%) | | |
| T2-1 | 7.4 | 14.8 | 120.7 | 0 |

## TABLE B-III
## SUMMARY OF DIRECT SHEAR TEST RESULTS*
## ASTM D3080-98

| Sample No. | Dry Density (pcf) | Moisture Content (%) | Unit Cohesion (psf) | Angle of Shear Resistance (degrees) |
|---|---|---|---|---|
| T2-1 | 123.3 | 7.8 | 370 | 37 |

*Samples remolded to 90 percent relative compaction at near or slightly above optimum moisture content.

## TABLE B-IV
## SUMMARY OF LABORATORY WATER-SOLUBLE SULFATE TEST RESULTS
## CALIFORNIA TEST NO. 417

| Sample No. | Water-Soluble Sulfate (%) | Sulfate Exposure |
|---|---|---|
| T2-1 | 0.045 | Negligible |

## TABLE B-V
## SUMMARY OF SINGLE-POINT CONSOLIDATION (COLLAPSE) TESTS
## ASTM D-2435-96

| Sample Number | In-situ Dry Density (pcf) | Moisture Content Before Test | Axial Load with Water Added (psf) | Consolidation Before Water Added (%) | Percent Collapse |
|---|---|---|---|---|---|
| T2-2 | 95.3 | 7.4 | 2,000 | 1.7 | 6.7 |
| T3-2 | 108.6 | 9.5 | 2,000 | 1.9 | 3.0 |
| T6-1 | 98.4 | 6.8 | 2,000 | 1.5 | 12.4 |

# APPENDIX C

# APPENDIX C

## RECOMMENDED GRADING SPECIFICATIONS

**FOR**

## HEMET 30
## SOUTH OF HIGHWAY 74 AND EAST OF JOEL DRIVE
## HEMET AREA OF
## RIVERSIDE COUNTY, CALIFORNIA

## PROJECT NO. T2214-22-02

# RECOMMENDED GRADING SPECIFICATIONS

## 1. GENERAL

1.1 These Recommended Grading Specifications shall be used in conjunction with the Geotechnical Report for the project prepared by Geocon. The recommendations contained in the text of the Geotechnical Report are a part of the earthwork and grading specifications and shall supersede the provisions contained hereinafter in the case of conflict.

1.2 Prior to the commencement of grading, a geotechnical consultant (Consultant) shall be employed for the purpose of observing earthwork procedures and testing the fills for substantial conformance with the recommendations of the Geotechnical Report and these specifications. The Consultant should provide adequate testing and observation services so that they may assess whether, in their opinion, the work was performed in substantial conformance with these specifications. It shall be the responsibility of the Contractor to assist the Consultant and keep them apprised of work schedules and changes so that personnel may be scheduled accordingly.

1.3 It shall be the sole responsibility of the Contractor to provide adequate equipment and methods to accomplish the work in accordance with applicable grading codes or agency ordinances, these specifications and the approved grading plans. If, in the opinion of the Consultant, unsatisfactory conditions such as questionable soil materials, poor moisture condition, inadequate compaction, and/or adverse weather result in a quality of work not in conformance with these specifications, the Consultant will be empowered to reject the work and recommend to the Owner that grading be stopped until the unacceptable conditions are corrected.

## 2. DEFINITIONS

2.1 **Owner** shall refer to the owner of the property or the entity on whose behalf the grading work is being performed and who has contracted with the Contractor to have grading performed.

2.2 **Contractor** shall refer to the Contractor performing the site grading work.

2.3 **Civil Engineer** or **Engineer of Work** shall refer to the California licensed Civil Engineer or consulting firm responsible for preparation of the grading plans, surveying and verifying as-graded topography.

2.4 **Consultant** shall refer to the soil engineering and engineering geology consulting firm retained to provide geotechnical services for the project.

2.5 **Soil Engineer** shall refer to a California licensed Civil Engineer retained by the Owner, who is experienced in the practice of geotechnical engineering. The Soil Engineer shall be responsible for having qualified representatives on-site to observe and test the Contractor's work for conformance with these specifications.

2.6 **Engineering Geologist** shall refer to a California licensed Engineering Geologist retained by the Owner to provide geologic observations and recommendations during the site grading.

2.7 **Geotechnical Report** shall refer to a soil report (including all addenda) which may include a geologic reconnaissance or geologic investigation that was prepared specifically for the development of the project for which these Recommended Grading Specifications are intended to apply.

## 3. MATERIALS

3.1 Materials for compacted fill shall consist of any soil excavated from the cut areas or imported to the site that, in the opinion of the Consultant, is suitable for use in construction of fills. In general, fill materials can be classified as *soil* fills, *soil-rock* fills or *rock* fills, as defined below.

3.1.1 **Soil fills** are defined as fills containing no rocks or hard lumps greater than 12 inches in maximum dimension and containing at least 40 percent by weight of material smaller than ¾ inch in size.

3.1.2 **Soil-rock fills** are defined as fills containing no rocks or hard lumps larger than 4 feet in maximum dimension and containing a sufficient matrix of soil fill to allow for proper compaction of soil fill around the rock fragments or hard lumps as specified in Paragraph 6.2. **Oversize rock** is defined as material greater than 12 inches.

3.1.3 **Rock fills** are defined as fills containing no rocks or hard lumps larger than 3 feet in maximum dimension and containing little or no fines. Fines are defined as material smaller than ¾ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.

3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9 and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.

3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.

3.6 During grading, soil or groundwater conditions other than those identified in the Geotechnical Report may be encountered by the Contractor. The Consultant shall be notified immediately to evaluate the significance of the unanticipated condition

## 4. CLEARING AND PREPARING AREAS TO BE FILLED

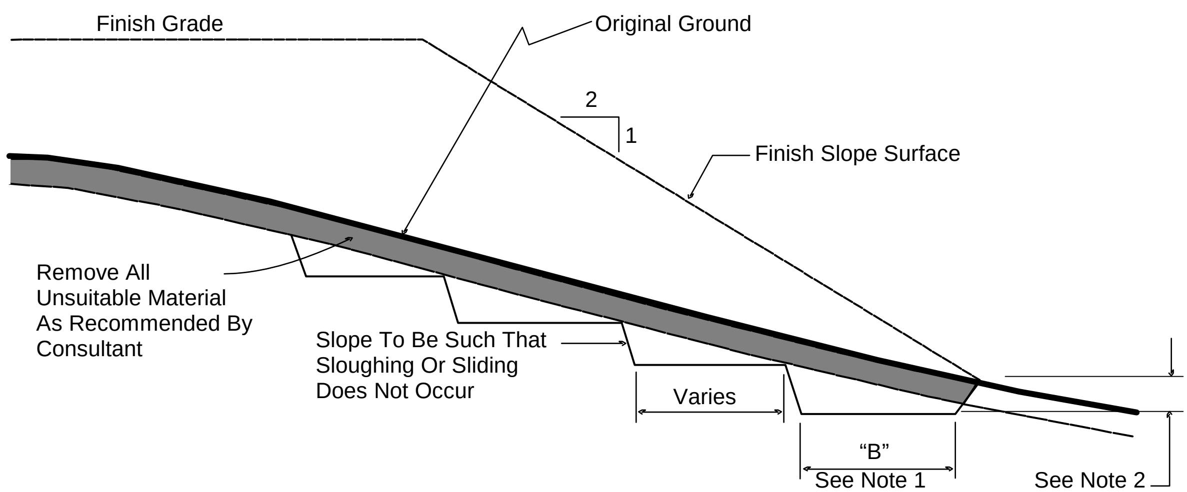
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material and shall be performed in areas to be graded. Roots and other projections exceeding 1½ inches in diameter shall be removed to a depth of 3 feet below the surface of the ground. Borrow areas shall be grubbed to the extent necessary to provide suitable fill materials.

4.2 Asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility or in an acceptable area of the project evaluated by Geocon and the property owner. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

4.3 After clearing and grubbing of organic matter and other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction should be observed and approved by a representative of the Consultant. The exposed surface shall then be plowed or scarified to a minimum depth of 6 inches and until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment to be used.

4.4 Where the slope ratio of the original ground is steeper than 5:1 (horizontal:vertical), or where recommended by the Consultant, the original ground should be benched in accordance with the following illustration.

TYPICAL BENCHING DETAIL




No Scale

DETAIL NOTES:

(1) Key width "B" should be a minimum of 10 feet, or sufficiently wide to permit complete coverage with the compaction equipment used. The base of the key should be graded horizontal, or inclined slightly into the natural slope.

(2) The outside of the key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational material. Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Consultant.

4.5 After areas to receive fill have been cleared and scarified, the surface should be moisture conditioned to achieve the proper moisture content, and compacted as recommended in Section 6 of these specifications.

## 5. COMPACTION EQUIPMENT

5.1 Compaction of *soil* or *soil-rock* fill shall be accomplished by sheepsfoot or segmented-steel wheeled rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Equipment shall be of such a design that it will be capable of compacting the *soil* or *soil-rock* fill to the specified relative compaction at the specified moisture content.

5.2 Compaction of *rock* fills shall be performed in accordance with Section 6.3.

## 6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

6.1 *Soil* fill, as defined in Paragraph 3.1.1, shall be placed by the Contractor in accordance with the following recommendations:

6.1.1 *Soil* fill shall be placed by the Contractor in layers that, when compacted, should generally not exceed 8 inches. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to obtain uniformity of material and moisture in each layer. The entire fill shall be constructed as a unit in nearly level lifts. Rock materials greater than 12 inches in maximum dimension shall be placed in accordance with Section 6.2 or 6.3 of these specifications.

6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557.

6.1.3 When the moisture content of *soil* fill is below that specified by the Consultant, water shall be added by the Contractor until the moisture content is in the range specified.

6.1.4 When the moisture content of the *soil* fill is above the range specified by the Consultant or too wet to achieve proper compaction, the *soil* fill shall be aerated by the Contractor by blading/mixing, or other satisfactory methods until the moisture content is within the range specified.

6.1.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted by the Contractor to a relative compaction of at least 90 percent. Relative compaction is defined as the ratio (expressed in percent) of the in-place dry density of the compacted fill to the maximum laboratory dry density as determined in accordance with ASTM D 1557. Compaction shall be continuous over the entire area, and compaction equipment shall make sufficient passes so that the specified minimum relative compaction has been achieved throughout the entire fill.

6.1.6 Where practical, soils having an Expansion Index greater than 50 should be placed at least 3 feet below finish pad grade and should be compacted at a moisture content generally 2 to 4 percent greater than the optimum moisture content for the material.

6.1.7 Properly compacted *soil* fill shall extend to the design surface of fill slopes. To achieve proper compaction, it is recommended that fill slopes be over-built by at least 3 feet and then cut to the design grade. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.

6.1.8 As an alternative to over-building of slopes, slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Upon completion, slopes should then be track-walked with a D-8 dozer or similar equipment, such that a dozer track covers all slope surfaces at least twice.

6.2 *Soil-rock* fill, as defined in Paragraph 3.1.2, shall be placed by the Contractor in accordance with the following recommendations:

6.2.1 Rocks larger than 12 inches but less than 4 feet in maximum dimension may be incorporated into the compacted *soil* fill, but shall be limited to the area measured 15 feet minimum horizontally from the slope face and 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.

6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.

6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.

6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.

6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.

6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:

6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.

6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.

6.3.3 Plate bearing tests, in accordance with ASTM D 1196, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

6.3.4 A representative of the Consultant should be present during *rock* fill operations to observe that the minimum number of "passes" have been obtained, that water is being properly applied and that specified procedures are being followed. The actual number of plate bearing tests will be determined by the Consultant during grading.

6.3.5 Test pits shall be excavated by the Contractor so that the Consultant can state that, in their opinion, sufficient water is present and that voids between large rocks are properly filled with smaller rock material. In-place density testing will not be required in the *rock* fills.

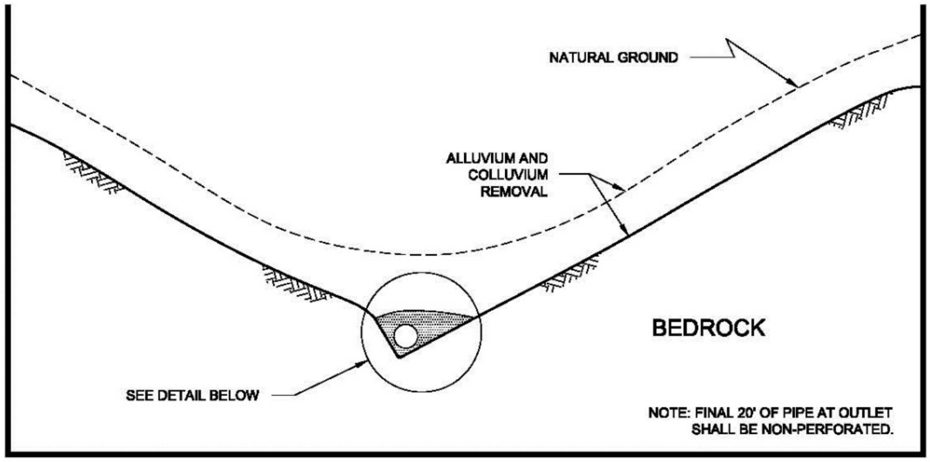
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant prior to commencing grading. The gradation of the graded filter material will be determined at the time the *rock* fill is being excavated. Materials typical of the *rock* fill should be submitted to the Consultant in a timely manner, to allow design of the graded filter prior to the commencement of *rock* fill placement.

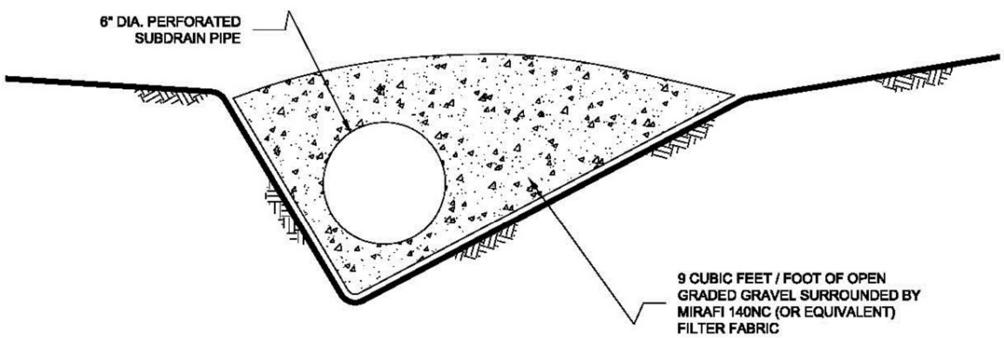
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

## 7. SUBDRAINS

7.1 The geologic units on the site may have permeability characteristics and/or fracture systems that could be susceptible under certain conditions to seepage. The use of canyon subdrains may be necessary to mitigate the potential for adverse impacts associated with seepage conditions. Canyon subdrains with lengths in excess of 500 feet or extensions of existing offsite subdrains should use 8-inch-diameter pipes. Canyon subdrains less than 500 feet in length should use 6-inch-diameter pipes.

TYPICAL CANYON DRAIN DETAIL




**NOTES:**

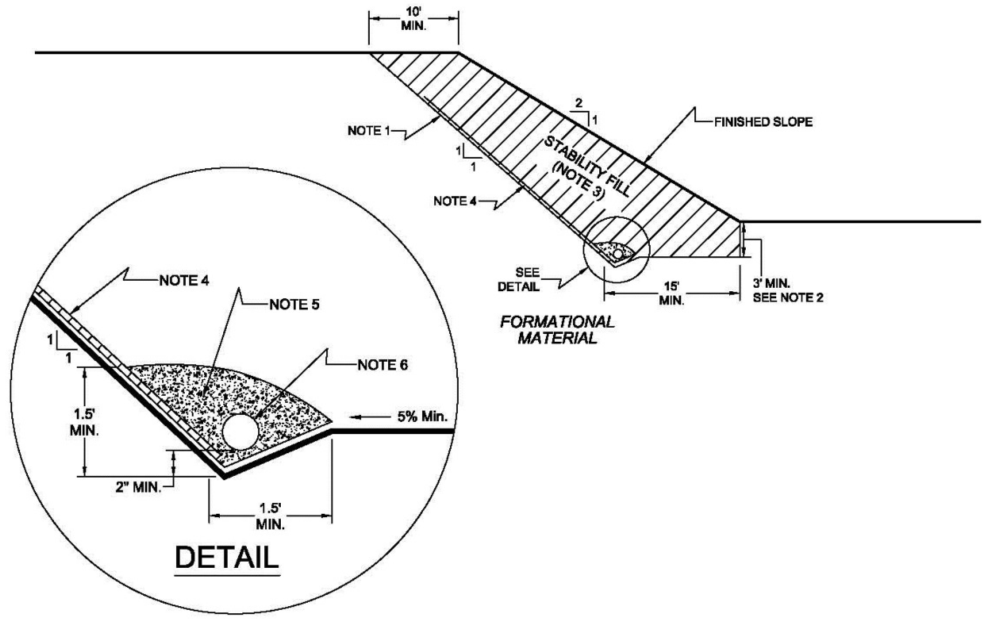
1......8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

2......6-INCH DIAMETER, SCHEDULE 40 PVC PERFORATED PIPE FOR FILLS LESS THAN 100-FEET IN DEPTH OR A PIPE LENGTH SHORTER THAN 500 FEET.

**NO SCALE**

7.2 Slope drains within stability fill keyways should use 4-inch-diameter (or lager) pipes.

## TYPICAL STABILITY FILL DETAIL




**NOTES:**

1.....EXCAVATE BACKCUT AT 1:1 INCLINATION (UNLESS OTHERWISE NOTED).

2.....BASE OF STABILITY FILL TO BE 3 FEET INTO FORMATIONAL MATERIAL, SLOPING A MINIMUM 5% INTO SLOPE.

3.....STABILITY FILL TO BE COMPOSED OF PROPERLY COMPACTED GRANULAR SOIL.

4.....CHIMNEY DRAINS TO BE APPROVED PREFABRICATED CHIMNEY DRAIN PANELS (MIRADRAIN G200N OR EQUIVALENT) SPACED APPROXIMATELY 20 FEET CENTER TO CENTER AND 4 FEET WIDE. CLOSER SPACING MAY BE REQUIRED IF SEEPAGE IS ENCOUNTERED.

5.....FILTER MATERIAL TO BE 3/4-INCH, OPEN-GRADED CRUSHED ROCK ENCLOSED IN APPROVED FILTER FABRIC (MIRAFI 140NC).

6.....COLLECTOR PIPE TO BE 4-INCH MINIMUM DIAMETER, PERFORATED, THICK-WALLED PVC SCHEDULE 40 OR EQUIVALENT, AND SLOPED TO DRAIN AT 1 PERCENT MINIMUM TO APPROVED OUTLET.

NO SCALE

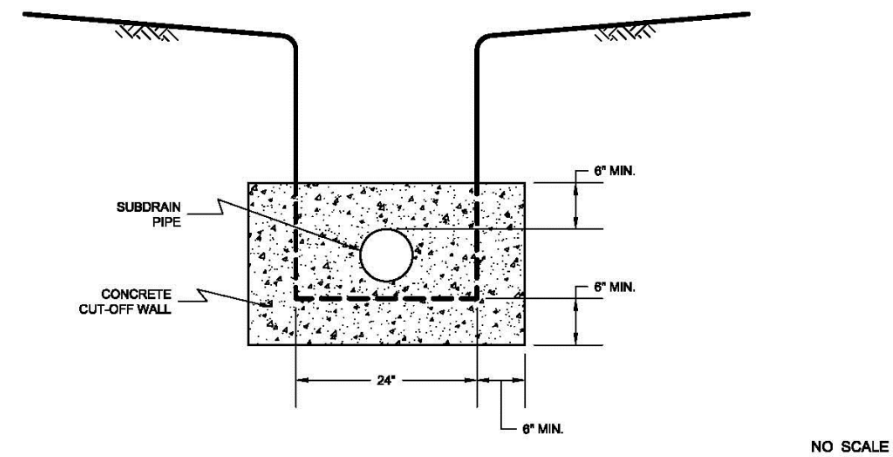
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing 40-scale grading plans.

7.4 *Rock* fill or *soil-rock* fill areas may require subdrains along their down-slope perimeters to mitigate the potential for buildup of water from construction or landscape irrigation. The subdrains should be at least 6-inch-diameter pipes encapsulated in gravel and filter fabric. *Rock* fill drains should be constructed using the same requirements as canyon subdrains.

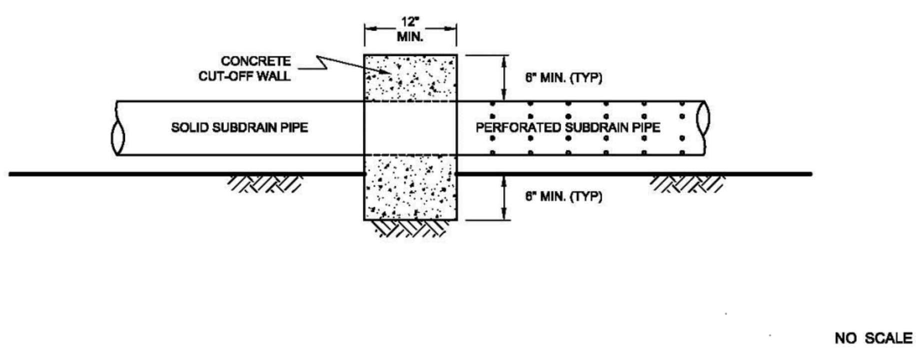
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW

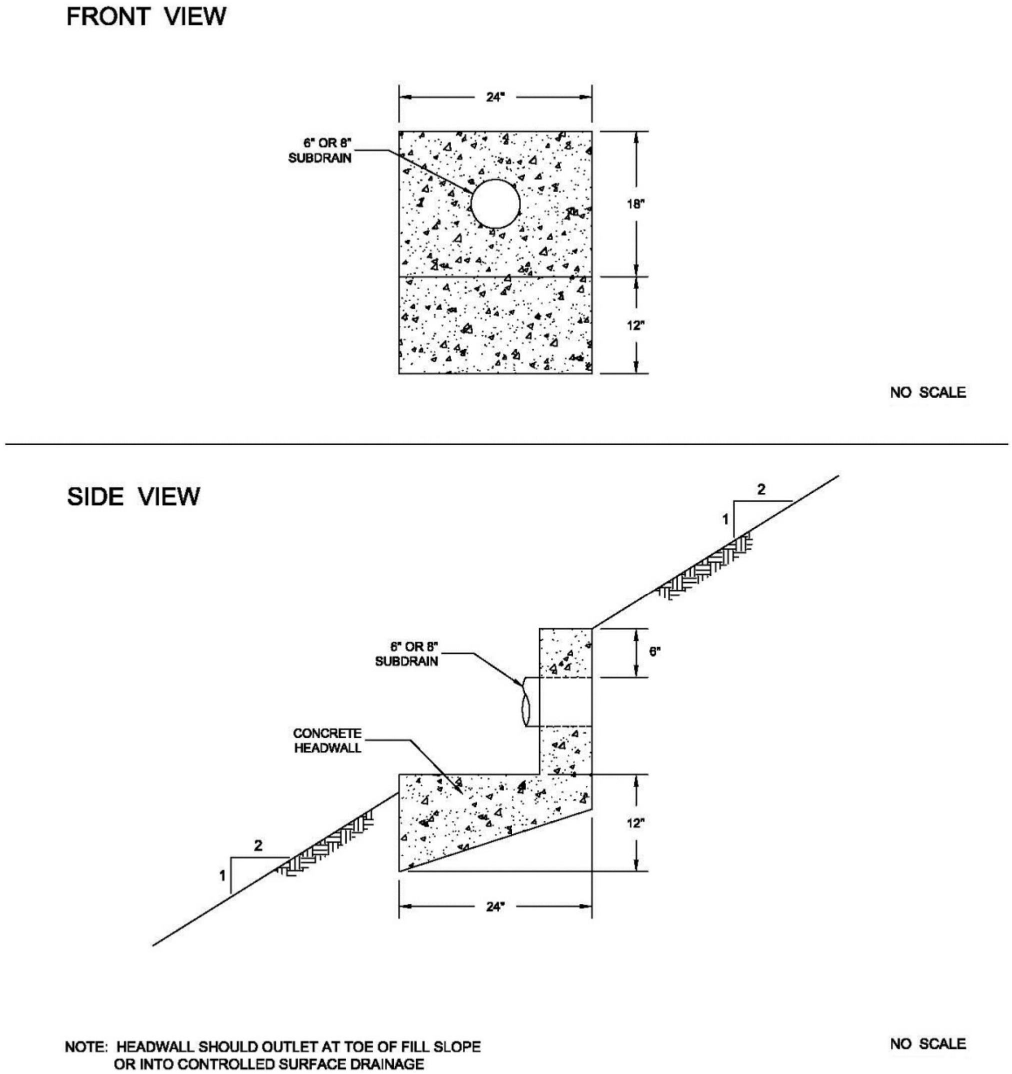



SIDE VIEW




7.6 Subdrains that discharge into a natural drainage course or open space area should be provided with a permanent headwall structure.

TYPICAL HEADWALL DETAIL




7.7 The final grading plans should show the location of the proposed subdrains. After completion of remedial excavations and subdrain installation, the project civil engineer should survey the drain locations and prepare an "as-built" map showing the drain locations. The final outlet and connection locations should be determined during grading operations. Subdrains that will be extended on adjacent projects after grading can be placed on formational material and a vertical riser should be placed at the end of the subdrain. The grading contractor should consider videoing the subdrains shortly after burial to check proper installation and functionality. The contractor is responsible for the performance of the drains.

# 8. OBSERVATION AND TESTING

8.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.

8.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.

8.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.

8.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.

8.5 We should observe the placement of subdrains, to check that the drainage devices have been placed and constructed in substantial conformance with project specifications.

8.6 Testing procedures shall conform to the following Standards as appropriate:

## 8.6.1 Soil and Soil-Rock Fills:

8.6.1.1 Field Density Test, ASTM D 1556, *Density of Soil In-Place By the Sand-Cone Method.*

8.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth)*.

8.6.1.3 Laboratory Compaction Test, ASTM D 1557, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop*.

8.6.1.4. Expansion Index Test, ASTM D 4829, *Expansion Index Test*.

## 9. PROTECTION OF WORK

9.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.

9.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

## 10. CERTIFICATIONS AND FINAL REPORTS

10.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.

10.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.