GEOTECHNICAL UPDATE & PERCOLATION TEST REPORT

WAREHOUSE BUILDING
SOUTHWEST CORNER OF RAMONA EXPRESSWAY & PERRIS BOULEVARD
PERRIS, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

PACIFIC DEVELOPMENT PARTNERS, LLC
SAN JUAN CAPISTRANO, CALIFORNIA

APRIL 28, 2020
PROJECT NO. T2400-22-02

GEOCON
WEST, INC.
GEOTECHNICAL ■ ENVIRONMENTAL ■ MATERIALS

Project No. T2400-22-02
April 28, 2020

Pacific Development Partners, LLC
30220 Rancho Viejo Road, Suite B
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Attention: Mr. Lars Anderson

Subject: GEOTECHNICAL UPDATE & PERCOLATION TEST REPORT
WAREHOUSE BUILDING
SOUTHWEST CORNER OF
RAMONA EXPRESSWAY & PERRIS BOULEVARD
PERRIS, CALIFORNIA

Dear Mr. Anderson:

In accordance with your authorization of Proposal No. IE-2431, Geocon West Inc. (Geocon) herein submits the results of our geotechnical update and percolation test results for the subject site. The accompanying report presents the results of our study, and the conclusions and recommendations pertaining to the geotechnical aspects of the proposed warehouse building. The site is considered suitable for 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 REPORT

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical update and percolation testing as it pertains to the construction of the proposed warehouse building at a site located immediately southwest of the corner of Ramona Expressway and Perris Boulevard, in the City of Perris, California (see *Vicinity Map*, Figure 1). Geocon performed a geotechnical investigation at the site in 2006 which serves as the basis for this update.

The purpose of this study was to evaluate the existing site geology and subsurface soil conditions, identify geologic and geotechnical constraints that may affect development of the property, and provide geotechnical recommendations as they pertain to the proposed development based on the 2019 California Building Code (CBC). The scope of this investigation also included a review of readily available published and unpublished geologic literature (see *List of References*).

The scope of this study included performing a site reconnaissance, drilling and testing of percolation borings, collecting and testing of soil samples, reviewing our 2006 geotechnical report for the site, performing engineering analyses, and preparing this report.

Our original subsurface investigation was performed on August 4 and 7, 2006. We drilled, logged, and sampled eighteen geotechnical borings to depths ranging between 16 and 51½ feet. On March 15 and 16, 2020 we drilled, logged, and sampled seven percolation test borings to depths of 5 and 11 feet in areas where storm water infiltration systems are proposed. The *Geologic Map* (Figure 2) presents the approximate locations of the geotechnical and percolation test borings. *Appendix A* provides a detailed discussion of the field investigation including logs of the borings and percolation test results.

Laboratory testing was performed on select soil samples collected during our field investigations. Our laboratory testing program consisted of in-situ dry density and moisture content, maximum dry density and optimum moisture content, direct shear strength, collapse/swell potential, consolidation characteristics, expansion index/potential, corrosion screening, and grain size distribution. Details of the laboratory tests and a summary of the test results are presented in *Appendix B.*

The recommendations presented herein are based on the engineering evaluation of data obtained from our field investigations and our understanding of the development as presently proposed. If project details vary significantly from those described herein, Geocon should be contacted to evaluate the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The subject site is located on the southwest corner of the intersection of Ramona Expressway and Perris Boulevard, in the City of Perris, California. The site is currently vacant with grass, weeds, and small shrubs within the interior, and some stumps of recently cut trees in the northeast corner. Based on available historic aerial photographs provided by *Historic Aerials* (NETR Online), it appears the site was previously utilized as a sod farm until sometime between 2005 and 2009. Storm water mitigation systems exist on the northwestern and northeastern corners of the site. The existing site grades range from approximately elevation 1,455 feet above Mean Sea Level (MSL) in the east to 1,462 feet above MSL in the west. The site is at latitude 33.8436 and longitude -117.2283.

Based on the referenced *Conceptual Site Plan* (2019) we understand the proposed development will consist of a 352,240-square-foot industrial building with a warehouse and associated offices. Parking and driveway areas will surround the building. Storm water infiltration swales are proposed along the western, northern, and eastern property boundaries. Based on the current site topography and surrounding grades, we anticipate cuts and fills will be on the order of 10 feet or less (exclusive of remedial grading).

Although we have not been provided structural loading information at this time, we expect that the proposed building will generally consist of reinforced concrete tilt-up walls supported on a conventional shallow foundation with a concrete slab-on-grade system, with column loads of up to 200 kips and wall loads of up to 10 kips per linear foot. Our preliminary geotechnical recommendations are based on these load assumptions; Geocon should be contacted to provide additional recommendations if higher loads are used in design.

The findings, conclusions, and recommendations presented herein are based on our site reconnaissance, field investigations and testing, laboratory testing, engineering analyses, and review of published geologic literature. Additionally, if project plans differ from the project descriptions provided herein, Geocon should be contacted for review of the plans and possible revisions to this report.

3. GEOLOGIC SETTING

The subject site, like the rest of southern California, is located within a seismically active region near the margin between the North American and Pacific tectonic plates. The site is located within the Perris Valley which is bounded on the west by the Perris Erosion Surface, the east by several granitic hills and mountains, most notably of which are the Lakeview Mountains, the north by the Box Springs Mountains, and the south by a relatively undefined area of the Menifee Valley (Jenkins, 1965). The Perris Valley is a north-northwest trending alluvial basin which has been filled with sediment emanating from the surrounding bedrock highlands. Drainage within the valley is to the south and west.

Major faults within this area include the San Jacinto Valley (Casa Loma and Claremont branches) and San Bernardino segments of the San Jacinto fault, and the Glen Ivy and Wildomar segments of the Elsinore fault. The Casa Loma fault is nearest to the site. Distances to local faults from the subject site are listed in Table 5.2 of this report.

4. SOIL AND GEOLOGIC CONDITIONS

During our 2006 and current field investigations, we encountered Pleistocene-age very old alluvium to the maximum depth explored of 51½ feet below the ground surface; this geologic unit was encountered across the site in its entirety. This geologic unit is depicted on the *Geologic Map* (Figure 2) and its nomenclature follows that of D.M. Morton (2003).

4.1 Very Old Alluvial Fan Deposits (Qvof)

The very old alluvial fan deposits were encountered in all of our borings from the surface to the maximum depths explored of 51½ feet. As encountered the unit was observed to consist of moist, brown, dark brown, and reddish brown, loose to dense sand with varying amounts of silt and clay. Discontinuous layers of silt and clay were observed within the main body of sand encountered.

5. GROUNDWATER

Groundwater or seepage were not encountered during either of our field investigations (2006 and 2020) at the site. According to the California Department of Water Resources' *Water Data Library*, well data recorded within the last ten years indicates the depth to shallow groundwater to range between 9 and 53 feet below ground surface within two miles of the site. Although groundwater was not encountered during our field investigations, it is not uncommon for seepage conditions to develop where none previously existed. Perched water and seepage are dependent on seasonal precipitation, irrigation, land use, among other factors, and vary as a result. Proper surface drainage will be important to future performance of the improvements.

6. GEOLOGIC HAZARDS

6.1 Faulting

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 CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (Bryant 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 of California Alquist-Priolo Earthquake Fault Zone (APEFZ) or a Riverside County Fault Hazard Zone (RCFHZ) for surface fault rupture hazards. No active or potentially active faults with the potential for surface fault rupture are known to pass directly beneath the site.

According to the *Fault Activity Map of California* (2010), the closest active fault to the site is the Casa Loma fault, located 8 miles southeast of the site. Faults within a 50-mile radius of the site are listed in Table 6.1.

TABLE 6.1
KNOWN ACTIVE FAULTS WITHIN 50 MILES OF THE SITE

Fault Name	Distance from Site (miles)	Direction from Site	Maximum Earthquake Magnitude (Mw)
Casa Loma	8	SE	6.9
Claremont	8	NE	6.7
Main St.	15	SW	6.8
Glen Ivy North	15	SW	6.8
Chino	20	W	6.7
Mill Creek	21	N	7.5
Clark	22	SE	7.2
Whittier	24	W	6.8
San Gorgonio Pass	25	E	7.0
Cucamonga	26	NW	7.0
San Jacinto	28	N	6.8
Glen Helen	28	N	6.7
North Branch	38	N	7.1
Sky Hi Ranch	42	N	7.2
Helendale	42	N	7.3
Coachella	44	E	7.5
Johnson Valley	46	N	6.7
Burnt Mountain	49	NE	6.5
Homestead Valley	50	N	7.3

Historic earthquakes in southern California of magnitude 6.0 and greater, their magnitude, distance, and direction from the site are listed in Table 5.1.2.

TABLE 5.1.2
HISTORIC EARTHQUAKE EVENTS WITH REPECT TO THE SITE

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Near Redlands	July 23, 1923	6.3	11	N
Long Beach	March 10, 1933	6.4	45	WSW
Tehachapi	July 21, 1952	7.5	129	NW
San Fernando	February 9, 1971	6.6	78	WNW
Whittier Narrows	October 1, 1987	5.9	51	WNW
Sierra Madre	June 28, 1991	5.8	53	WNW
Landers	June 28, 1992	7.3	52	ENE
Big Bear	June 28, 1992	6.4	34	NE
Northridge	January 17, 1994	6.7	79	WNW
Hector Mine	October 16, 1999	7.1	76	NE
Ridgecrest China Lake Fault	July 5, 2019	7.1	134	N

6.2 Ground Rupture

Ground surface rupture occurs when movement along a fault is sufficient to cause a gap or rupture where the upper edge of the fault zone intersects that earth surface. The potential for ground rupture is considered to be very low due to the absence of active or potentially active faults at the subject site.

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. Seismically induced 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 of a sufficient level to induce liquefaction.

According to the Riverside County Information Technology (RCIT) *Map My County* public web data, the site is located within an area mapped as having a "low" potential for liquefaction.

We performed a liquefaction analysis of the soils underlying the site using the spreadsheet template LIQ2_30.WQ1 developed by Thomas F. Blake (1996). This program utilizes the 1996 NCEER method of analysis. Our liquefaction potential evaluation was performed by utilizing a groundwater depth of greater than 50 feet, a magnitude 8.1 earthquake, and the site-specific peak horizontal acceleration for the site.

Due to the lack of shallow groundwater, liquefaction is not a design consideration for the site. However, an evaluation of seismically induced "dry-sand" settlement indicates some of the alluvium below the planned improvements and anticipated depth of engineered fill could be prone to seismic settlement during a high-magnitude earthquake. The resulting seismic settlement is estimated to be up to 1½ inch. Differential seismic settlement of the soils is expected to be on the order of ¾ of an inch over a horizontal distance of 30 feet. An analysis of seismically induced "dry-sand" settlement is included on Figure 3.

6.4 Expansive Soil

The geologic units near the ground surface at the site generally consist of sand with lesser extents of silt and clay. Laboratory testing on samples indicated this soil is "non-expansive" as defined by 2019 CBC Section 1803.5.3, with Expansion Indices of 3 and 18 for the site, which are classified as "very low" (Expansion Index [EI] between 0 and 20) in accordance with ASTM D4829.

6.4 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 indicates that potentially collapsible surficial soil exists on the north-central portion of the site in proximity to borings B-15 and B-16, where select samples collected from the borings were tested for hyrdocompression, producing test results of 3.4 and 1.6 percent, respectively, when water was added at a pressure of 2,000 psf. This increased potential for collapse is likely associated with a lower in-situ moisture content when comparing the test results against hydrocompression tests performed on samples collected in the other borings.

6.5 Seiches and Tsunamis

Seiches are large waves which overspill from a large body of water due to aseismic event. The site is located approximately 2.1 miles east-southeast of the Perris Reservoir. Based on the California Department of Water Resources' online *Dam Breach Inundation Map*, an inundation scenario indicates the site could be impacted by flooding.

A tsunami is a series of long-period waves generated in the ocean by a sudden displacement of large volumes of water. Causes of tsunamis include underwater earthquakes, volcanic eruptions, or offshore slope failures. The site is located approximately 37 miles from the Pacific Ocean at an elevation greater than 1,400 feet MSL. Therefore, the risk of tsunamis affecting the site is negligible and not a design consideration.

6.6 Flooding

The site is located in a mapped area of minimal flood hazard, as per information provided by the Federal Emergency Management Agency Flood Map Service Center, *Flood Map 06065C1430H*, effective August 18, 2014.

6.7 Landslides

Due to the relatively level topography at the site, we opine that landslides are not present at the property or at a location that could impact the subject site.

6.7 Rock Fall Hazards

Rock falls are not a design consideration due to the lack of natural bedrock slopes above and adjacent to the site.

6.7 Slope Stability

Although a grading plan was not provided for our review as of the date of this report, we expect that graded slopes on the order of 8 feet or less will be incorporated in the design of the detention basins that are located along the northern, western, and eastern site boundaries. In general, permanent cut and fill slopes, or fill over cut slopes, inclined no steeper than 2:1 (h:v) with slope heights of 8 feet or less will possess Factors of Safety equal to or greater than 1.5 under static loading and 1.1 under pseudo-static loading, assuming they are constructed of on-site materials compacted as recommended herein. Graded slopes should be designed in accordance with the requirements of the local building codes of the City of Perris and the 2019 CBC. Proposed slopes should be reviewed when a grading plan is available and additional recommendations provided as needed.

7. SITE INFILTRATION

Percolation testing was performed in general accordance with Table 1 Infiltration Basin Option 2 of Appendix A of the Riverside County – *Low Impact Development BMP Design Handbook* (Handbook). The percolation tests were performed in general accordance with Section 2.3 *Shallow Percolation Test* (for test holes 10 feet or less in depth) and *Deep Percolation Test* (for test holes greater than 10 feet in depth) methods. Seven percolation tests were conducted within borings P-1 through P-7. The tests were performed at depths of approximately 5 and 11 feet below ground surface. Test borings were drilled using 8-inch-diameter hollow-stem augers. A 3-inch-diameter perforated PVC pipe encased in silt filter sock was placed in each test hole and approximately 2 inches of gravel was placed at the bottom of the perforated PVC pipe. The percolation tests were performed approximately 24 hours after the borings were presaturated. The shallow test holes (5 feet in depth) were filled with a minimum of 20 inches of water, with readings taken at 30-minute intervals. The deep test holes (11 feet in depth) were filled with water to within approximately 4 feet of the ground surface, with readings taken at 30-minute intervals.

The percolation test locations are depicted on the *Geologic Map* (Figure 2). Percolation test logs are presented in *Appendix A* of this report, with the percolation test results summarized in Table 7.0. Percolation test results should be provided to the civil engineer or storm water mitigation system designer. 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 shall 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 FOR PERCOLATION AREAS

Parameter	P-1	P-2	P-3	P-4	P-5	P-6	P-7
Depth (inches)	5	5	11	5	5	11	5
Test Type	Normal	Normal	Normal	Normal	Normal	Normal	Normal
Change in head over time: ΔH (inches)	0.0	0.2	1.1	0.2	0.2	2.3	0.8
Average head: Havg (inches)	30.7	24.5	83.1	22.8	23.9	81.5	26
Time Interval (minutes): Δt (minutes)	30	30	30	30	30	30	30
Radius of test hole: r (inches)	4	4	4	4	4	4	4
Tested Infiltration Rate: It (inches/hour)	0.00	0.04	0.10	0.04	0.04	0.11	0.12

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 From a geotechnical engineering standpoint, the site is suitable for construction of the proposed industrial / warehouse development provided the recommendations presented herein are implemented in design and construction of the project.

8.1.2 Potential geologic hazards at the site include seismic shaking, unsuitable near surface alluvium, hydrocompression, and potentially expansive soils.

8.1.3 The site is located approximately 8 miles from the nearest active fault. Based on our background research and previous investigation, it is our opinion active, potentially active, or inactive faults do not extend across the site. Risks associated with seismic activity consist of the potential for moderate to strong seismic shaking.

8.1.4 Our field investigation indicates the site is underlain by very old alluvial fan deposits. The upper portion of the alluvium across the site is not considered suitable for the support of compacted fill and settlement-sensitive structures. Remedial grading of the surficial soil will be required as discussed herein. The existing 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 Granular soils having little to no cohesion may be subject to caving in un-shored excavations and should be expected at the site.

8.1.6 Remedial grading will address the hydrocompression potential of the near-surface soils on the north-central portion of the site in proximity to borings B-15 and B-16.

8.1.7 Changes in the design, location or elevation of improvements, as outlined in this report, should be reviewed by this office. Once final grading plans become available, they should be reviewed by this office to evaluate the necessity for review and possible revision of this report.

8.2 Excavation and Soil Characteristics

8.2.1 The *in-situ* soils should generally be excavatable with moderate effort using conventional earth moving equipment in proper functioning order.

8.2.2 The soils encountered during this investigation should be considered "non-expansive" (expansion index [EI] of 20 or less) as defined by the 2019 CBC, Section 1813.5.3. Table 8.2.2 presents soil classifications based on the expansion index. Based on the laboratory test results, we expect that the soil encountered will possess a "very low" expansion potential (EI between 0 and 20). Should medium to highly expansive soils be encountered at the site, they should be selectively graded to not be placed within 4 feet of the proposed improvements.

TABLE 8.2.2
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	

8.2.3 Laboratory tests were performed on a representative sample of the site materials to measure the percentage of water-soluble sulfate content. *Appendix B* presents results of the laboratory water-soluble sulfate content tests. Test results indicate the on-site materials tested possess a sulfate content of up to 0.014% (140 parts per million [ppm]) equating to an exposure class of "S0" to concrete structures as defined by 2019 CBC Section 1904.3 and ACI 318. Table 8.2.3 below presents a summary of concrete requirements set forth by 2019 CBC Section 1904.3 and ACI 318.

TABLE 8.2.3
REQUIREMENTS FOR CONCRETE
EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Sulfate 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 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 along the access roads or from nearby developments (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

8.2.5 Laboratory testing indicates the site soils have a minimum electrical resistivity of 811 ohm-cm, possess up to 340 parts per million (ppm) chloride, possess up to 140 ppm sulfate, and have a low tested pH of 6.5. As shown in Table 8.2.5 below, the site would be classified as "corrosive" to buried improvements, in accordance with the Caltrans Corrosion Guidelines (Caltrans, 2018).

TABLE 8.2.5
CALTRANS CORROSION GUIDELINES

Corrosion Exposure	Resistivity (ohm-cm)	Chloride (ppm)	Sulfate (ppm)	pH
Corrosive	**<1,100**	500 or greater	1,500 or greater	5.5 or less

8.2.6 Geocon does not practice in the field of corrosion engineering; therefore, based on the corrosivity of site soils, further evaluation by a corrosion engineer should be performed for site improvements susceptible to corrosion.

8.3 Grading

8.3.1 Earthwork operations should be observed and the compacted fill tested by representatives of Geocon.

8.3.2 Grading should be performed in accordance with the recommendations provided herein, the *Recommended Grading Specifications* contained in *Appendix C* of this report, and the grading ordinances of the City of Perris.

8.3.3 A preconstruction conference should be held at the site prior to the beginning of grading operations with a representative of the City of Perris, contractor, civil engineer, and geotechnical engineer in attendance. Special soil handling requirements can be discussed at that time.

8.3.4 Site preparation should commence with the removal of existing improvements from areas to be graded. The areas to receive compacted fill shall be stripped of vegetation, existing undocumented fill (if present), and loose or disturbed soils.

8.3.5 The upper portion of alluvium within a 1:1 (h:v) projection of the limits of grading should be removed to expose competent alluvium having a minimum of 85 percent relative compaction as determined by ASTM D1557. Removals in proposed building structure areas should extend to depths on the order of 4 to 8 feet below the ground surface, or at least 3 feet below the bottom of planned foundations; remedial removal depths for structural areas are depicted on the *Geologic Map* (Figure 2). Removals in pavement and walkway areas should extend at least 3 feet below subgrade and into competent alluvium. Areas of loose, dry, or compressible soils will require a deeper excavation and processing prior to fill placement. The actual depth of removal should be evaluated by the engineering geologist during grading operations. Where over-excavation and compaction is to be conducted, the excavations should be extended laterally beyond the building footprint for a minimum distance of 5 feet or a distance equal to the depth of removal, whichever is greater. The bottom of the excavations should be scarified to a depth of at least 1 foot, moisture conditioned to 0 to 2 percent above optimum moisture content, and properly compacted to at least 90 percent of the maximum dry density as determined by ASTM D1557.

8.3.6 Where relatively loose, soft, or wet soils are encountered in the site excavations, subgrade stabilization will be required prior to placing fill or installing utilities. Where required, subgrade stabilization can be achieved by over-excavating the loose or soft materials and replacing with compacted fill, placing a reinforcing geogrid at the bottom of the excavation, placing 3-inch diameter rock in the soft bottom and working the rock into soil until it is stabilized, placing gravel wrapped in filter fabric at the bottom of the excavation, or other method recommended by the contractor with guidance by the engineering geologist based on the conditions encountered. Where used, gravel should consist of a 12- to 18- inch thick layer of washed angular ¾ inch gravel atop a filter fabric (Mirafi 500X or equivalent) on the excavation bottom. The filter fabric should be placed in a manner so that the gravel does not have direct contact with the soil. Once the gravel is placed and vibrated to a relatively dense state, a top layer of filter fabric should be placed to cover the gravel. Recommendations for stabilizing excavation bottoms should be based on an evaluation in the field by Geocon at the time of construction.

8.3.7 The site soils are suitable for re-use as an engineered fill provided oversize material (greater than 6 inches) and deleterious debris is removed. Deleterious debris must not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the geotechnical engineer. Existing underground improvements planned for removal should be excavated and the resulting depressions properly backfilled in accordance with the procedures described herein.

8.3.8 Import fill (if necessary) should consist of granular materials with a "low" expansion potential (EI of 50 or less), less corrosive than onsite soils, generally free of deleterious material and contain no rock fragments larger than 6 inches. Geocon should be notified of the import soil source and should perform geotechnical laboratory testing of import soil to evaluate its suitability prior to its arrival at the site for use as fill material. Environmental testing of import fill should be performed by the project environmental consultant in accordance with City of Perris requirements.

8.3.9 Excavated site soils should be thoroughly blended and moisture conditioned prior to placement and compaction. Fill and backfill soils should be placed in horizontal loose layers no thicker than will allow for adequate bonding and compaction (approximately 6 to 8 inches thick), moisture conditioned to 0 to 2 percent above optimum moisture content, and compacted to at least 90 percent of the maximum dry density, as determined by ASTM D1557. Fill materials placed below the moisture content recommended will require additional moisture conditioning prior to placing additional fill.

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 with similar site soils, the shrinkage of the alluvium is expected to be on the order of 5 to 10 percent, when compacted to at least 90 percent of the laboratory maximum dry density. 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 City of Perris 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 sands (Sand Equivalent greater than 30) to a depth of at least one foot over the pipe. <u>The use of uniformly graded crushed rock is only acceptable if used in conjunction with filter fabric to prevent the gravel from having direct contact with soil.</u> The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. 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 0 to 2 percent 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. The data was calculated using the online application *Seismic Design Maps*, provided by OSHPD. 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 below are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 8.6.1
2019 CBC SEISMIC DESIGN PARAMETERS

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	0.579g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.0	Table 1613.2.3(1)
Site Coefficient, F_V	*1.721	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.5g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	*0.996	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.0g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	*0.664	Section 1613.2.4 (Eqn 16-39)

Note: Per Section 11.4.8 of ASCE/SEI 7-16, a ground motion hazard analysis shall be performed for projects for Site Class "E" sites with Ss greater than or equal to 1.0g and for Site Class "D" and "E" sites with S1 greater than 0.2g. Section 11.4.8 also provides exceptions which indicates that the ground motion hazard analysis may be waived provided the exceptions are followed. Using the code based values presented in the table above, in lieu of performing a ground motion hazard analysis, requires the exceptions outlined in ASCE 7-16 Section 11.4.8 be followed.

*See Section 11.4.8

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 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.5g	Figure 22-7
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.55g	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.

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 (v4.2.0). 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 hypocentral distance of 13.7 kilometers from the site.

8.6.5 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 Shallow Foundation and Concrete Slabs-On-Grade

8.7.1 The foundation recommendations presented herein are for the proposed building subsequent to the recommended grading. We understand that the future building will be supported on a conventional shallow foundation with concrete slabs-on-grade, deriving support in newly placed engineered fill.

8.7.2 The foundation for the structure may consist of either continuous strip footings and/or isolated spread footings. Conventionally reinforced continuous footings should be at least 24 inches wide and extend at least 2 feet below lowest adjacent pad grade. Isolated spread footings should have a minimum width of 48 inches and should extend at least 2 feet below lowest adjacent pad grade. At least 4 feet of compacted fill should be placed below the bottom level of foundations (see the *Grading* section of this report for earthwork recommendations). Footings subject to heavy structural loading should be tied-up to each other by tie beams and/or grade beams. A wall/column footing dimension detail depicting footing embedment is provided on Figure 4.

8.7.3 From a geotechnical engineering standpoint, concrete slabs-on-grade for the structure should be at least 4 inches thick and be reinforced with at least No. 3 steel reinforcing bars placed 24 inches on center in both directions. The concrete slab-on-grade recommendations are based on soil support characteristics only. The project structural engineer should evaluate the structural requirements of the concrete slab for supporting equipment and storage loads. A thicker concrete slab may be required for heavier loading conditions. To reduce the effects of differential settlement on the foundation system, thickened slabs and/or an increase in steel reinforcement can provide a benefit to reduce concrete cracking

8.7.4 Following remedial grading, foundations for the buildings may be designed for an allowable soil bearing pressure of 3,000 psf (dead plus live load). The allowable bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.

8.7.5 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 on the order of 1¾ inch and to occur below the heaviest loaded structural element, with differential static settlement to be on the order of ~~¾~~ 1 inch over a horizontal distance of 40 feet; settlement of the foundation system is expected to occur on initial application of loading. Seismic settlement is estimated to be on the order of 1½ inch, with differential seismic settlement to be on the order of ¾ of an inch over a horizontal distance of 30 feet.

8.7.6 Once the design and foundation loading configuration proceeds to a more finalized plan, the estimated settlements within this report should be reviewed and revised, if necessary.

8.7.7 Steel reinforcement for continuous footings should consist of at least two No. 4 steel reinforcing bars placed horizontally in the footings, one near the top and one near the bottom. Steel reinforcement for the spread footings should be designed by the project structural engineer.

8.7.8 Foundation excavation bottoms must be observed and approved in writing by a qualified representative of Geocon, prior to placement of reinforcing steel or concrete.

8.7.9 Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder. The vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06). The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity-controlled environment.

8.7.10 The bedding sand thickness should be evaluated 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. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

8.7.11 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisturized to maintain a moist condition between 0 and 2 percent above optimum moisture content.

8.7.12 The recommendations of this report are intended to reduce the potential for cracking of slabs due to expansive soil (if present), differential settlement of existing soil or soil with varying thicknesses. However, even with the incorporation of the recommendations presented herein, foundations, 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 and/or controlled 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.

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

8.8 Miscellaneous Foundations

8.8.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures which will not be tied to the proposed structure may be supported on conventional shallow foundations bearing on a minimum of 2 feet of newly placed engineered fill which extends laterally at least 2 feet beyond the foundation area. Where excavation and compaction cannot be performed or is undesirable, such as adjacent to property lines, foundations may derive support in the undisturbed alluvium generally found at or below a depth of 3 feet, and should be deepened as necessary to maintain a minimum 5 foot embedment below grade.

8.8.2 If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width and a minimum of 24 inches in depth below the lowest adjacent grade, bearing on the recommended thickness of engineered fill. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

8.8.3 Foundation excavations should be observed and approved in writing by the geotechnical engineer, prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated.

8.9 Retaining Walls

8.9.1 The recommendations presented below are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 10 feet that have been backfilled with select granular site soils or import with a “low” expansion potential (EI of 50 or less). In the event that cantilever walls higher than 10 feet are planned, Geocon should be contacted 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 40 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 65 pcf is recommended. These soil pressures assume 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, the walls should be designed for a soil pressure equivalent to the pressure exerted by a fluid density of 62 pcf.

8.9.4 The structural engineer should determine the seismic design category for the project in accordance with Section 1613 of the 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 (Section 1803.5.12 of the 2019 CBC).

8.9.5 An incremental seismic load of 25 pcf should be used for design of walls that support more than 6 feet of backfill in accordance with Section 1803.5.12 of the 2019 CBC. The pressure should be taken as an inverted triangular distribution with the zero-pressure point at the toe of the wall and 25H (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 7-10 Section 11.8.3.

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. Typical retaining wall drainage details are shown on Figure 5. 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 assume a properly compacted backfill (EI of 50 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 Design

8.10.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. A passive pressure exerted by an equivalent fluid weight of 300 pounds per cubic foot (pcf) with a maximum earth pressure of 3,000 psf should be used for the design of footings or shear keys poured neat against newly compacted fill. The allowable passive pressure assumes 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 newly compacted fill soil and concrete of 0.35 should be used for design. When combining passive pressure and friction for lateral resistance, the passive component should be reduced by one-third.

8.11 Exterior Concrete Flatwork

8.11.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein assuming the subgrade materials possess a “low” expansion potential (expansion index of 50 or less). Subgrade soils should be compacted to 90 percent relative compaction, at 0 to 2 percent 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 24 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.

8.11.2 The exterior flatwork has the potential for distress should the subgrade soils become wet or saturated. 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.

8.11.3 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 or differential settlement. The steel reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork.

8.11.4 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.

8.11.5 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.

8.12 Preliminary Pavement Recommendations

8.12.1 The final pavement sections for driveways and parking lot areas should be based on the R-value of the subgrade soils encountered at final subgrade elevation. The civil engineer should evaluate the final traffic index for the pavements. Pavements should be designed and constructed in accordance with County of Riverside *Ordinance 461* when final Traffic Indices and R-value test results of subgrade soil are completed. We have assumed an R-value of 30 for on-site soils and have utilized an R-Value of 78 for Class 2 Aggregate Base material, for the purposes of this preliminary analysis. Preliminary flexible pavement sections are presented in Table 8.12.1.

TABLE 8.12.1
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Location	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Parking Lots and Access Roads - **Light** Vehicular Traffic Loads and Equipment	6.0	30	4	8
Parking Lots and Access Roads – **Medium and Heavy** Vehicular Traffic Loads and Equipment	9.0	30	6	12

8.12.2 The upper 12 inches of the subgrade soil should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density at 0 to 2 percent over optimum moisture content beneath pavement sections.

8.12.3 Prior to construction of new pavement sections, remedial grading should be performed in accordance with the earthwork recommendations in this report. Asphalt concrete should conform to Section 203-6 of the Greenbook. Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation"* (Caltrans). Aggregate base materials should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Asphalt concrete should be compacted to a density of 95 percent of the laboratory Hveem density in accordance with ASTM D 1561.

8.12.4 A rigid Portland cement concrete (PCC) pavement section should be placed in driveway aprons and cross gutters, and may be used in driveways and parking areas where desired. 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	C and D
Average daily truck traffic, ADTT	300 and 700

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)
Light Truck Traffic (TC = C)	7.5
Medium and Heavy Truck Traffic (TC = D)	8.0

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 at 0 to 2 percent above optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,500 psi (pounds per square inch). Aggregate 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 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 The performance of pavement 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, the perimeter curb should extend at least 6 inches below the level of the base materials.

8.13 Elevator Pit Design

8.13.1 If used, the elevator pit slab and retaining walls should be designed by the project structural engineer. Elevator pit slab and walls may be designed in accordance with the recommendations in the foundation and retaining wall sections of this report.

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

8.13.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the retaining wall section of this report, and the typical retaining wall drainage details shown on Figure 5.

8.13.4 We recommend that the exterior walls and slab be waterproofed to prevent excessive moisture inside of the elevator pit. Waterproofing design and installation is not the responsibility of the geotechnical engineer.

8.14 Elevator Piston

8.14.1 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to a foundation or shoring pile, or the drilled excavation could compromise the existing foundation or pile support, especially if the drilling is performed subsequent to the foundation or pile construction.

8.14.2 Some caving is expected and the contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the geotechnical engineer is required.

8.14.3 The annular space between the piston casing and drilled excavation wall should be filled with a minimum of 2-sack slurry pumped from the bottom up. As an alternative, pea gravel may be utilized. The use of soil to backfill the annular space is not acceptable.

8.15 Temporary Excavations and Shoring

8.15.1 Excavations of up to 10 feet in vertical height are expected during the construction of the site improvements. The contractor’s competent person should evaluate the necessity for lay back of vertical cut areas. Vertical excavations up to 5 feet may be attempted where loose soils or caving sands are not present, and where not surcharged by existing structures or vehicle/construction equipment loads.

8.15.2 Vertical excavations greater than 5 feet will require sloping or shoring measures in order to provide a stable excavation. Due to existing improvements adjacent to the site and the relatively loose nature of the site soils, we expect shoring will be needed.

8.15.3 We expect that braced shoring, such as conventionally braced shields, cross-braced hydraulic shoring, or driven sheet piles 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.

8.15.4 We recommend that an equivalent fluid pressure based on the table below be utilized for design of temporary shoring. These pressures are based on the assumption that the shoring is supporting a level backfill and there are no hydrostatic pressures above the bottom of the excavation.

TABLE 8.15.4
RECOMMENDED SHORING PRESSURES

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 10	35	60	55

8.15.5 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.

8.15.6 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.

8.15.7 In addition to the recommended earth pressure, the upper 5 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 assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected. Higher surcharge loads may be required to account for construction equipment.

8.15.8 It is difficult to accurately predict the amount of deflection of a shored embankment. 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, we recommend 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.

8.16 Surface Drainage

8.16.1 Proper 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.

8.16.2 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.

8.16.3 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 be used to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes. In addition, where landscaping is planned adjacent to 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.

8.16.4 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. Down-gradient and adjacent structures may be subjected to seeps, movement of foundations and slabs, or other impacts as a result of water infiltration.

8.17 Plan Review

8.17.1 Geocon should review the grading and foundation plans for the project prior to final submittal to verify that the plans have been prepared in substantial conformance with the recommendations of this report. Additional analyses may be required after review of the project plans.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. 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 the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon West, Inc. should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon West, Inc.

2. 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 engineer and contractor 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.

3. The findings of this report are valid as of the date of this report. 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.

4. 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 assume 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 assume the role of Geotechnical Engineer of Record.

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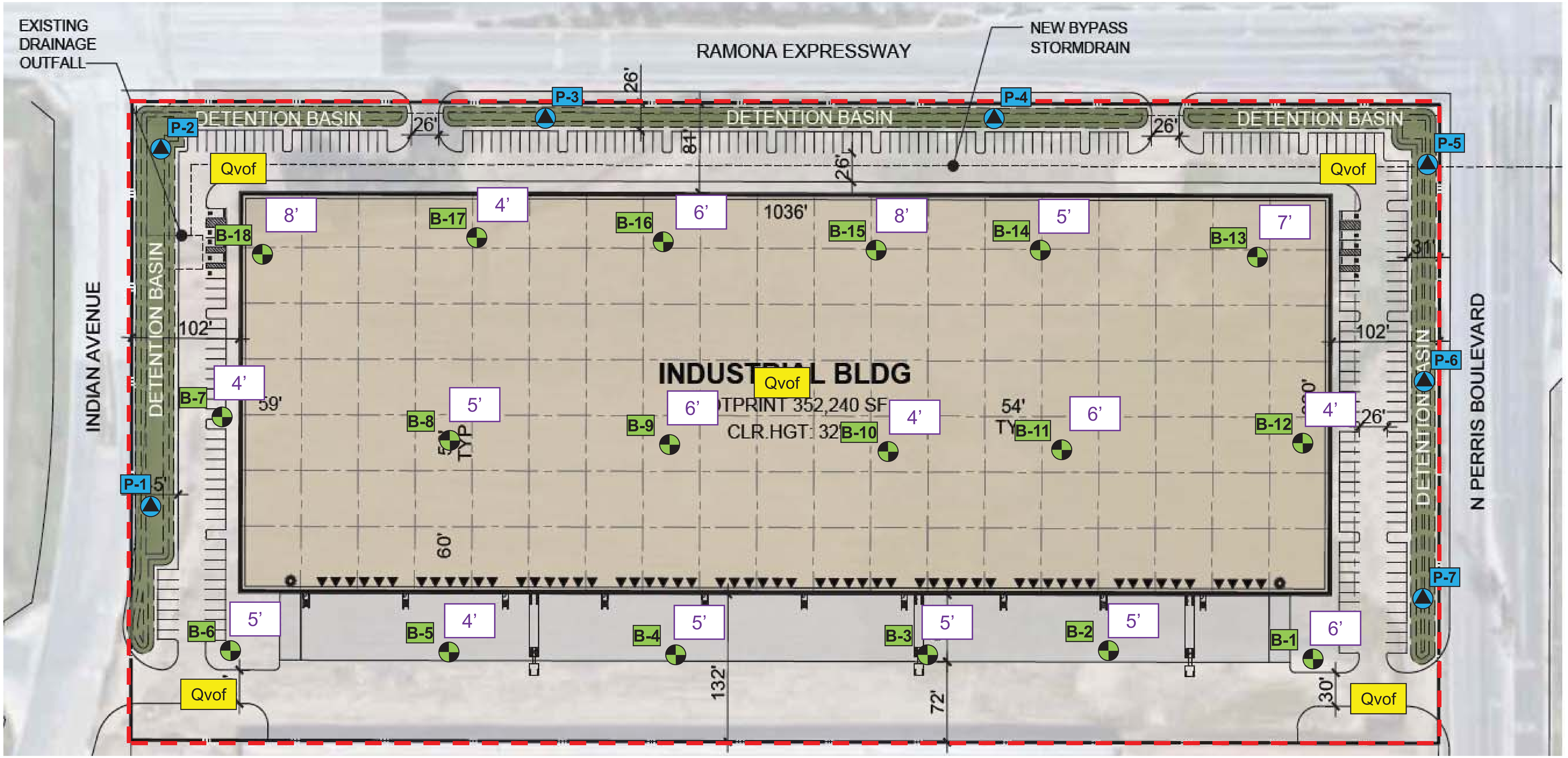
SOURCE: Google Earth, 2020

VICINITY MAP

GEOCON WEST, INC.
GEOTECHNICAL, ENVIRONMENTAL, MATERIALS
41571 CORNING PLACE #101, MURRIETA, CALIFORNIA 92562
PHONE 951-304-2300 FAX 951-304-2392

WAREHOUSE BUILDING
SWC OF RAMONA EXPRESSWAY
& PERRIS BOULEVARD
PERRIS, CALIFORNIA

LCW			APRIL 2020	PROJECT NO. T2400-22-02	FIG. 1



Source: Pacific Development Partners, LLC., *Conceptual Site Plan*, dated July, 2019.

0 100 200

SCALE 1" = 100'

GEOCON LEGEND

Locations are approximate

B-18 .... GEOTECHNICAL BORING LOCATION (2006)

P-7 .... PERCOLATION TEST BORING LOCATION

8' .... ANTICIPATED REMOVAL DEPTHS IN BUILDING STRUCTURE AREAS. SEE REPORT FOR REMOVAL DEPTHS IN ALL OTHER AREAS.

.... PROJECT LIMITS

Qvof .... VERY OLD ALLUVIAL FAN DEPOSITIS

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GEOLOGIC MAP

WAREHOUSE BUILDING
SWC OF RAMONA EXPRESSWAY
& PERRIS BOULEVARD
PERRIS, CALIFORNIA

LCW			APRIL 2020	PROJECT NO. T2400-22-02	FIG. 2

Project : WAREHOUSE BUILDING
File No. : T2400-22-02
Boring : B-4 (with other boring data incorporated)

TECHNICAL ENGINEERING AND DESIGN GUIDES AS ADAPTED FROM THE US ARMY CORPS OF ENGINEERS, NO. 9
EVALUATION OF EARTHQUAKE-INDUCED SETTLEMENTS IN DRY SANDY SOILS
MAXIMUM CONSIDERED EARTHQUAKE

MCE EARTHQUAKE INFORMATION:

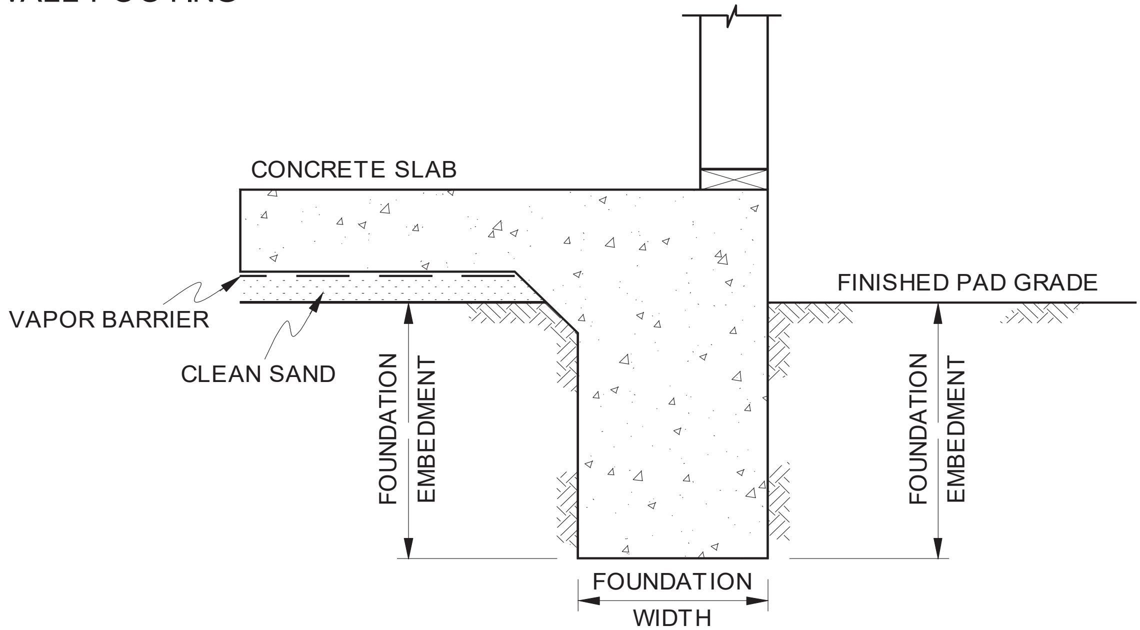
Earthquake Magnitude:	8.10
Peak Horiz. Acceleration (g):	0.550

Depth of Base of Strata (ft)	Thickness of Layer (ft)	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress [Tav]	Field SPT [N]	Correction Factor [Cer]	Relative Density [Dr] (%)	Correction Factor [Cn]	Corrected [N1]60	rd Factor	Maximum Shear Mod. [Gmax] (tsf)	[yeff]*[Geff] [Gmax]	yeff Shear Strain	[yeff]*100%	Volumetric Strain M7.5 [E15} (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Estimated Settlement [S] (inches)
1.0	1.0	0.5	132.6	0.03	0.02	0.012	11	1.3	77.0	2.0	31.7	1.0	210.762	5.57E-05	7.80E-05	0.008	4.49E-03	21.3669	5.27E-03	0.001
2.0	1.0	1.5	132.6	0.10	0.07	0.036	11	1.3	77.0	2.0	31.7	1.0	365.050	9.45E-05	1.90E-04	0.019	1.09E-02	21.3669	1.28E-02	0.003
3.0	1.0	2.5	132.6	0.17	0.11	0.059	11	1.3	77.0	2.0	31.7	1.0	471.277	1.20E-04	1.70E-04	0.017	9.79E-03	21.3669	1.15E-02	0.003
4.0	1.0	3.5	132.6	0.23	0.16	0.083	11	1.3	77.0	2.0	31.7	1.0	557.623	1.39E-04	1.70E-04	0.017	9.79E-03	21.3669	1.15E-02	0.003
5.0	1.0	4.5	135.8	0.30	0.20	0.107	17	1.3	90.7	1.9	42.6	1.0	698.966	1.40E-04	1.50E-04	0.015	6.05E-03	21.3669	7.10E-03	0.002
6.0	1.0	5.5	135.8	0.37	0.25	0.131	17	1.3	90.7	1.7	39.1	1.0	752.674	1.57E-04	1.50E-04	0.015	6.70E-03	21.3669	7.86E-03	0.002
7.0	1.0	6.5	135.8	0.43	0.29	0.155	17	1.3	90.7	1.5	36.5	1.0	800.669	1.71E-04	1.50E-04	0.015	7.28E-03	21.3669	8.54E-03	0.002
8.0	1.0	7.5	135.8	0.50	0.34	0.179	9	1.3	61.3	1.4	21.5	1.0	721.908	2.15E-04	4.50E-04	0.045	4.12E-02	21.3669	4.83E-02	0.012
9.0	1.0	8.5	135.8	0.57	0.38	0.203	9	1.3	61.3	1.4	20.6	1.0	758.335	2.29E-04	4.50E-04	0.045	4.33E-02	21.3669	5.08E-02	0.012
10.0	1.0	9.5	136.5	0.64	0.43	0.226	13	1.3	70.6	1.3	25.6	1.0	862.266	2.21E-04	4.50E-04	0.045	3.34E-02	21.3669	3.92E-02	0.009
11.0	1.0	10.5	136.5	0.71	0.47	0.250	13	1.3	70.6	1.2	24.7	1.0	896.147	2.31E-04	4.50E-04	0.045	3.49E-02	21.3669	4.09E-02	0.010
12.0	1.0	11.5	136.5	0.78	0.52	0.274	13	1.3	70.6	1.2	23.9	0.9	928.195	2.40E-04	3.70E-04	0.037	2.99E-02	21.3669	3.50E-02	0.008
13.0	1.0	12.5	136.5	0.84	0.57	0.297	13	1.3	70.6	1.1	23.2	0.9	958.665	2.49E-04	3.70E-04	0.037	3.09E-02	21.3669	3.63E-02	0.009
14.0	1.0	13.5	139.1	0.91	0.61	0.320	12	1.3	61.7	1.1	22.5	0.9	986.358	2.57E-04	3.70E-04	0.037	3.22E-02	21.3669	3.77E-02	0.009
15.0	1.0	14.5	139.1	0.98	0.66	0.344	12	1.3	61.7	1.0	21.9	0.9	1014.718	2.64E-04	3.70E-04	0.037	3.31E-02	21.3669	3.89E-02	0.009
16.0	1.0	15.5	139.1	1.05	0.70	0.367	12	1.3	61.7	1.0	21.4	0.9	1041.961	2.70E-04	3.70E-04	0.037	3.41E-02	21.3669	4.00E-02	0.010
17.0	1.0	16.5	139.1	1.12	0.75	0.390	12	1.3	61.7	1.0	21.0	0.9	1068.204	2.76E-04	3.70E-04	0.037	3.50E-02	21.3669	4.10E-02	0.010
18.0	1.0	17.5	139.1	1.19	0.80	0.413	12	1.3	61.7	0.9	20.5	0.9	1093.545	2.82E-04	3.70E-04	0.037	3.58E-02	21.3669	4.20E-02	0.010
19.0	1.0	18.5	139.1	1.26	0.84	0.436	12	1.3	61.7	0.9	20.2	0.9	1118.067	2.87E-04	3.70E-04	0.037	3.66E-02	21.3669	4.29E-02	0.010
20.0	1.0	19.5	136.5	1.33	0.89	0.458	21	1.3	75.3	0.9	31.9	0.9	1337.828	2.49E-04	3.70E-04	0.037	2.11E-02	21.3669	2.48E-02	0.006
21.0	1.0	20.5	136.5	1.40	0.94	0.479	21	1.3	75.3	0.9	31.3	0.9	1362.864	2.52E-04	3.70E-04	0.037	2.16E-02	21.3669	2.54E-02	0.006
22.0	1.0	21.5	136.5	1.47	0.98	0.501	21	1.3	75.3	0.8	30.7	0.9	1387.188	2.56E-04	3.70E-04	0.037	2.21E-02	21.3669	2.59E-02	0.006
23.0	1.0	22.5	136.5	1.53	1.03	0.522	21	1.3	75.3	0.8	30.2	0.9	1410.853	2.59E-04	3.00E-04	0.030	1.83E-02	21.3669	2.15E-02	0.005
24.0	1.0	23.5	136.5	1.60	1.07	0.543	21	1.3	75.3	0.8	29.7	0.9	1433.907	2.62E-04	3.00E-04	0.030	1.87E-02	21.3669	2.19E-02	0.005
25.0	1.0	24.5	136.5	1.67	1.12	0.563	26	1.3	78.2	0.8	36.4	0.9	1566.555	2.46E-04	3.00E-04	0.030	1.46E-02	21.3669	1.72E-02	0.004
26.0	1.0	25.5	136.5	1.74	1.16	0.584	26	1.3	78.2	0.8	35.8	0.9	1589.664	2.48E-04	3.00E-04	0.030	1.49E-02	21.3669	1.75E-02	0.004
27.0	1.0	26.5	136.5	1.81	1.21	0.604	26	1.3	78.2	0.8	35.2	0.9	1612.238	2.50E-04	3.00E-04	0.030	1.52E-02	21.3669	1.78E-02	0.004
28.0	1.0	27.5	136.5	1.88	1.26	0.623	26	1.3	78.2	0.7	34.7	0.9	1634.307	2.52E-04	3.00E-04	0.030	1.55E-02	21.3669	1.82E-02	0.004
29.0	1.0	28.5	136.5	1.94	1.30	0.643	26	1.3	78.2	0.7	34.2	0.9	1655.903	2.54E-04	3.00E-04	0.030	1.57E-02	21.3669	1.85E-02	0.004
30.0	1.0	29.5	136.5	2.01	1.35	0.662	29	1.3	77.8	0.7	38.2	0.9	1748.230	2.45E-04	3.00E-04	0.030	1.38E-02	21.3669	1.62E-02	0.004
31.0	1.0	30.5	136.5	2.08	1.39	0.680	29	1.3	77.8	0.7	37.7	0.9	1769.594	2.47E-04	3.00E-04	0.030	1.40E-02	21.3669	1.64E-02	0.004
32.0	1.0	31.5	136.5	2.15	1.44	0.699	29	1.3	77.8	0.7	37.2	0.9	1790.542	2.48E-04	3.00E-04	0.030	1.42E-02	21.3669	1.67E-02	0.004
33.0	1.0	32.5	136.5	2.22	1.48	0.717	29	1.3	77.8	0.7	36.8	0.9	1811.093	2.49E-04	3.00E-04	0.030	1.45E-02	21.3669	1.69E-02	0.004
34.0	1.0	33.5	136.5	2.28	1.53	0.734	29	1.3	77.8	0.7	36.3	0.8	1831.269	2.50E-04	3.00E-04	0.030	1.47E-02	21.3669	1.72E-02	0.004
35.0	1.0	34.5	139.1	2.35	1.58	0.752	18	1.3	58.1	0.7	24.9	0.8	1639.594	2.84E-04	3.00E-04	0.030	2.30E-02	21.3669	2.70E-02	0.006
36.0	1.0	35.5	139.1	2.42	1.62	0.769	18	1.3	58.1	0.7	24.7	0.8	1657.859	2.85E-04	3.00E-04	0.030	2.33E-02	21.3669	2.74E-02	0.007
37.0	1.0	36.5	139.1	2.49	1.67	0.786	18	1.3	58.1	0.6	24.4	0.8	1675.824	2.85E-04	3.00E-04	0.030	2.36E-02	21.3669	2.77E-02	0.007
38.0	1.0	37.5	139.1	2.56	1.72	0.803	18	1.3	58.1	0.6	24.2	0.8	1693.504	2.86E-04	3.00E-04	0.030	2.39E-02	21.3669	2.80E-02	0.007
39.0	1.0	38.5	136.5	2.63	1.76	0.819	23	1.3	62.5	0.6	28.7	0.8	1816.221	2.70E-04	3.00E-04	0.030	1.95E-02	21.3669	2.28E-02	0.005
40.0	1.0	39.5	136.5	2.70	1.81	0.835	23	1.3	62.5	0.6	28.4	0.8	1833.712	2.70E-04	3.00E-04	0.030	1.97E-02	21.3669	2.31E-02	0.006
41.0	1.0	40.5	139.1	2.77	1.85	0.850	19	1.3	54.3	0.6	24.4	0.8	1766.726	2.84E-04	3.00E-04	0.030	2.36E-02	21.3669	2.76E-02	0.007
42.0	1.0	41.5	139.1	2.84	1.90	0.865	19	1.3	54.3	0.6	24.2	0.8	1783.521	2.84E-04	3.00E-04	0.030	2.38E-02	21.3669	2.79E-02	0.007
43.0	1.0	42.5	139.1	2.91	1.95	0.880	19	1.3	54.3	0.6	24.0	0.8	1800.080	2.84E-04	3.00E-04	0.030	2.41E-02	21.3669	2.82E-02	0.007
44.0	1.0	43.5	139.1	2.98	1.99	0.895	19	1.3	54.3	0.6	23.8	0.8	1816.414	2.85E-04	3.00E-04	0.030	2.43E-02	21.3669	2.85E-02	0.007
45.0	1.0	44.5	139.1	3.05	2.04	0.909	19	1.3	54.3	0.6	23.6	0.8	1832.532	2.85E-04	1.00E-02	1.000	8.19E-01	21.3669	9.60E-01	0.230
46.0	1.0	45.5	136.5	3.12	2.09	0.923	28	1.3	63.3	0.6	31.2	0.8	2033.756	2.59E-04	1.00E-02	1.000	5.86E-01	21.3669	6.87E-01	0.165
47.0	1.0	46.5	136.5	3.18	2.13	0.936	28	1.3	63.3	0.6	31.0	0.8	2050.166	2.59E-04	1.00E-02	1.000	5.92E-01	21.3669	6.94E-01	0.167
48.0	1.0	47.5	136.5	3.25	2.18	0.949	28	1.3	63.3	0.6	30.7	0.8	2066.370	2.59E-04	1.00E-02	1.000	5.98E-01	21.3669	7.01E-01	0.168
49.0	1.0	48.5	136.5	3.32	2.22	0.961	28	1.3	63.3	0.6	30.5	0.8	2082.376	2.58E-04	1.00E-02	1.000	6.03E-01	21.3669	7.07E-01	0.170
50.0	1.0	49.5	136.5	3.39	2.27	0.973	28	1.3	63.3	0.6	30.2	0.8	2098.188	2.58E-04	1.00E-02	1.000	6.09E-01	21.3669	7.14E-01	0.171

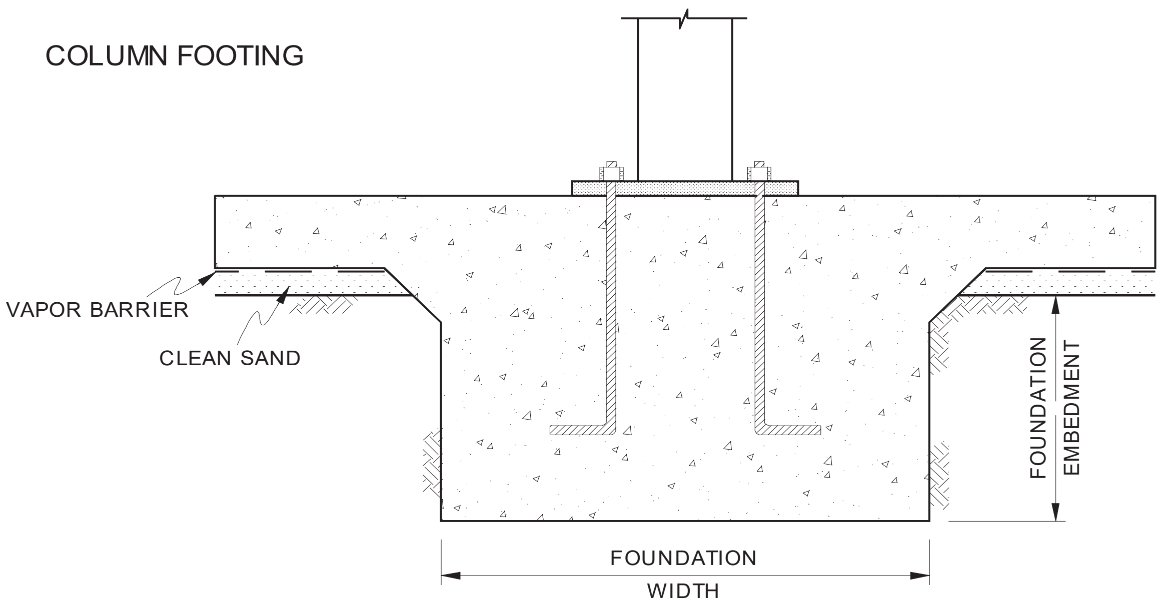
TOTAL SETTLEMENT = **1.34**

FIGURE 3

WALL FOOTING



COLUMN FOOTING



NOTE: SEE REPORT FOR FOUNDATION WIDTH AND DEPTH RECOMMENDATION

NO SCALE

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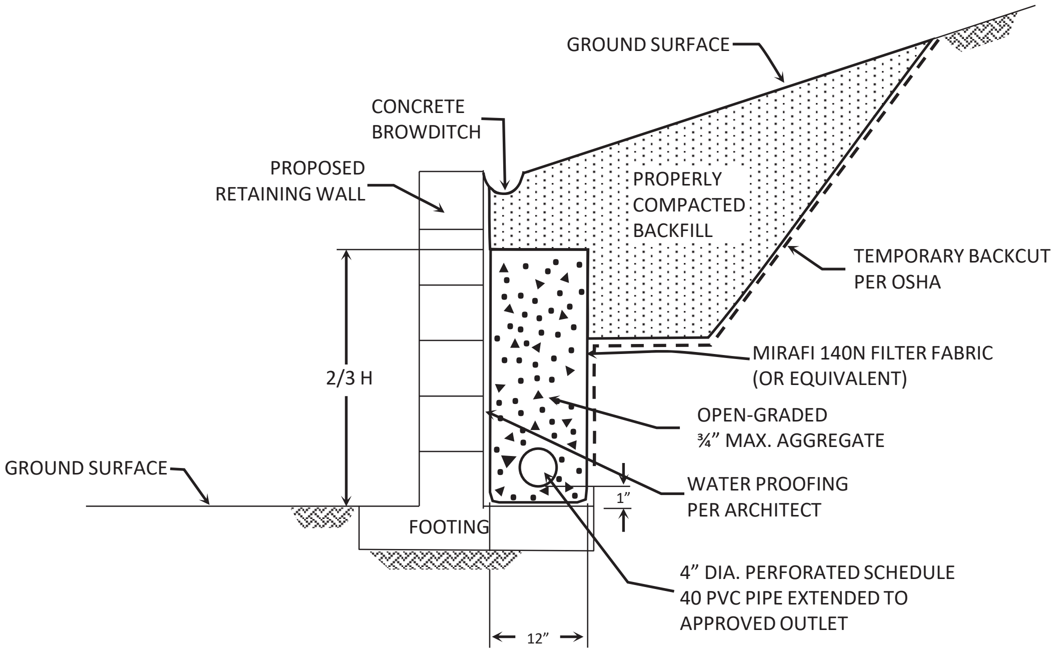
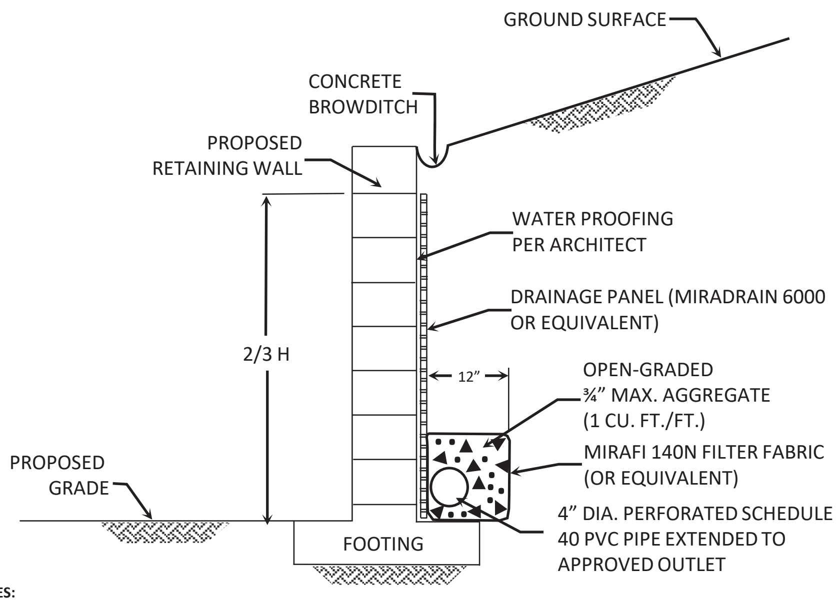
WALL / COLUMN FOOTING DETAIL

WARAEHOUSE BUILDING

SWC OF RAMONA EXPRESSWAY
& PERRIS BOULEVARD
PERRIS, CALIFORNIA

APRIL 2020	PROJECT NO. T2400-22-02	FIG. 4





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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WAREHOUSE BUILDING
SWC OF RAMONA EXPRESSWAY
& PERRIS BOULEVARD
PERRIS, CALIFORNIA

APRIL 2020	PROJECT NO. T2400-22-02	FIG. 5

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

Field work for our investigation included a site reconnaissance, subsurface explorations, soil sampling, and percolation testing. Our original subsurface exploration took place on August 4 and 7, 2006, where we drilled, logged, and sampled eighteen geotechnical borings to depths ranging between 16 and 51½ feet. On March 15 and 16, 2020 we drilled, logged, and sampled seven percolation test borings to depths of 5 and 11 feet in areas where storm water infiltration systems are proposed. All borings were drilled utilizing a truck mounted CME-75 hollow-stem auger drilling rig. The *Geologic Map*, Figure 2, presents the locations of our exploratory borings.

We collected bulk and relatively undisturbed samples from the borings by driving a 3-inch O. D. California Modified Sampler and a 2-inch O. D. Split-Spoon Sampler into the “undisturbed” soil mass with blows from a 140-pound hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2³/₈-inch inside diameter brass sampler rings to facilitate removal and testing. The samplers were driven 18 inches into the bottom of the excavations. Blow counts are recorded for every 6 inches the sampler is driven. The penetration resistances shown on the boring logs are shown in terms of blows per foot. The values indicated on the boring logs are the sum of the last 12 inches of the sampler if driven 18 inches. If the sampler was not driven for 18 inches, an approximate value is calculated in term of blows per foot or the final 6-inch interval is reported. These values are not to be taken as N-values, adjustments have not been applied. Relatively undisturbed samples and bulk samples of disturbed soils were transported to our laboratory for testing. We estimated elevations shown on the boring logs from either *Google Earth Pro* or other available topographic information.

We visually examined the soil conditions encountered within the borings, classified, and logged them in general conformance with the Unified Soil Classification System (USCS). Logs of the geotechnical and percolation test borings are presented on Figures A-1 through A-25. The logs depict the general soil and geologic conditions encountered and the depth at which we obtained the soil samples.

Percolation testing was performed on March 17, 2020 in accordance with *Riverside County Flood Control and Water Conservation District, LID BMP Manual, Appendix A*. The percolation tests were run in general accordance with Section 2.3 *Shallow Percolation Test* (for test holes 10 feet or less in depth) and *Deep Percolation Test* (for test holes greater than 10 feet in depth) methods. The percolation test data is presented on Figures A-26 and A-32.

PROJECT NO. T2400-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING B 1** ELEV. (MSL.) ~1455' DATE COMPLETED 08-04-2006 EQUIPMENT CME 75 BY: K. COX	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	B1-1			CL	**ALLUVIUM** Stiff, moist, brown, Sandy CLAY			
2	B1-2					18	123.6	12.5
4				SM	Medium dense, moist, brown, Silty, fine to medium SAND			
6	B1-3					17		
8	B1-4					19	120.6	13.2
10	B1-5					17		
12								
14								
16	B1-6					22		
					BORING TERMINATED AT 16 FEET No groundwater encountered			

Figure A-1,
Log of Boring B 1, Page 1 of 1

T2400-22-01.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.

GEOCON

PROJECT NO. T2400-22-01

BORING B 2

ELEV. (MSL.) ~1455' DATE COMPLETED 08-04-2006

EQUIPMENT CME 75 BY: K. COX

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	**ALLUVIUM** Dense, moist, brown, Silty, fine to medium SAND; some mica			
2	B2-1					52		
4								
	B2-2				-Becomes loose and fine grained at 5'	13	107.3	12.9
6								
				SP	Medium dense, moist, brown, fine to coarse SAND; trace silt			
8	B2-3					41	121.6	6.7
10	B2-4					29		
12								
				SM	Medium dense, moist, brown, Silty, fine SAND			
14								
	B2-5					23		
16					BORING TERMINATED AT 16 FEET No groundwater encountered			

T2400-22-01.GPJ

Figure A-2,
Log of Boring B 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.

GEOCON

PROJECT NO. T2400-22-01

BORING B 3

ELEV. (MSL.) ~1455' DATE COMPLETED 08-04-2006

EQUIPMENT CME 75 BY: K. COX

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**ALLUVIUM** Medium dense, moist, brown, Clayey, fine to medium SAND			
2	B3-1					35		
4								
	B3-2			SM	Medium dense, moist, brown, Silty, fine to medium SAND	33	129.0	10.3
6								
8	B3-3					22	119.5	10.6
10	B3-4			SP	Medium dense, moist, brown, fine to medium SAND; trace silt	24		
12								
14				SM	Medium dense, moist, brown, Silty, fine SAND; trace clay			
	B3-5					19		
16					BORING TERMINATED AT 16 FEET No groundwater encountered			

Figure A-3,
Log of Boring B 3, Page 1 of 1

T2400-22-01.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.

GEOCON

PROJECT NO. T2400-22-01

BORING B 4

ELEV. (MSL.) ~1455' DATE COMPLETED 08-04-2006

EQUIPMENT CME 75 BY: K. COX

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B4-1			SM	**ALLUVIUM** Medium dense, moist, brown, Silty, fine to medium SAND; trace clay			
2	B4-2					11		
4								
6	B4-3					27	120.7	12.5
8	B4-4				-Becomes fine grained at 7'	9		
10	B4-5				-Becomes fine to medium grained at 10'	21		
12								
14				ML	Stiff, moist, brown, Sandy SILT			
16	B4-6					12		
18								
20	B4-7			SM	Medium dense, moist, brown, Silty, fine to medium SAND	34		
22								
24								
26	B4-8					26		
28								

Figure A-4,
Log of Boring B 4, Page 1 of 2

T2400-22-01.GPJ

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

GEOCON

BORING B 4

ELEV. (MSL.) ~1455' DATE COMPLETED 08-04-2006

EQUIPMENT CME 75 BY: K. COX

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B4-9					46		
32								
34								
	B4-10			ML	Very stiff, moist, brown SILT; trace sand	18		
36								
38								
				SM	Medium dense, moist, brown, Silty, fine to medium SAND			
40	B4-11					37		
				ML	Very stiff, moist, brown SILT; trace sand			
42								
44								
	B4-12					19		
46				SM	Medium dense, moist, brown, Silty, very fine SAND			
48								
50	B4-13					44		
					BORING TERMINATED AT 51 FEET No groundwater encountered			

Figure A-4,
Log of Boring B 4, Page 2 of 2

T2400-22-01.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.

GEOCON

BORING B 5

ELEV. (MSL.) ~1455'   DATE COMPLETED 08-04-2006

EQUIPMENT CME 75   BY: K. COX

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**ALLUVIUM** Medium dense, moist, brown, Clayey, fine to medium SAND			
2	B5-1					22	128.0	10.9
4								
6	B5-2					29	121.8	11.9
8	B5-3			SM	Medium dense, moist, brown, Silty, fine to medium SAND	32		
10	B5-4					20		
12								
14								
16	B5-5				-Becomes fine grained at 15'	18		
					BORING TERMINATED AT 16 FEET No groundwater encountered			

T2400-22-01.GPJ

Figure A-5,
Log of Boring B 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.

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PROJECT NO. T2400-22-01

BORING B 6

ELEV. (MSL.) ~1455' DATE COMPLETED 08-04-2006

EQUIPMENT CME 75 BY: K. COX

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B6-1			SM	**ALLUVIUM** Medium dense, moist, brown, Silty, fine to medium SAND			
2	B6-2					18	124.2	9.9
4								
6	B6-3					30	127.3	11.9
8	B6-4				-Becomes fine grained at 7'	41	118.3	13.6
10	B6-5				-Becomes fine to medium grained at 10'	26		
12								
14								
16	B6-6				-Becomes fine grained at 15'	36		
					BORING TERMINATED AT 16 FEET No groundwater encountered			

Figure A-6,
Log of Boring B 6, Page 1 of 1

T2400-22-01.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.

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PROJECT NO. T2400-22-01

BORING B 7

ELEV. (MSL.) ~1455' DATE COMPLETED 08-04-2006

EQUIPMENT CME 75 BY: K. COX

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0				SC	**ALLUVIUM** Medium dense, moist, brown, Clayey, fine to medium SAND			
2	B7-1					24	123.2	12.5
4								
	B7-2			SM	Medium dense, moist, brown, Silty, fine to medium SAND	29	122.2	14.8
6								
8	B7-3			SP	Medium dense, moist, brown, fine to medium SAND; trace silt	30		
10	B7-4				-Becomes clean, fine to medium sand at 10'	14		
12								
14								
	B7-5			SC	Medium dense, moist, brown, Clayey, fine to medium SAND	33		
16					BORING TERMINATED AT 16 FEET No groundwater encountered			

Figure A-7,
Log of Boring B 7, Page 1 of 1

T2400-22-01.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.

GEOCON

BORING B 8

ELEV. (MSL.) ~1455' DATE COMPLETED 08-04-2006

EQUIPMENT CME 75 BY: K. COX

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B8-1			SM	**ALLUVIUM** Medium dense, moist, brown, Silty, fine to coarse SAND			
2	B8-2					14	123.4	9.5
4								
	B8-3					26	125.1	8.6
6								
8	B8-4				-Becomes fine to medium grained at 7½'	32		
10	B8-5			SP	Medium dense, moist, brown, fine to medium SAND; trace silt	33		
12								
14								
	B8-6			SM	Medium dense, moist, brown, Silty, fine to medium SAND	35		
16					BORING TERMINATED AT 16 FEET No groundwater encountered			

Figure A-8,
Log of Boring B 8, Page 1 of 1

T2400-22-01.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.

GEOCON

BORING B 9

ELEV. (MSL.) ~1455' DATE COMPLETED 08-04-2006

EQUIPMENT CME 75 BY: K. COX

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	**ALLUVIUM** Medium dense, moist, brown, Silty, fine to medium SAND			
2	B9-1					18	122.7	5.9
4	B9-2					18	124.2	11.1
6								
8	B9-3					32	120.8	11.3
10	B9-4					23		
12								
14	B9-5				-Becomes fine grained at 15'	34		
16					BORING TERMINATED AT 16 FEET No groundwater encountered			

Figure A-9,
Log of Boring B 9, Page 1 of 1

T2400-22-01.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.

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PROJECT NO. T2400-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING B 10** ELEV. (MSL.) **~1455'** DATE COMPLETED **08-04-2006** EQUIPMENT **CME 75** BY: **K. COX**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**ALLUVIUM** Loose, moist, brown, Silty, fine to medium SAND			
2	B10-1					13	120.9	6.6
4	B10-2				-Becomes medium dense at 5'	26	130.0	10.0
6								
8	B10-3					38	126.0	12.5
10	B10-4					22		
12								
14								
	B10-5				-Becomes fine grained at 15'	39		
16					BORING TERMINATED AT 16 FEET No groundwater encountered			

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Figure A-10,
Log of Boring B 10, 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.

GEOCON

BORING B 11

ELEV. (MSL.) ~1455' DATE COMPLETED 08-04-2006

EQUIPMENT CME 75 BY: K. COX

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B11-1			SC	**ALLUVIUM** Loose, moist, brown, Clayey, fine SAND			
2	B11-2					12	121.5	13.0
4								
	B11-3			SM	Loose, moist, brown, Silty, fine to medium SAND	14	121.9	11.4
6								
8	B11-4					13		
				SP	Medium dense, moist, brown, clean SAND			
10	B11-5					20		
				ML	Stiff, moist, brown, Sandy SILT			
12								
14								
	B11-6			SM	Medium dense, moist, brown, Silty, fine to medium SAND	30		
16					BORING TERMINATED AT 16 FEET No groundwater encountered			

T2400-22-01.GPJ

Figure A-11,
Log of Boring B 11, 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. T2400-22-01

BORING B 12

ELEV. (MSL.) ~1455' DATE COMPLETED 08-04-2006

EQUIPMENT CME 75 BY: K. COX

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	**ALLUVIUM** Medium dense, moist, brown, Silty, fine to medium SAND			
2								
	B12-1					41	128.8	11.8
4								
	B12-2					31	123.8	13.4
6								
8	B12-3					22		
				SP	Medium dense, moist, brown, fine to coarse SAND; trace silt			
10	B12-4					18		
				SM	Medium dense, moist, brown, Silty, fine to medium SAND			
12								
14								
	B12-5				-Becomes fine grained at 15'	29		
16					BORING TERMINATED AT 16 FEET No groundwater encountered			

Figure A-12,
Log of Boring B 12, Page 1 of 1

T2400-22-01.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.

GEOCON

PROJECT NO. T2400-22-01

BORING B 13

ELEV. (MSL.) ~1455' DATE COMPLETED 08-07-2006

EQUIPMENT CME 75 BY: K. COX

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B13-1			SM	**ALLUVIUM** Medium dense, moist, brown, Silty, fine to medium SAND			
2	B13-2					14	123.5	7.4
4								
	B13-3					2		
6				SC	Loose, moist, olive-brown, Clayey, fine to medium SAND			
8	B13-4					24	117.1	17.2
				SM	Medium dense, moist, brown, Silty, fine to medium SAND			
10	B13-5					10		
12								
14								
	B13-6					38		
16				ML	Very stiff, moist, brown SILT; trace sand			
18								
20	B13-7					21		
22								
24								
	B13-8			SM	Very dense, moist, brown, Silty, fine to coarse SAND, some gravel	84/11"		
26								
28								

T2400-22-01.GPJ

Figure A-13,
Log of Boring B 13, Page 1 of 2

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.

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BORING B 13

ELEV. (MSL.) ~1455' DATE COMPLETED 08-07-2006

EQUIPMENT CME 75 BY: K. COX

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B13-9			ML	Stiff, moist, light brown SILT; trace sand and gravel	10		
32								
34								
	B13-10				-Becomes brown	20		
36								
38								
40	B13-11			CL	Stiff, moist, brown CLAY; trace sand	15		
42								
44								
	B13-12			ML	Very stiff, moist, brown SILT; trace sand	34		
46								
48								
50	B13-13			SM	Medium dense, moist, brown, Silty, fine to medium SAND	28		
					BORING TERMINATED AT 51½ FEET No groundwater encountered			

Figure A-13,
Log of Boring B 13, Page 2 of 2

T2400-22-01.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.

GEOCON

PROJECT NO. T2400-22-01

BORING B 14

ELEV. (MSL.) ~1455' DATE COMPLETED 08-07-2006

EQUIPMENT CME 75 BY: K. COX

DEPTH IN FEET	SAMPLE NO.	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0		SM	**ALLUVIUM** Medium dense, moist, brown, Silty, fine to medium SAND			
3	B14-1			43		
5.5	B14-2			39	128.0	7.4
8	B14-3			39		
10.5	B14-4			37		
15.5	B14-5			27		
		ML	Very stiff, moist, brown, SILT; trace sand			
16			BORING TERMINATED AT 16 FEET No groundwater encountered			

Figure A-14,
Log of Boring B 14, Page 1 of 1

T2400-22-01.GPJ

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.

GEOCON

PROJECT NO. T2400-22-01

BORING B 15

ELEV. (MSL.) ~1455' DATE COMPLETED 08-07-2006

EQUIPMENT CME 75 BY: K. COX

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B15-1			SM	**ALLUVIUM** Medium dense, moist, brown, Silty, fine to medium SAND; voids			
2	B15-2					37		
4								
6	B15-3					30	116.9	4.7
8	B15-4				-Becomes dense with no voids at 7'	47		
10	B15-5			ML	Very stiff, moist, brown SILT	38		
12								
14								
16	B15-6					18		
					BORING TERMINATED AT 16 FEET No groundwater encountered			

T2400-22-01.GPJ

Figure A-15,
Log of Boring B 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.

GEOCON

BORING B 16

ELEV. (MSL.) ~1455' DATE COMPLETED 08-07-2006

EQUIPMENT CME 75 BY: K. COX

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	**ALLUVIUM** Medium dense, moist, brown, Silty, fine to medium SAND			
2								
	B16-1					37		
4								
	B16-2					31	128.7	6.8
6								
8	B16-3					40	120.6	6.4
10	B16-4					26		
12								
14				ML	Stiff, moist, brown SILT; trace sand			
	B16-5					15		
16					BORING TERMINATED AT 16 FEET No groundwater encountered			

T2400-22-01.GPJ

Figure A-16,
Log of Boring B 16, 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.

GEOCON

PROJECT NO. T2400-22-01

BORING B 17

ELEV. (MSL.) ~1455' DATE COMPLETED 08-07-2006

EQUIPMENT CME 75 BY: K. COX

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	**ALLUVIUM** Medium dense, moist, brown, Silty, fine to medium SAND			
2								
	B17-1					28	129.5	10.2
4								
	B17-2					27	129.1	10.1
6								
8	B17-3					35	123.6	9.1
10	B17-4			SP	Medium dense, moist, brown, fine to coarse SAND; trace silt	22		
12								
14								
	B17-5			SM	Medium dense, moist, brown, Silty, fine to medium SAND	29		
16					BORING TERMINATED AT 16 FEET No groundwater encountered			

Figure A-17,
Log of Boring B 17, Page 1 of 1

T2400-22-01.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.

GEOCON

BORING B 18

ELEV. (MSL.) ~1455' DATE COMPLETED 08-07-2006

EQUIPMENT CME 75 BY: K. COX

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B18-1			SM	**ALLUVIUM** Medium dense, moist, brown, Silty, fine to medium SAND			
2	B18-2					22		
4								
6	B18-3				-Becomes fine grained at 5'	39	124.6	12.4
8	B18-4				-Becomes loose at 7½'	8		
10	B18-5					11		
12								
14								
16	B18-6					7		
18								
20	B18-7				-Becomes dense at 20'	45		
22								
24								
26	B18-8			ML	Very stiff, moist, brown, Sandy SILT	21		
28								

Figure A-18,
Log of Boring B 18, Page 1 of 2

T2400-22-01.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.

GEOCON

BORING B 18

ELEV. (MSL.) ~1455' DATE COMPLETED 08-07-2006

EQUIPMENT CME 75 BY: K. COX

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
30	B18-9					21		
32								
34								
36	B18-10			SM	Medium dense, moist, brown, Silty, fine SAND	20		
38								
40	B18-11			ML	Very stiff, moist, brown, Sandy SILT	54		
42								
44								
46	B18-12					16		
48								
50	B18-13			SM	Dense, moist, brown, Silty, fine to medium SAND	65		

BORING TERMINATED AT 51 FEET

No groundwater encountered

Figure A-18,
Log of Boring B 18, Page 2 of 2

T2400-22-01.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.

GEOCON

PROJECT NO. T2400-22-02

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P-1 ELEV. (MSL.) 1460 DATE COMPLETED 03/16/2020 EQUIPMENT CME 75 BY: Weidman	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**VERY OLD ALLUVIUM (Qvof)** Silty SAND, medium dense, moist, dark brown; fine to coarse sand; suficial grass			
2								
4	P1@4.5-5'			ML	Sandy SILT, stiff, moist, dark brown; fine to medium sand			
					Total Depth = 5' Groundwater not encountered Backfilled with cuttings 03/17/2020			

Figure A-19,
Log of Boring P-1, Page 1 of 1

T2400-22-02 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

BORING P-2

ELEV. (MSL.) 1459 DATE COMPLETED 03/16/2020

EQUIPMENT CME 75 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				SM	**VERY OLD ALLUVIUM (Qvof)** Silty SAND, medium dense, moist, dark brown; fine to coarse sand; suficial grass			
2					-Becomes brown; increase in fine and medium sand			
4	P2@4.5-5'				-Increase in coarse sand			
					Total Depth = 5' Groundwater not encountered Backfilled with cuttings 03/17/2020			

Figure A-20,
Log of Boring P-2, Page 1 of 1

T2400-22-02 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. T2400-22-02

BORING P-3

ELEV. (MSL.) **1458** DATE COMPLETED **03/16/2020**

EQUIPMENT **CME 75** 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				SM	**VERY OLD ALLUVIUM (Qvof)** Silty SAND, medium dense, moist, dark brown; fine to coarse sand; suficial gravel			
2								
4								
6								
8					-Increase in coarse sand			
10	P3@10.5-11			ML	Sandy SILT, stiff, moist, reddish brown; fine to medium sand; gravel lens			
					Total Depth = 11' Groundwater not encountered Backfilled with cuttings 03/17/2020			

Figure A-21,
Log of Boring P-3, Page 1 of 1

T2400-22-02 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. T2400-22-02

BORING P-4

ELEV. (MSL.) **1458** DATE COMPLETED **03/16/2020**

EQUIPMENT **CME 75** 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				SM	**VERY OLD ALLUVIUM (Qvof)** Silty SAND, medium dense, moist, dark brown; fine to coarse sand; suficial grass			
2								
4					-Becomes reddish brown; dense			
	P4@4.5-5'							
					Total Depth = 5' Groundwater not encountered Backfilled with cuttings 03/17/2020			

Figure A-22,
Log of Boring P-4, Page 1 of 1

T2400-22-02 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. T2400-22-02

BORING P-5

ELEV. (MSL.) **1458** DATE COMPLETED **03/16/2020**

EQUIPMENT **CME 75** 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				SM	**VERY OLD ALLUVIUM (Qvof)** Silty SAND, medium dense, moist, dark brown; fine to coarse sand; suficial grass			
2								
4	P5@4.5-5'			ML	Sandy SILT, stiff, moist, dark reddish brown; fine to medium sand			

Total Depth = 5'
Groundwater not encountered
Backfilled with cuttings 03/17/2020

Figure A-23,
Log of Boring P-5, Page 1 of 1

T2400-22-02 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. T2400-22-02

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P-6 ELEV. (MSL.) 1458 DATE COMPLETED 03/16/2020 EQUIPMENT CME 75 BY: Weidman	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**VERY OLD ALLUVIUM (Qvof)** Silty SAND, medium dense, moist, dark brown; fine to coarse sand; suficial gravel			
2					-Becomes damp			
4								
6					-Becomes brown; increase in coarse sand			
8					-Becomes very dense			
10	P6@10.5-1				-Becomes dark brown; dense; moist			
					Total Depth = 11' Groundwater not encountered Backfilled with cuttings 03/17/2020			

Figure A-24,
Log of Boring P-6, Page 1 of 1

T2400-22-02 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. T2400-22-02

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P-7 ELEV. (MSL.) 1458 DATE COMPLETED 03/16/2020 EQUIPMENT CME 75 BY: Weidman	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**VERY OLD ALLUVIUM (Qvof)** Silty SAND, medium dense, moist, dark brown; fine to coarse sand; suficial grass			
2					-Increase in coarse sand			
4	P7@4.5-5'							
					Total Depth = 5' Groundwater not encountered Backfilled with cuttings 03/17/2020			

Figure A-25,
Log of Boring P-7, Page 1 of 1

T2400-22-02 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:	PDP Perris Perc UGI	**Project No.:**	T2400-22-02
Test Hole No.:	P-1	**Date Excavated:**	3/16/2020
Length of Test Pipe:	60.0 inches	**Soil Classification:**	ML
Height of Pipe above Ground:	0.0 inches	**Presoak Date:**	3/16/2020
Depth of Test Hole:	60.0 inches	**Perc Test Date:**	3/17/2020
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	8:35 AM 9:00 AM	25	25	32.6	32.4	0.2	104.2
2	9:00 AM 9:25 AM	25	50	32.4	32.3	0.1	208.3

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	9:25 AM 9:55 AM	30	30	32.3	32.3	0.0	No Rate
2	9:55 AM 10:25 AM	30	60	32.3	32.3	0.0	No Rate
3	10:25 AM 10:55 AM	30	90	32.3	32.3	0.0	No Rate
4	10:55 AM 11:25 AM	30	120	32.3	32.3	0.0	No Rate
5	11:25 AM 11:55 AM	30	150	32.3	32.3	0.0	No Rate
6	11:55 AM 12:25 PM	30	180	30.8	30.8	0.0	No Rate
7	12:25 PM 12:55 PM	30	210	30.8	30.8	0.0	No Rate
8	12:55 PM 1:25 PM	30	240	30.8	30.8	0.0	No Rate
9	1:25 PM 1:55 PM	30	270	30.8	30.7	0.1	250.0
10	1:55 PM 2:25 PM	30	300	30.7	30.7	0.0	No Rate
11	2:25 PM 2:55 PM	30	330	30.7	30.7	0.0	No Rate
12	2:55 PM 3:25 PM	30	360	30.7	30.7	0.0	No Rate

Infiltration Rate (in/hr):	0.0
Radius of test hole (in):	4
Average Head (in):	30.7

Figure A-26

PERCOLATION TEST REPORT

Project Name:	PDP Perris Perc UGI	**Project No.:**	T2400-22-02
Test Hole No.:	P-2	**Date Excavated:**	3/16/2020
Length of Test Pipe:	60.0 inches	**Soil Classification:**	SM
Height of Pipe above Ground:	0.0 inches	**Presoak Date:**	3/16/2020
Depth of Test Hole:	60.0 inches	**Perc Test Date:**	3/17/2020
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	8:36 AM 9:01 AM	25	25	31.3	30.6	0.7	34.7
2	9:01 AM 9:26 AM	25	50	30.6	29.9	0.7	34.7

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	9:26 AM 9:56 AM	30	30	29.9	29.3	0.6	50.0
2	9:56 AM 10:26 AM	30	60	29.3	28.8	0.5	62.5
3	10:26 AM 10:56 AM	30	90	28.8	27.7	1.1	27.8
4	10:56 AM 11:26 AM	30	120	27.7	27.0	0.7	41.7
5	11:26 AM 11:56 AM	30	150	28.0	26.3	1.7	17.9
6	11:56 AM 12:26 PM	30	180	26.3	25.9	0.4	83.3
7	12:26 PM 12:56 PM	30	210	25.9	25.8	0.1	250.0
8	12:56 PM 1:26 PM	30	240	25.8	25.2	0.6	50.0
9	1:26 PM 1:56 PM	30	270	25.2	25.1	0.1	250.0
10	1:56 PM 2:26 PM	30	300	25.1	24.8	0.2	125.0
11	2:26 PM 2:56 PM	30	330	24.8	24.6	0.2	125.0
12	2:56 PM 3:26 PM	30	360	24.6	24.4	0.2	125.0

Infiltration Rate (in/hr):	0.04
Radius of test hole (in):	4
Average Head (in):	24.5

Figure A-27

PERCOLATION TEST REPORT

Project Name:		PDP Perris Perc UGI			**Project No.:**		T2400-22-02
Test Hole No.:		P-3			**Date Excavated:**		3/16/2020
Length of Test Pipe:			132.0	inches	**Soil Classification:**		ML
Height of Pipe above Ground:			0.0	inches	**Presoak Date:**		3/16/2020
Depth of Test Hole:			132.0	inches	**Perc Test Date:**		3/17/2020
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	8:37 AM 9:02 AM	25	25	104.4	96.5	7.9	3.2
2	9:02 AM 9:27 AM	25	50	96.5	93.4	3.1	8.0

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	9:27 AM 9:57 AM	30	30	93.4	90.5	2.9	10.4
2	9:57 AM 10:27 AM	30	60	90.5	88.9	1.6	19.2
3	10:27 AM 10:57 AM	30	90	88.9	87.5	1.4	20.8
4	10:57 AM 11:27 AM	30	120	87.5	85.2	2.3	13.2
5	11:27 AM 11:57 AM	30	150	85.2	84.0	1.2	25.0
6	11:57 AM 12:27 PM	30	180	84.0	83.2	0.8	35.7
7	12:27 PM 12:57 PM	30	210	83.2	82.6	0.6	50.0
8	12:57 PM 1:27 PM	30	240	88.8	87.1	1.7	17.9
9	1:27 PM 1:57 PM	30	270	87.1	86.2	1.0	31.2
10	1:57 PM 2:27 PM	30	300	86.2	84.7	1.4	20.8
11	2:27 PM 2:57 PM	30	330	84.7	83.6	1.1	27.8
12	2:57 PM 3:27 PM	30	360	83.6	82.6	1.1	27.8

Infiltration Rate (in/hr):	0.1		
Radius of test hole (in):	4		**Figure A-28**
Average Head (in):	83.1		

PERCOLATION TEST REPORT

Project Name:		PDP Perris Perc UGI			**Project No.:**		T2400-22-02
Test Hole No.:		P-4			**Date Excavated:**		3/16/2020
Length of Test Pipe:			60.0	inches	**Soil Classification:**		SM
Height of Pipe above Ground:			0.0	inches	**Presoak Date:**		3/16/2020
Depth of Test Hole:			60.0	inches	**Perc Test Date:**		3/17/2020
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	8:38 AM 9:03 AM	25	25	29.3	29.0	0.2	104.2
2	9:03 AM 9:28 AM	25	50	29.0	28.2	0.8	29.8

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	9:28 AM 9:58 AM	30	30	28.2	27.7	0.5	62.5
2	9:58 AM 10:28 AM	30	60	27.7	27.5	0.2	125.0
3	10:28 AM 10:58 AM	30	90	27.5	26.3	1.2	25.0
4	10:58 AM 11:28 AM	30	120	26.3	25.1	1.2	25.0
5	11:28 AM 11:58 AM	30	150	25.1	24.2	0.8	35.7
6	11:58 AM 12:28 PM	30	180	24.2	23.3	1.0	31.3
7	12:28 PM 12:58 PM	30	210	24.7	24.4	0.4	83.3
8	12:58 PM 1:28 PM	30	240	24.4	24.1	0.2	125.0
9	1:28 PM 1:58 PM	30	270	24.1	23.8	0.4	83.3
10	1:58 PM 2:28 PM	30	300	23.8	23.4	0.4	83.3
11	2:28 PM 2:58 PM	30	330	23.4	22.9	0.5	62.5
12	2:58 PM 3:28 PM	30	360	22.9	22.7	0.2	125.0

Infiltration Rate (in/hr):	0.04	
Radius of test hole (in):	4	**Figure A-29**
Average Head (in):	22.8	

PERCOLATION TEST REPORT							
Project Name:		PDP Perris Perc UGI			**Project No.:**		T2400-22-02
Test Hole No.:		P-5			**Date Excavated:**		3/16/2020
Length of Test Pipe:			60.0	inches	**Soil Classification:**		ML
Height of Pipe above Ground:			0.0	inches	**Presoak Date:**		3/16/2020
Depth of Test Hole:			60.0	inches	**Perc Test Date:**		3/17/2020
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	8:39 AM 9:04 AM	25	25	33.5	32.4	1.1	23.1
2	9:04 AM 9:29 AM	25	50	32.4	31.4	1.0	26.0
		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	9:29 AM 9:59 AM	30	30	31.4	30.5	1.0	31.3
2	9:59 AM 10:29 AM	30	60	30.5	29.6	0.8	35.7
3	10:29 AM 10:59 AM	30	90	29.6	28.9	0.7	41.7
4	10:59 AM 11:29 AM	30	120	28.9	28.2	0.7	41.7
5	11:29 AM 11:59 AM	30	150	26.8	26.3	0.5	62.5
6	11:59 AM 12:29 PM	30	180	26.3	25.9	0.4	83.3
7	12:29 PM 12:59 PM	30	210	25.9	25.4	0.5	62.5
8	12:59 PM 1:29 PM	30	240	25.4	25.1	0.4	83.3
9	1:29 PM 1:59 PM	30	270	25.1	24.8	0.2	125.0
10	1:59 PM 2:29 PM	30	300	24.8	24.6	0.2	125.0
11	2:29 PM 2:59 PM	30	330	24.6	24.0	0.6	50.0
12	2:59 PM 3:29 PM	30	360	24.0	23.8	0.2	125.0
Infiltration Rate (in/hr):			0.04				
Radius of test hole (in):			4				**Figure A-30**
Average Head (in):			23.9				

PERCOLATION TEST REPORT

Project Name:	PDP Perris Perc UGI	**Project No.:**	T2400-22-02
Test Hole No.:	P-6	**Date Excavated:**	3/16/2020
Length of Test Pipe:	132.0 inches	**Soil Classification:**	SM
Height of Pipe above Ground:	0.0 inches	**Presoak Date:**	3/16/2020
Depth of Test Hole:	132.0 inches	**Perc Test Date:**	3/17/2020
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	8:40 AM 9:05 AM	25	25	93.6	86.0	7.6	3.3
2	9:05 AM 9:30 AM	25	50	86.0	82.7	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	9:30 AM 10:00 AM	30	30	82.7	80.3	2.4	12.5
2	10:00 AM 10:30 AM	30	60	92.4	88.1	4.3	6.9
3	10:30 AM 11:00 AM	30	90	88.1	84.4	3.7	8.1
4	11:00 AM 11:30 AM	30	120	99.5	95.6	3.8	7.8
5	11:30 AM 12:00 PM	30	150	91.2	87.6	3.6	8.3
6	12:00 PM 12:30 PM	30	180	87.6	85.1	2.5	11.9
7	12:30 PM 1:00 PM	30	210	85.1	82.3	2.8	10.9
8	1:00 PM 1:30 PM	30	240	92.2	89.5	2.6	11.4
9	1:30 PM 2:00 PM	30	270	89.5	87.4	2.2	13.9
10	2:00 PM 2:30 PM	30	300	87.4	84.8	2.5	11.9
11	2:30 PM 3:00 PM	30	330	84.8	82.7	2.2	13.9
12	3:00 PM 3:30 PM	30	360	82.7	80.4	2.3	13.2

Infiltration Rate (in/hr):	0.11
Radius of test hole (in):	4
Average Head (in):	81.5

Figure A-31

PERCOLATION TEST REPORT

Project Name:	PDP Perris Perc UGI	**Project No.:**	T2400-22-02
Test Hole No.:	P-7	**Date Excavated:**	3/16/2020
Length of Test Pipe:	60.0 inches	**Soil Classification:**	SM
Height of Pipe above Ground:	0.0 inches	**Presoak Date:**	3/16/2020
Depth of Test Hole:	60.0 inches	**Perc Test Date:**	3/17/2020
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	8:41 AM 9:06 AM	25	25	28.3	27.6	0.7	34.7
2	9:06 AM 9:31 AM	25	50	27.6	26.5	1.1	23.1

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	9:31 AM 10:01 AM	30	30	26.5	26.2	0.4	83.3
2	10:01 AM 10:31 AM	30	60	26.2	25.4	0.7	41.7
3	10:31 AM 11:01 AM	30	90	25.4	25.0	0.5	62.5
4	11:01 AM 11:31 AM	30	120	25.0	24.5	0.5	62.5
5	11:31 AM 12:01 PM	30	150	24.0	22.8	1.2	25.0
6	12:01 PM 12:31 PM	30	180	22.8	21.6	1.2	25.0
7	12:31 PM 1:01 PM	30	210	21.6	18.8	2.8	10.9
8	1:01 PM 1:31 PM	30	240	29.9	28.8	1.1	27.8
9	1:31 PM 2:01 PM	30	270	28.8	28.1	0.7	41.7
10	2:01 PM 2:31 PM	30	300	28.1	27.2	0.8	35.7
11	2:31 PM 3:01 PM	30	330	27.2	26.4	0.8	35.7
12	3:01 PM 3:31 PM	30	360	26.4	25.6	0.8	35.7

Infiltration Rate (in/hr):	0.12
Radius of test hole (in):	4
Average Head (in):	26.0

Figure A-32

APPENDIX B

APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with current, generally accepted test methods of ASTM International (ASTM) or other suggested procedures. For our laboratory testing program of our 2006 geotechnical investigation, we analyzed selected soil samples for in-situ dry density and moisture content, maximum dry density and optimum moisture content, direct shear strength, collapse/swell potential, consolidation characteristics, expansion index/potential, and corrosion screening. For our current laboratory testing program, we determined the grain size distribution of the soil encountered at the bottom of our percolation test borings. The results of our laboratory testing are presented on Figures B-1 through B-11.

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 undisturbed samples were tested to evaluate their in-place dry density and moisture content, shear strength, collapse potential, and consolidation characteristics. Disturbed bulk samples were tested to obtain maximum dry density and optimum moisture content, expansion characteristics, soluble sulfate content, potential of hydrogen, resistivity, and chloride content. Results of the laboratory tests are presented in tabular and graphic form herewith.

TABLE B-I
SUMMARY OF LABORATORY MAXIMUM DRY DENSITY AND OPTIMUM MOISTURE CONTENT TEST RESULTS
ASTM D 1557-02

Sample No.	Description	Maximum Dry Density (pcf)	Optimum Moisture Content (% dry wt.)
B1-1	SM - Dark brown, Silty, fine to coarse SAND	133.5	7.5
B11-1	SM - Dark brown, Silty, fine to coarse SAND	136.4	8.1
B18-1	SM - Gray brown, Silty, fine to medium SAND , with little clay	131.9	8.4

TABLE B-II
SUMMARY OF LABORATORY EXPANSION INDEX TEST RESULTS
ASTM D4829-03

Sample No.	Moisture Content		Dry Density (pcf)	Expansion Index
	Before Test (%)	After Test (%)		
B1-1*	8.7	18.1	116.6	18
B8-1*	7.5	14.7	121.7	3

* Expansion index was corrected in accordance with §10.2.3 of ASTM D4829.

TABLE B-III
SUMMARY OF DIRECT SHEAR TEST RESULTS

Sample No.	Dry Density (pcf)	Moisture Content (%)	Unit Cohesion (psf)	Angle of Shear Resistance (degrees)
B11-1	122.8	8.0	180	31
B18-1	117.0	10.0	210	26

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*
B4-4	0.014%	Negligible
B15-1	0.002%	Negligible

* Per UBC Table 19-A-4.

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
B1-4	120.6	13.2	2,000	1.7	0.1
B2-2	107.3	12.9	2,000	2.1	0.8
B3-3	119.5	10.6	2,000	1.9	0.4
B5-1	128.0	10.9	2,000	1.6	0.0
B6-2	124.2	9.9	2,000	1.5	0.3
B6-3	127.3	11.9	2,000	2.8	0.7
B7-2	122.2	14.8	2,000	1.8	0.3
B8-2	123.4	9.5	2,000	1.6	0.6
B9-2	124.2	11.1	2,000	1.6	0.2
B9-3	120.8	11.3	2,000	1.4	0.3
B10-2	130.0	10.0	2,000	1.9	0.2
B11-2	121.5	13.0	2,000	2.1	0.4
B11-3	121.9	11.4	2,000	1.5	0.2
B12-1	128.8	11.8	2,000	2.0	0.7
B13-4	117.1	17.2	2,000	1.7	0.2
B15-3	116.9	4.7	2,000	1.5	3.4
B16-2	128.7	6.8	2,000	1.5	1.6
B17-2	129.1	10.1	2,000	1.9	0.4

Negative sign indicates soil expansion

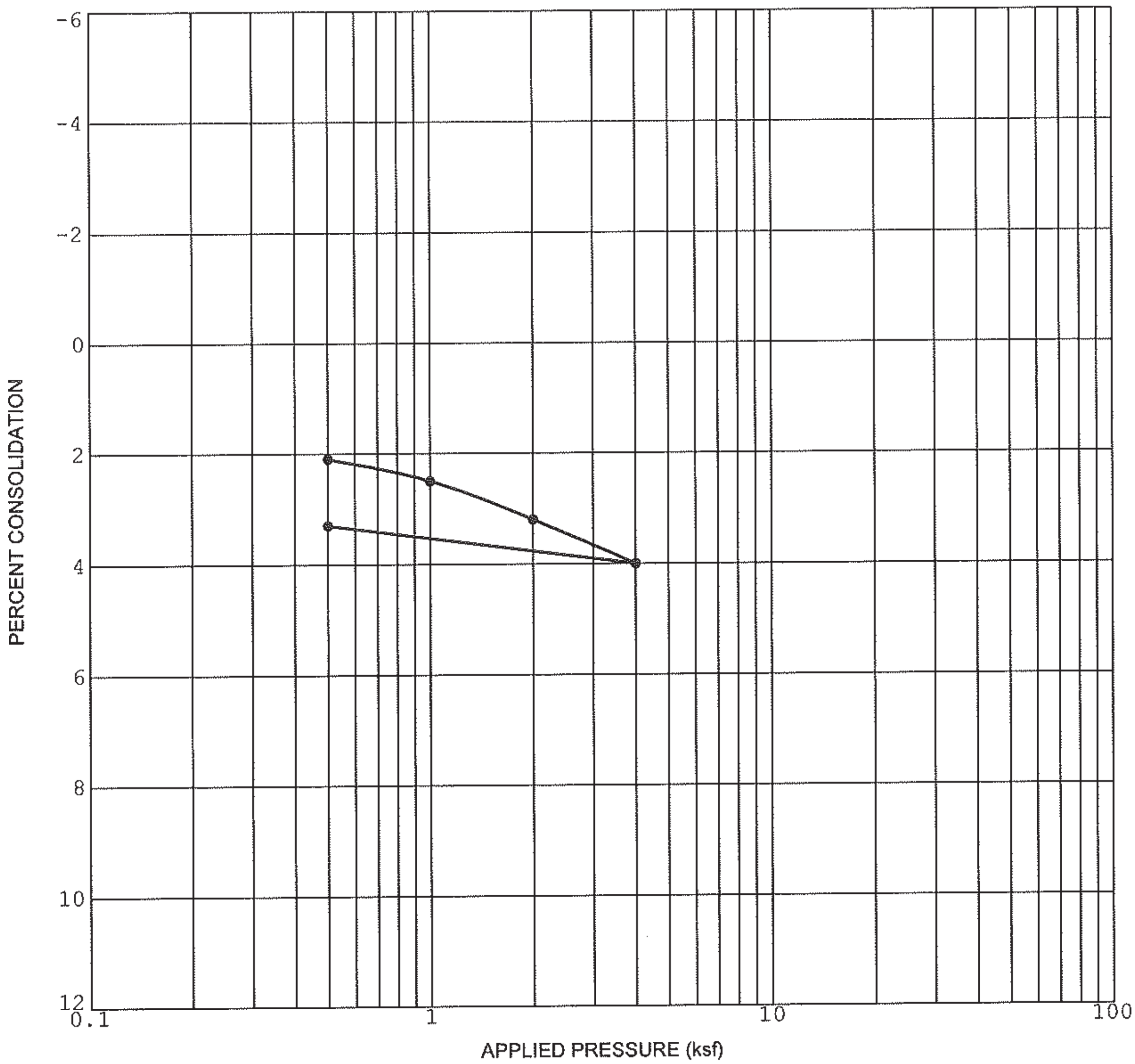
TABLE B-VI
SUMMARY OF PH, RESISTIVITY AND CHLORIDE TESTS

Sample No.	pH	Chloride (ppm)	Resistivity (Ohm-cm)
B4-1	7.4	340	811
B15-1	6.5	21	5408

Resistivity and pH tests were performed in accordance with Cal Trans Test 532.

PROJECT NO. T2400-22-01

SAMPLE NO. B4-3



Initial Dry Density (pcf)	120.7
Initial Water Content (%)	12.5

Initial Saturation (%)	88.7
Sample Saturated at (ksf)	2.0

CONSOLIDATION CURVE

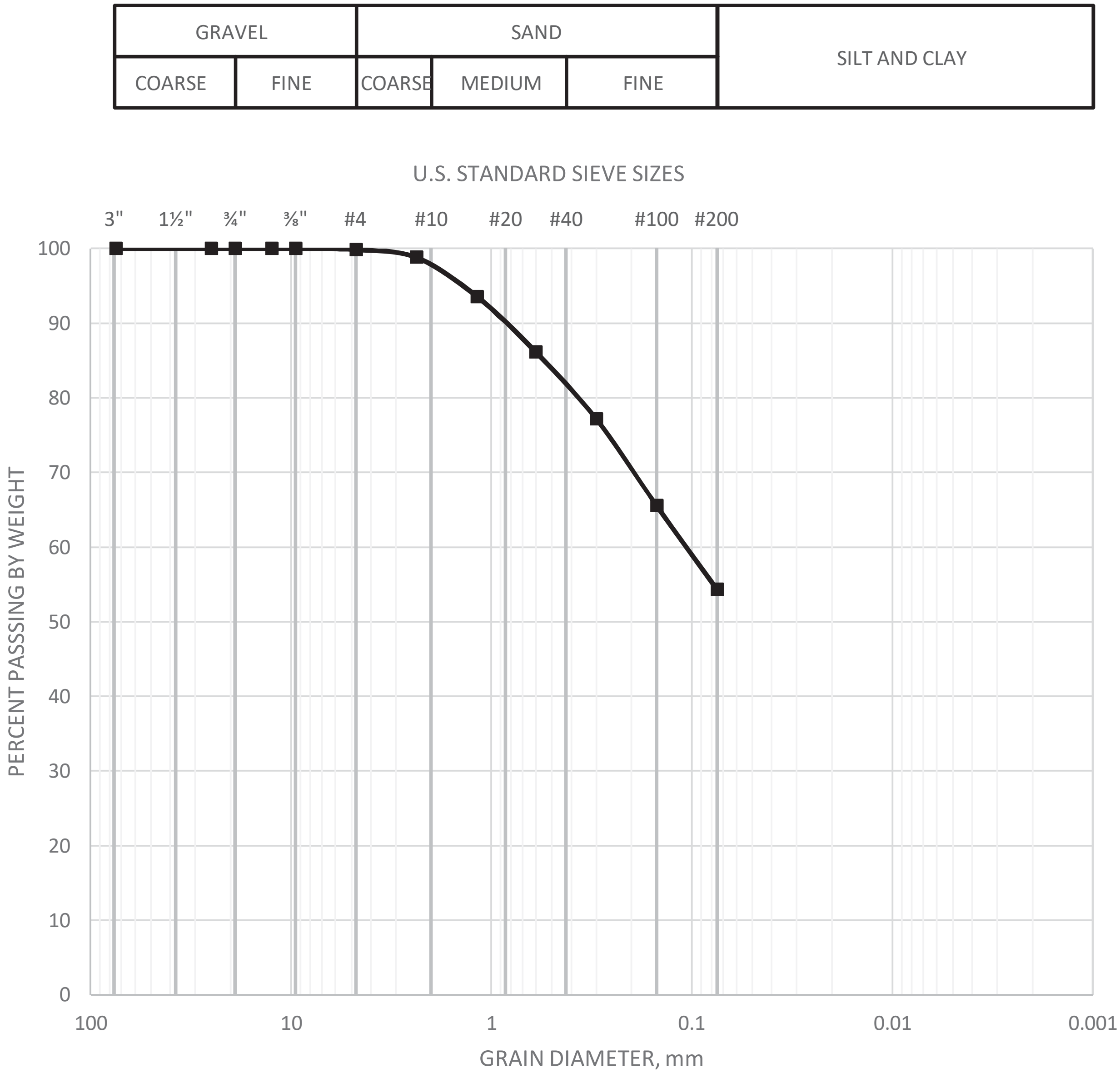
RAMONA CROSSING

PARIS, CALIFORNIA

T2400-22-01.GPJ

GEOCON

Figure B-4



SAMPLE	CLASSIFICATION	D60	D30	D10
P1@4.5-5'	Sandy SILT (ML), dark brown			

GEOCON

GRAIN SIZE DISTRIBUTION

ASTM D-422

Checked by: MJ

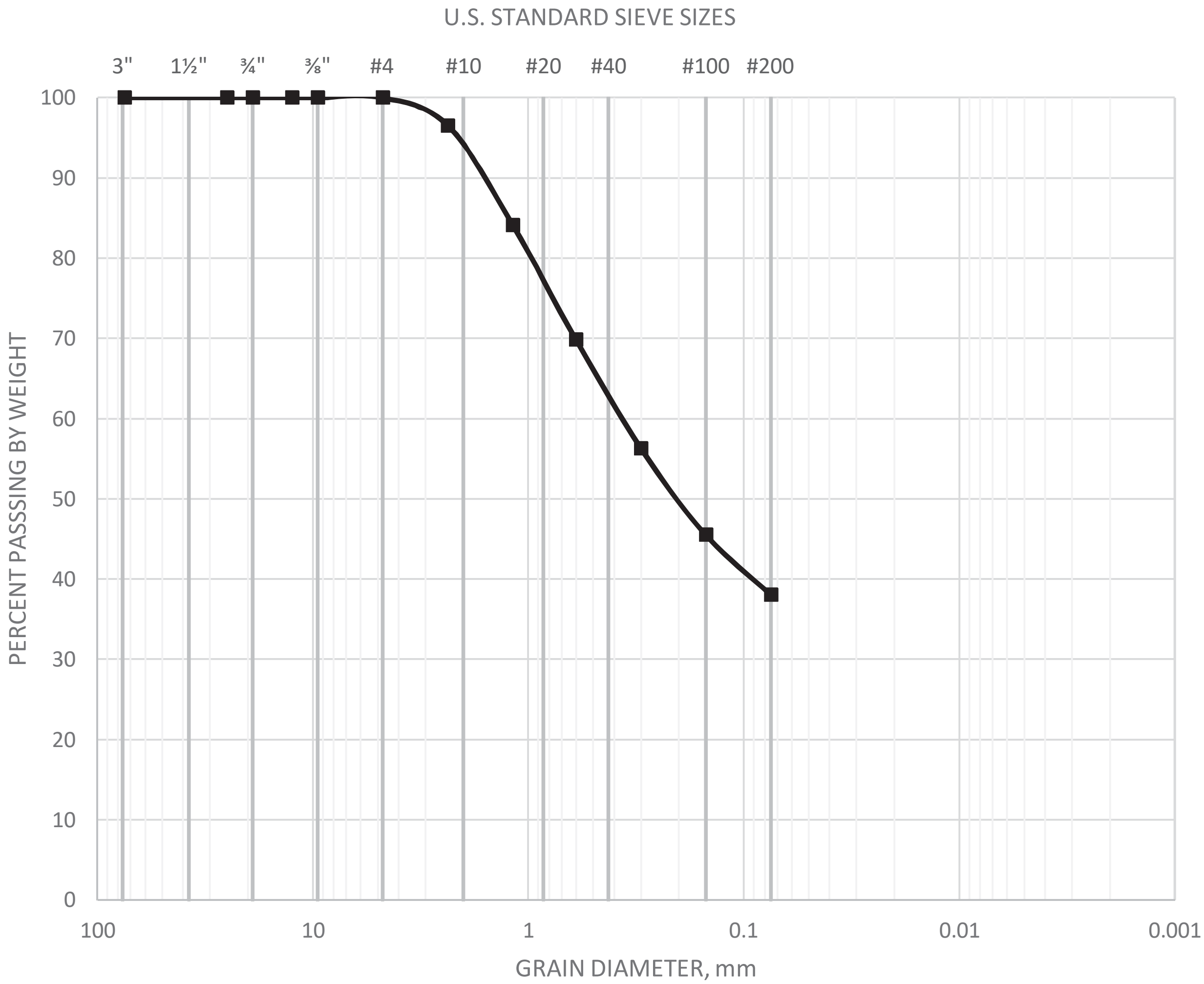
Project No.: T2400-22-02

WAREHOUSE BUILDING
SWC RAMONA EXPRESSWAY & PERRIS BOULEVARD
PERRIS, CALIFORNIA

April 2020

Figure B-5

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SAMPLE	CLASSIFICATION	D60	D30	D10
P2@4.5-5'	Silty SAND (SM), brown	0.38	0.04	0.005

GEOCON

GRAIN SIZE DISTRIBUTION

ASTM D-422

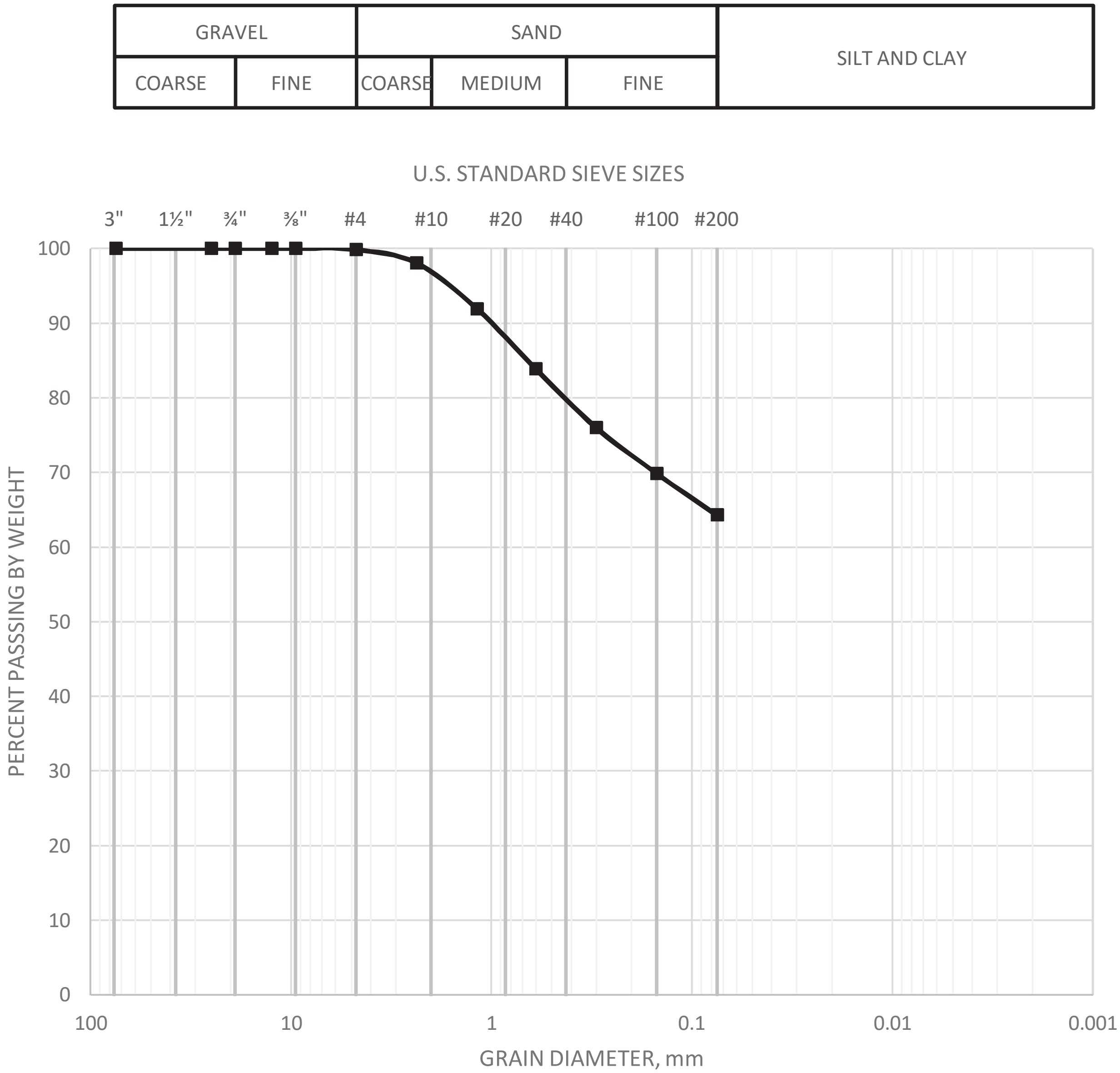
Checked by: MJ

Project No.: T2400-22-02

WAREHOUSE BUILDING
SWC RAMONA EXPRESSWAY & PERRIS BOULEVARD
PERRIS, CALIFORNIA

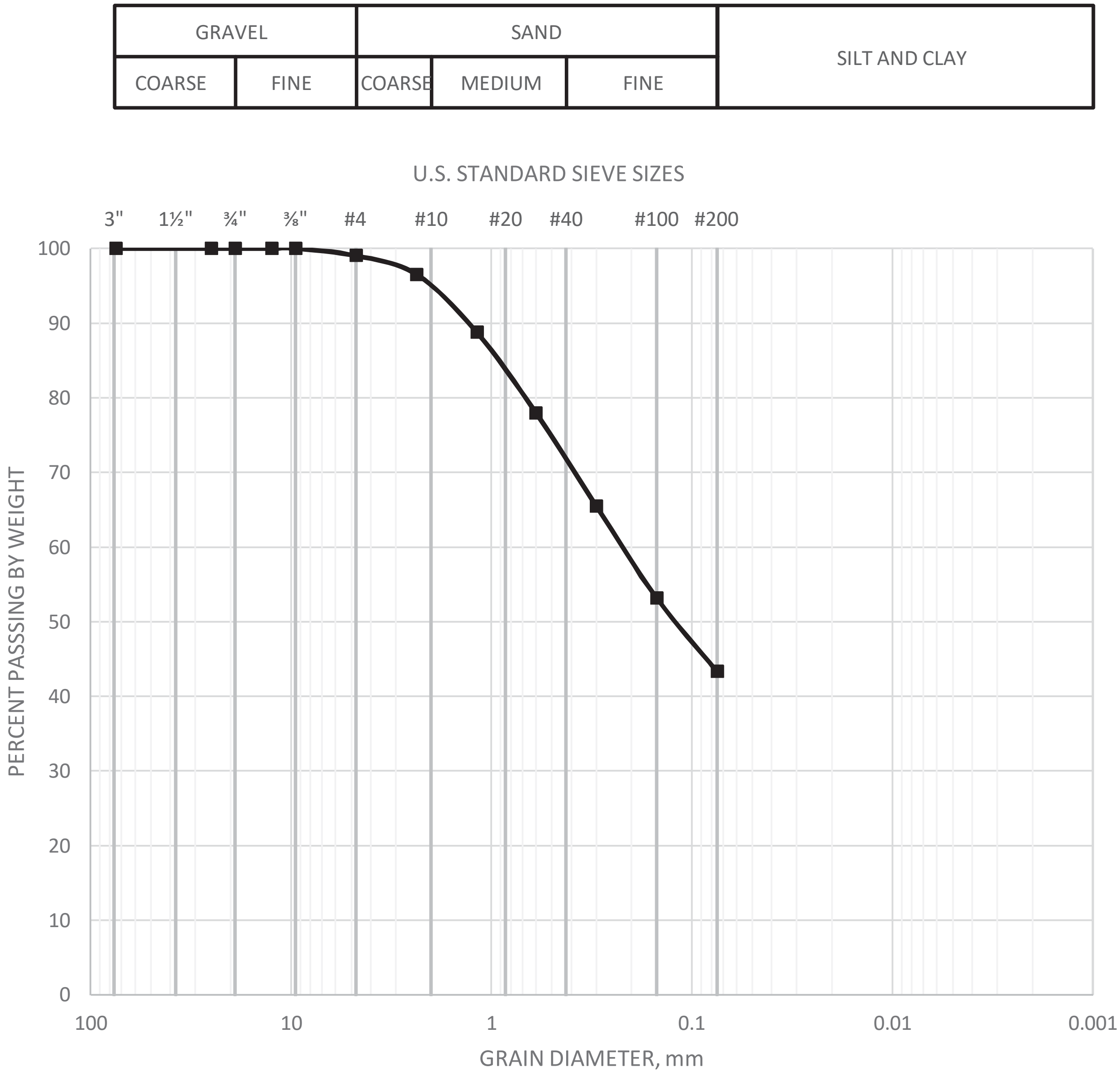
April 2020

Figure B-6



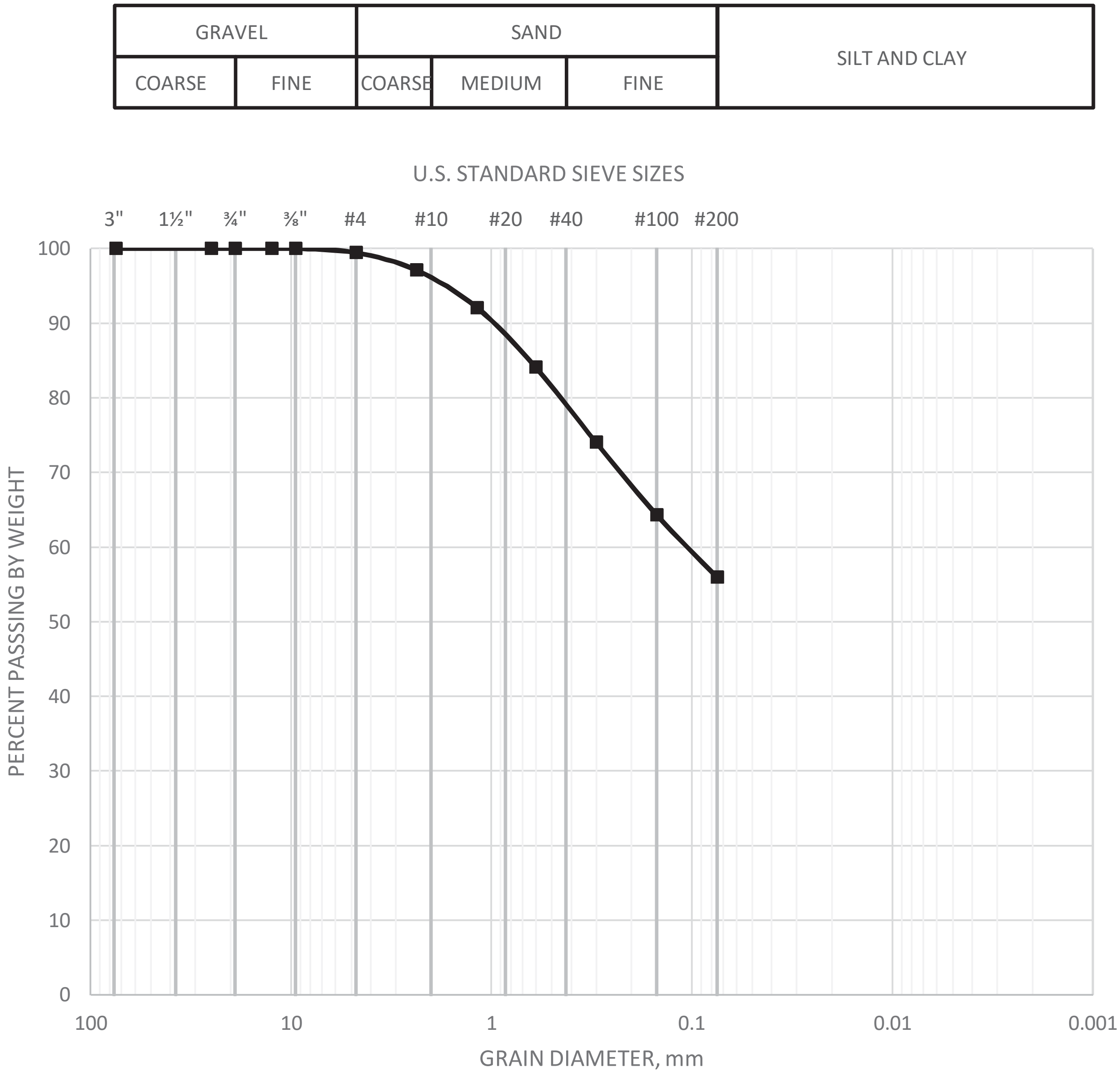
SAMPLE	CLASSIFICATION	D60	D30	D10
P3@10.5-11'	Sandy SILT (ML), reddish brown			

GEOCON	**GRAIN SIZE DISTRIBUTION**	Project No.: T2400-22-02
	ASTM D-422	WAREHOUSE BUILDING SWC RAMONA EXPRESSWAY & PERRIS BOULEVARD PERRIS, CALIFORNIA
	Checked by: MJ	April 2020 Figure B-7



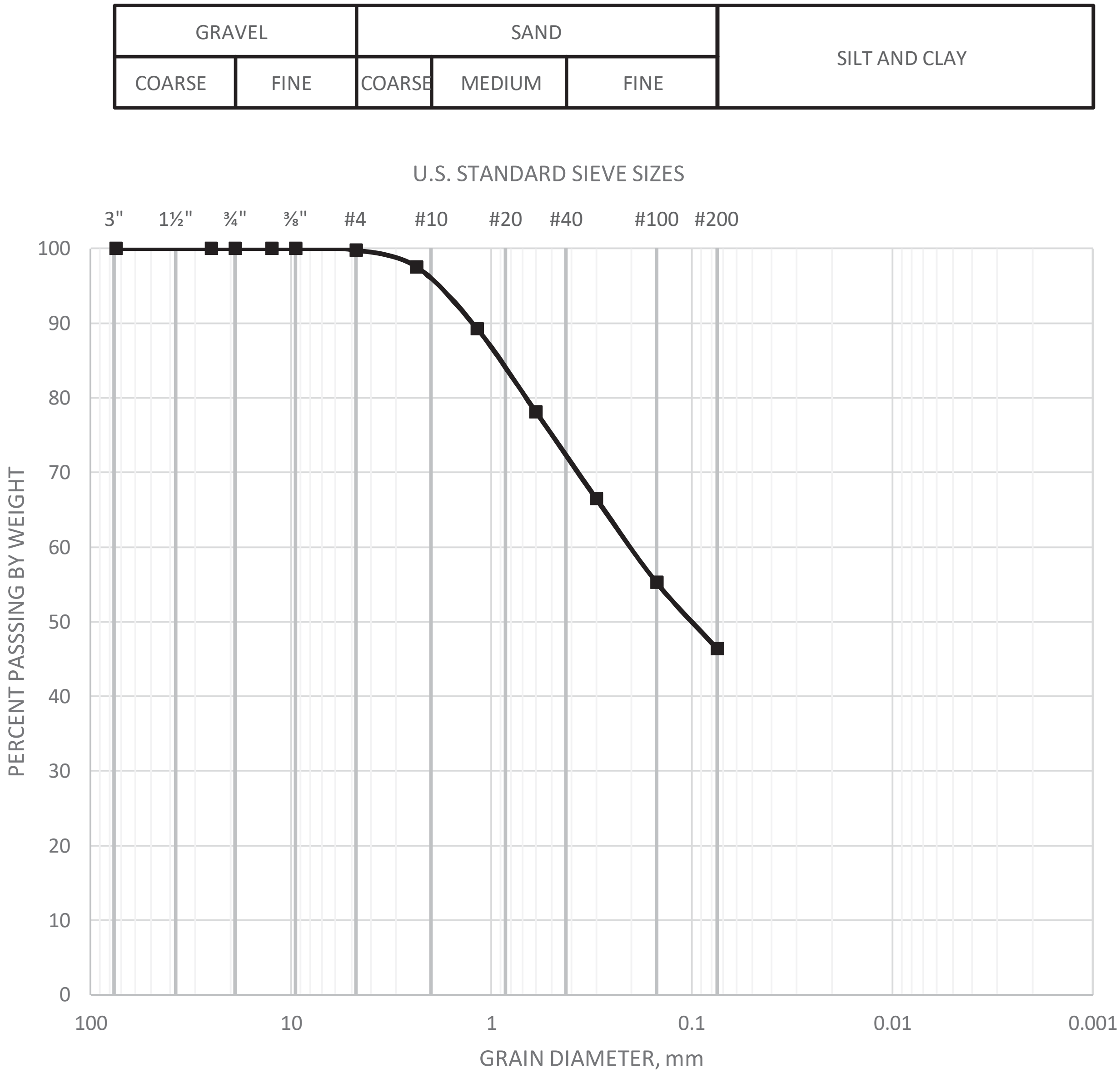
SAMPLE	CLASSIFICATION	D60	D30	D10
P4@4.5-5'	Silty SAND (SM), reddish brown	0.22	0.027	0.009

GEOCON	**GRAIN SIZE DISTRIBUTION** ASTM D-422	Project No.: T2400-22-02
		WAREHOUSE BUILDING SWC RAMONA EXPRESSWAY & PERRIS BOULEVARD PERRIS, CALIFORNIA
	Checked by: MJ	April 2020 — Figure B-8



SAMPLE	CLASSIFICATION	D60	D30	D10
P5@4.5-5'	Sandy SILT (ML), dark reddish brown			

GEOCON	**GRAIN SIZE DISTRIBUTION** ASTM D-422 Checked by: MJ	Project No.: T2400-22-02 WAREHOUSE BUILDING SWC RAMONA EXPRESSWAY & PERRIS BOULEVARD PERRIS, CALIFORNIA April 2020 Figure B-9



SAMPLE	CLASSIFICATION	D60	D30	D10
P6@10.5-11'	Silty SAND (SM), dark brown	0.21	0.02	0.004

GEOCON

GRAIN SIZE DISTRIBUTION

ASTM D-422

Checked by: MJ

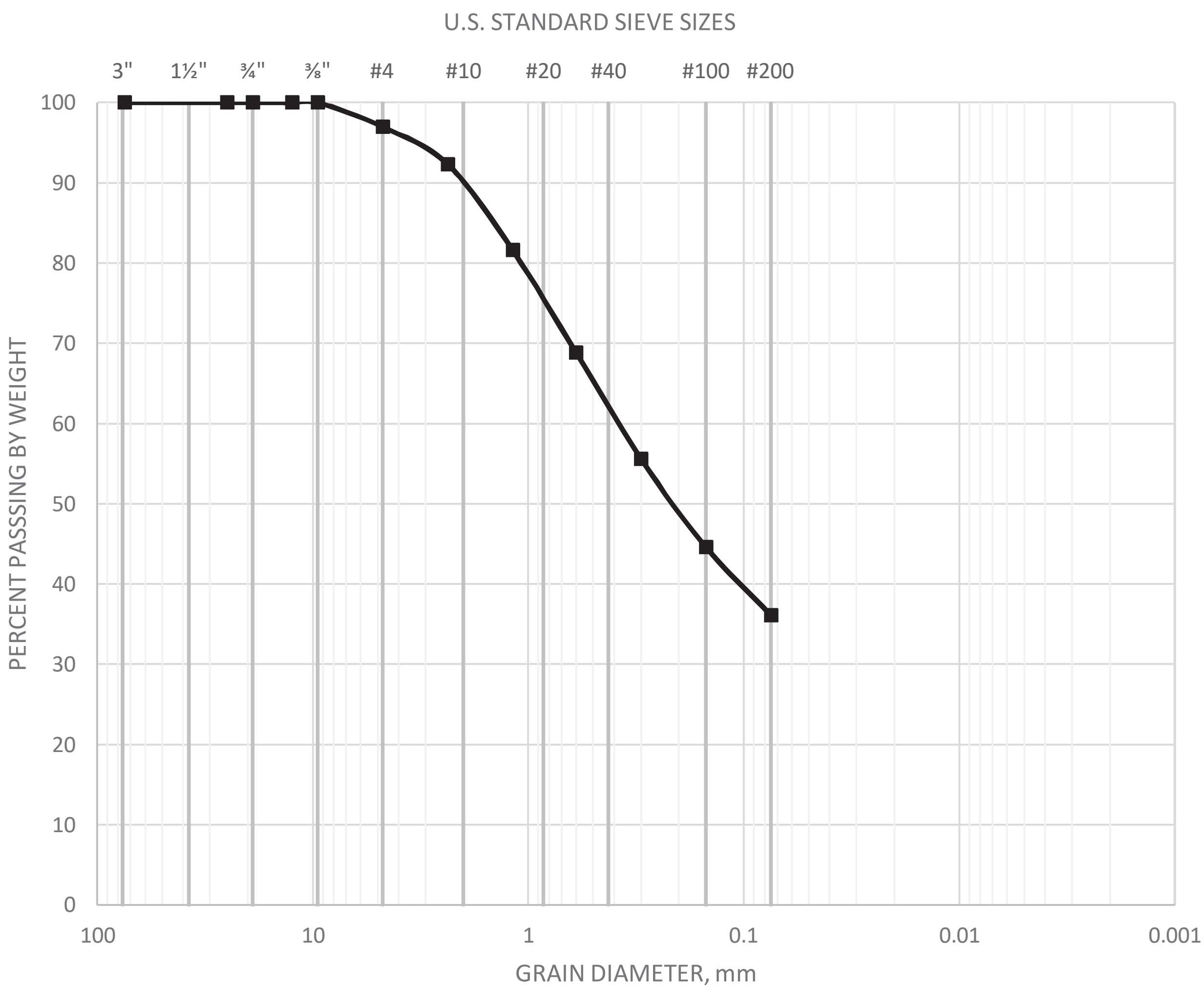
Project No.: T2400-22-02

WAREHOUSE BUILDING
SWC RAMONA EXPRESSWAY & PERRIS BOULEVARD
PERRIS, CALIFORNIA

April 2020

Figure B-10

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SAMPLE	CLASSIFICATION	D60	D30	D10
P7@4.5-5'	Silty SAND with trace gravel (SM), dark brown	0.4	0.044	0.008

GEOCON

GRAIN SIZE DISTRIBUTION

ASTM D-422

Checked by: MJ

Project No.: T2400-22-02

WAREHOUSE BUILDING
SWC RAMONA EXPRESSWAY & PERRIS BOULEVARD
PERRIS, CALIFORNIA

April 2020

Figure B-11

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

WAREHOUSE BUILDING
SOUTHWEST CORNER OF RAMONA EXPRESSWAY & PERRIS BOULEVARD
PERRIS, CALIFORNIA

PROJECT NO. T2400-22-02

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

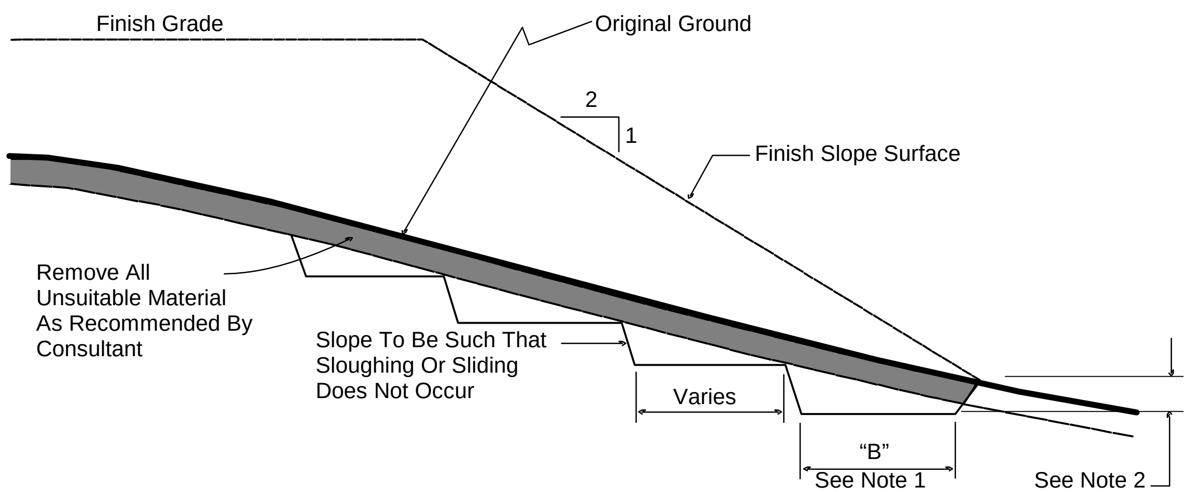
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material 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.

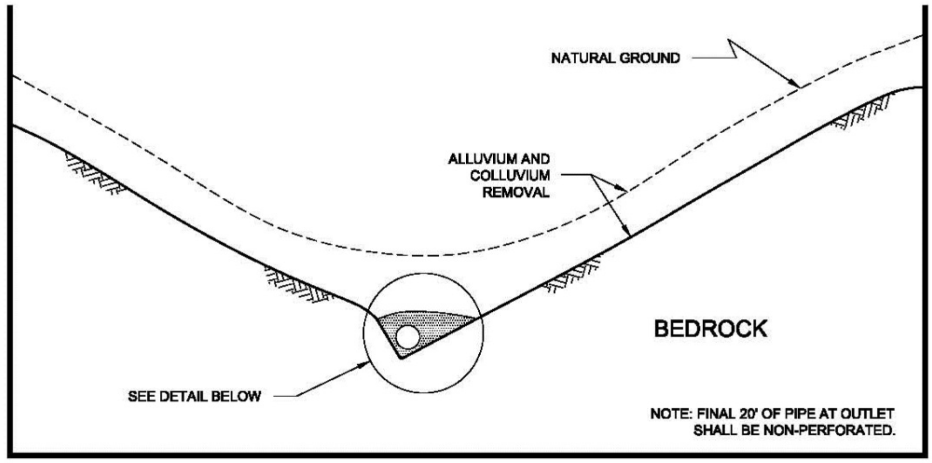
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant 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.

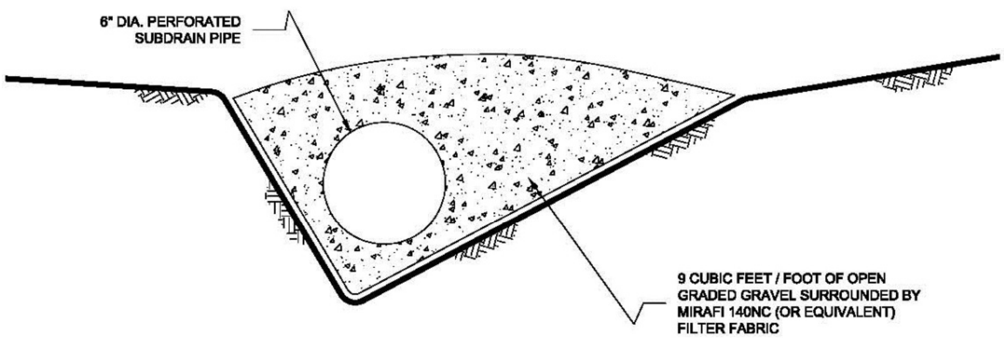
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

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:

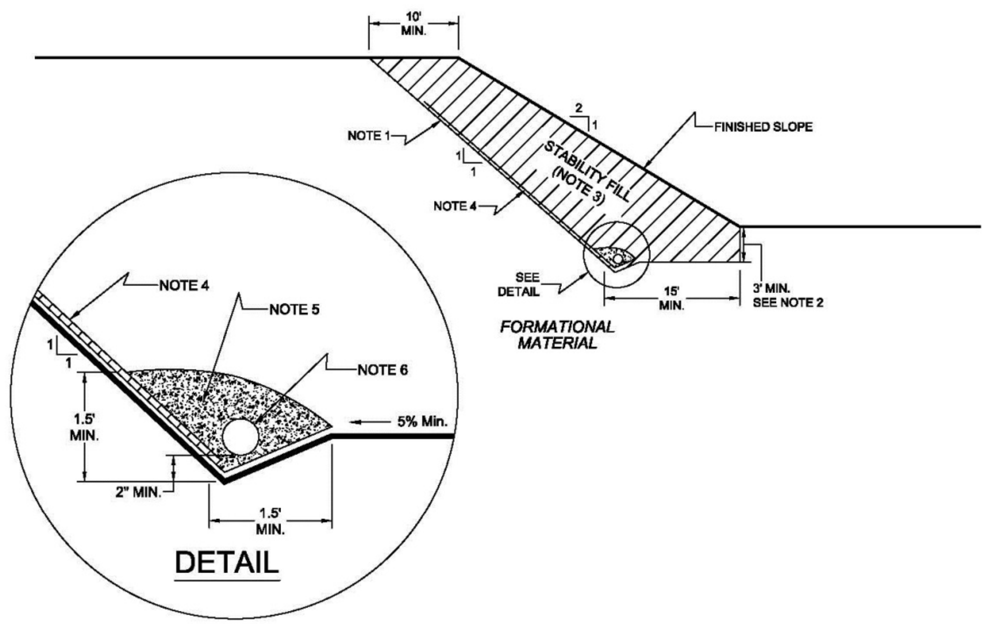
1......8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

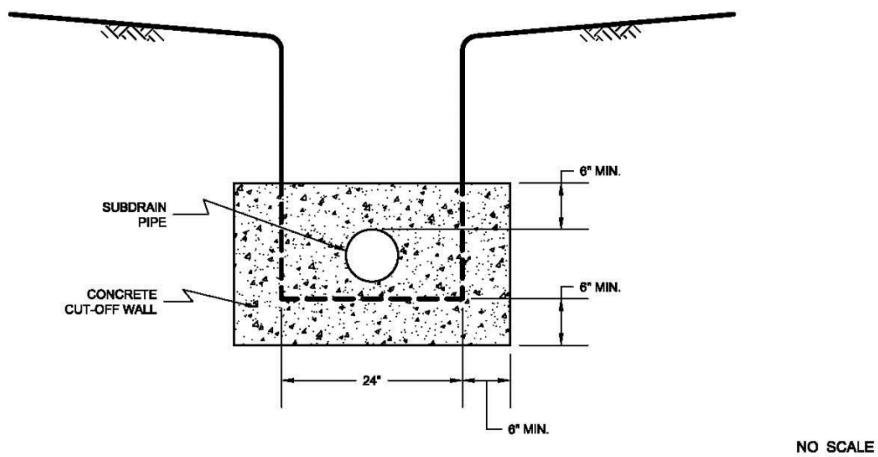
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

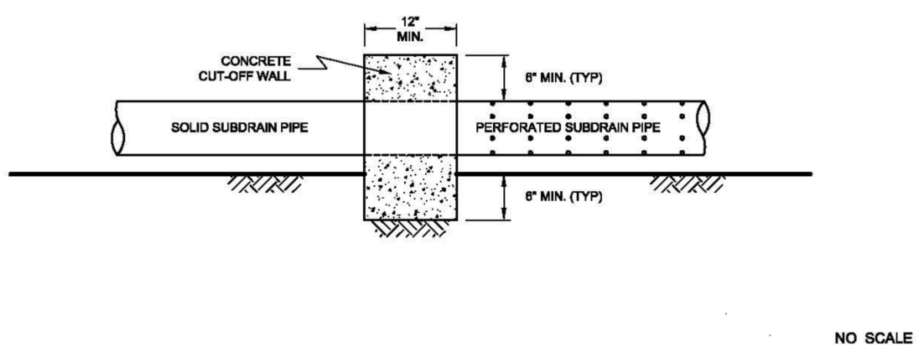
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



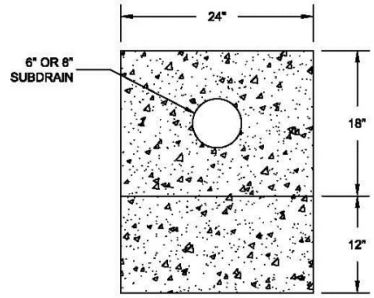
SIDE VIEW



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

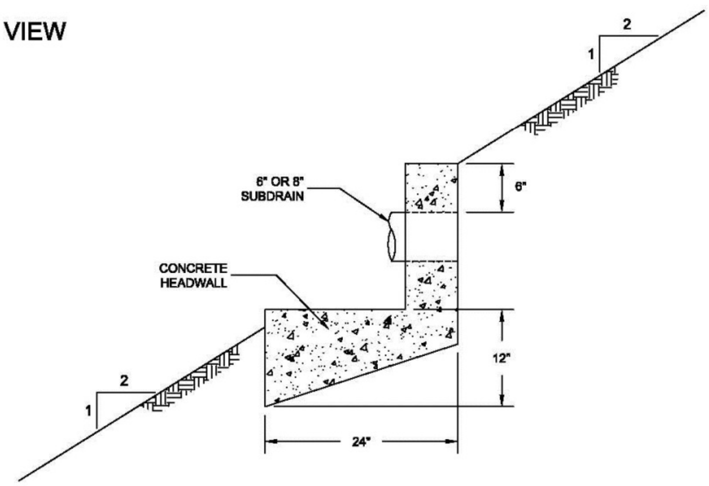
TYPICAL HEADWALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW



NOTE: HEADWALL SHOULD OUTLET AT TOE OF FILL SLOPE
OR INTO CONTROLLED SURFACE DRAINAGE

NO SCALE

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.