APPENDIX D
GEOTECHNICAL INVESTIGATION

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

GEOTECHNICAL INVESTIGATION

CORONADO RESIDENTIAL
THORNTON AVENUE
MENIFEE, CALIFORNIA

PREPARED FOR

QUINN COMMUNITIES
ENCINITAS, CALIFORNIA

MAY 13, 2022
PROJECT NO. T2974-22-01

Project No. T2974-22-01
May 13, 2022

Quinn Communities
364 2nd Street, Suite 5
Encinitas, California 92024

Attention: Mr. Stefan LaCasse

Subject: GEOTECHNICAL INVESTIGATION
CORONADO RESIDENTIAL
THORNTON AVENUE
MENIFEE, CALIFORNIA

Dear Mr. LaCasse:

In accordance with your authorization of Geocon Proposal IE-2962 dated March 9, 2022, Geocon West, Inc. (Geocon) herein submits the results of our due diligence geotechnical investigation for the proposed development. The accompanying report presents the results of our study and conclusions and recommendations pertaining to the geotechnical aspects of the proposed residential project. The site is considered suitable for 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 INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our geotechnical investigation for the proposed Coronado residential development planned for approximate 10-acres of undeveloped land (APNs 335-440-001 & -002) located south of Thornton Avenue, north of Esther Lane, between Family Circle and Amber Rock Drive in Menifee, California (see *Vicinity Map*, Figure 1). The purpose of the geotechnical investigation is to evaluate the surface and subsurface soil conditions and general site geology, and to identify geotechnical constraints that may affect development of the property.

The scope of our investigation included review of previous project reports, geologic mapping, subsurface exploration, percolation testing, laboratory testing, engineering analyses, and the preparation of this report. A summary of the information reviewed for this study is presented in the *List of References*.

The site was explored on April 19 and 20, 2022, using a Case 580 backhoe by excavating eight geotechnical test pits to depths between 8 and 18 feet below existing ground surface and six percolation test pits between 4 and 8 feet below existing ground. The approximate locations of the test pits and percolation tests are depicted on the *Geologic Map* (Figure 2). In-place moisture/density tests were taken within the test pits using a Troxler moisture density gauge per ASTM D6839, the test results are included on the test pit logs in Appendix A. A detailed discussion of the field investigation, including test pit logs and nuclear gauge moisture density results is presented in *Appendix A*.

Laboratory tests were performed on selected soil samples obtained during the investigation to evaluate pertinent physical soil properties for the proposed residential development. *Appendix B* presents a summary of the laboratory test results.

Recommendations presented herein are based on analyses of data obtained from our site investigation and our understanding of proposed site development. References reviewed to prepare this report are provided in the *List of References*. 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 site will be developed to include 74 residences, two water quality management plan / best management practice areas, interior streets, improvements along the south side of Thornton Avenue, and improvement and paving of Esther Lane from the western project boundary to Murrieta Road. The site is bounded on the north by Thornton Avenue, the south by Esther Lane, the west by a single-family residential development and on the east by undeveloped land. The latitude and longitude of the site are 33.7241 degrees, -117.2094 degrees, respectively.

At the time of our investigation the site was covered with a light to moderate growth of grass and weeds. The property is undeveloped and has been periodically plowed in the past. Based on historic aerial images, the site has been undeveloped since at least 1967. A stream crossed the southwestern corner of the site where a storm water channel is currently located. A ditch is present along the northern property boundary within the western half of the site. At the time of our study, this ditch had standing water and dense growth of riparian vegetation. A large storm water basin is present north of Thornton Avenue across from the site, within Sun Ranch Community Park. Esther Lane is currently an unpaved dirt road. Thornton Avenue along the site is a paved roadway with one lane in each direction.

The square site is generally level; sloping slightly to the southeastern corner. Site elevations range from a high of 1,461 feet above mean sea level (MSL) in the west central area of the property to a low of 1,448 feet MSL in the southeastern area of the site. Drainage is by sheet flow to the southeast.

Grading plans were not available for our review at the time of this investigation. However, the *Site Plan Layout Study Scheme 1*, prepared by FM Civil on March 28, 2022, depicts 74 residential lots with interior streets, and entry/exit points on Thornton Avenue and Esther Lane. Based on current ground surface at the site and surrounding roadways, we expect finished site elevations will be within about 3 feet of existing grades

Structural plans and loading information were not available for our review at the time of this investigation. We expect the residential structures will be between one and three stories in height, supported by conventional spread footing foundations and slab-on-grade floors. We expect structural column loads will not exceed 200 kips and wall loads will not exceed 2 kips per linear foot.

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

3. GEOLOGIC SETTING

The site is located between the Perris and Menifee valleys within an alluvial filled valley between granitic and metamorphic highlands of low to moderate relief. The property is located with the Perris block within the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are bounded on the north by the Transverse Ranges and the Cucamonga/Sierra Madre faults, the east by the San Andreas Fault. The province extends offshore to the west and south to the tip of Baja California. The Peninsular Ranges are characterized by granitic highlands of low to moderate relief surrounded by alluvial plains and valleys. Locally, Menifee is located near the center of the Perris Block which is a stable bedrock block bounded by the Elsinore and San Jacinto faults and extends from Riverside to Murrieta. We encountered recent alluvium overlying very old alluvium within the site. No faults or landslides are geologically mapped within or near the site (Morton, 2003).

4. GEOLOGIC MATERIALS

4.1 General

The primary geologic units at the site consist of alluvium, and very old alluvium. Some undocumented artificial fill is present on site in the form of stockpiles, generated on site and also dumped on site from adjacent residences. Geologic unit classification follows that of Morton, 2003. The descriptions of the soil and geologic conditions are depicted on the *Geologic Map*, Figure 2, discussed on the test pit logs in detail, and generally described in order of increasing age below.

4.2 Undocumented Artificial Fill (afu)

Undocumented artificial fill was observed near the western area of the site. It appears some fill was generated on site by a localized excavation and by adjacent homeowners to the west. Fill may also be present in the vicinity of the storm water channel in the southwestern area of the property. The fill is brown, dry to moist and consists of silty sand. The end-dumped stockpile fill may contain lawn waste and trash.

4.3 Alluvium (Qa)

Alluvium was encountered at the surface across the site to depths of 6 to 8 feet. The alluvium encountered consists predominantly of silty sand. The alluvium can be characterized as loose to medium dense, dry to slightly moist, and yellowish to strong brown. Porosity was observed and decreased with depth.

4.4 Very Old Alluvium (Qvof)

Very old alluvium was encountered below the alluvium in all test pits to the maximum depth explored of 18 feet. The very old alluvium encountered consists predominantly of red brown silty sand which is dense, slightly moist, and contains calcite deposits. Clay development is also observed within the soil.

4.5 Groundwater

We did not encounter groundwater during our investigation to the maximum depth explored of 18 feet. According to the California Department of Water Resources, wells within a one-mile radius indicated a depth to groundwater between 50 and 75 feet below the existing ground surface. It is not uncommon for seepage conditions to develop where none previously existed. Groundwater and seepage are dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. Proper surface drainage will be important to future performance of the project.

5. GEOLOGIC HAZARDS

5.1 Surface Fault Rupture

The numerous faults in southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on data developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (Bryant and Hart, 2007). 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 Fault Zone or a Riverside County Fault Hazard Zone. The closest active fault to the site is the Wildomar branch of the Elsinore fault zone located approximately ½ miles southwest. Faults within a 50-mile radius of the site are listed in Table 5.1.1. 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 5.1.1
ACTIVE FAULTS WITHIN 50 MILES OF THE SITE

Fault Name	Maximum Magnitude (Mw)	Distance from Site (mi)	Direction from Site
Glen Ivy North	6.8	7	W
Wildomar	6.8	8	SW
Casa Loma	6.9	12	NE
Claremont	6.9	14	NE
San Jacinto Valley	6.9	16	N
Clark	7.2	19	E
San Gorgonio	7.0	23	NE
San Gorgonio Pass	7.0	23	NE
Chino	6.7	24	NW
Glen Helen	6.7	28	N
Whittier	6.8	30	NW
San Andreas South	7.5	37	NE
Cucamonga	6.9	37	N
Pinto Mountain	7.2	37	NE
Morongo	7.2	41	NE
Coyote Creek	6.8	42	SE
Newport-Inglewood	7.1	44	W
San Andreas North	7.5	44	NE
North Frontal	7.2	48	N

TABLE 5.1.2
HISTORIC EARTHQUAKE EVENTS WITH RESPECT 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	19	N
Long Beach	March 10, 1933	6.4	44	W
Tehachapi	July 21, 1952	7.5	135	NW
San Fernando	February 9, 1971	6.6	83	WNW
Whittier Narrows	October 1, 1987	5.9	55	WNW
Sierra Madre	June 28, 1991	5.8	59	NW
Landers	June 28, 1992	7.3	55	NE
Big Bear	June 28, 1992	6.4	40	NE
Northridge	January 17, 1994	6.7	83	WNW
Hector Mine	October 16, 1999	7.1	81	NE
Ridgecrest China Lake Fault	July 5, 2019	7.1	143	NNW

5.2 Liquefaction

Liquefaction is a phenomenon in which saturated cohesionless soils are subject to a temporary loss of shear strength due to pore pressure buildup under the cyclic shear stresses associated with earthquakes. Primary factors that trigger liquefaction are: moderate to strong ground shaking (seismic source), relatively clean, loose granular soils (primarily poorly-graded sands and silty sands), and saturated soil conditions (shallow groundwater).

Based on the lack of shallow groundwater and the presence of shallow relatively dense very old alluvium, liquefaction would not be a design consideration for the proposed development. Furthermore, the potential for seismic “dry-sand” settlement to occur is considered low and would not be a design consideration.

5.3 Expansive Soil

The on-site soils generally consist of silty sands. Laboratory testing result indicates a sample of the near surface soil exhibits a “very low” expansion potential (expansion index [EI] of 20 or less) with test results showing an Expansion Index of 0.

5.4 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.

Remedial grading recommendations provided herein should be implemented during grading operations to reduce the hydrocompression potential of surficial soils. Hydrocompression would therefore not be a design consideration for this site due to the planned remedial grading coupled with the presence of shallow relatively dense very old alluvium.

5.5 Landslides

We did not observe evidence of previous or incipient slope instability within or adjacent to the site. Further, no landslides have been geologically mapped on or adjacent to the site. Therefore, landslide hazard to the site is not a design consideration.

5.6 Rockfall

The property is not located adjacent to bedrock hills. Therefore, rockfall hazards are not a design consideration for this project.

5.7 Slope Stability

Based on the project information available for our review and existing site grades, we do not expect slopes to be planned for this development. However, if slopes are planned, we do not expect slopes higher than 3 feet to be graded at slope inclinations of 2:1 or flatter. These slopes are expected to have adequate factors of safety in excess of 1.5 under static conditions and 1.1 under pseudo-static conditions, if constructed of onsite soils. Once detailed grading plans are available, this report should be reviewed and the stability of individual slopes should be evaluated, if necessary.

5.8 Tsunamis and Seiches

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 first order driving force for locally generated tsunamis offshore southern California is expected to be tectonic deformation from large earthquakes (Legg et al., 2002). The site is located greater than 30 miles from the nearest coastline, with the Santa Ana Mountains lying between the site and the Pacific Ocean; therefore, the risk associated with tsunamis is not a design consideration.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is not located downstream from Lake Perris or Canyon Lake. Therefore, a seiche hazard from this reservoir is not a design consideration.

6. SITE INFILTRATION

Percolation testing was performed in accordance with the procedures outlined in *Riverside County Flood Control and Water Conservation District LID BMP*, *Appendix A* for the proposed infiltration structures along the eastern area of the site. The percolation test locations are depicted on the *Geologic Map*, Figure 2.

Percolation test pits P-1 through P-6 were excavated to proposed basin bottom elevations as directed by FM Civil. One-foot excavations were performed at the bottom of each test pit to perform the percolation testing. Percolation data sheets are presented in *Appendix A* of this report. Results of the converted percolation test rates to infiltration test rates are presented in Table 6.

TABLE 6
INFILTRATION TEST RATES FOR PERCOLATION AREAS

Parameter	P-1	P-2	P-3	P-4	P-5	P-6
Depth (inches)	60	60	120	120	84	96
Test Type	Sandy	Sandy	Sandy	Sandy	Sandy	Sandy
Change in head over time: ΔH (inches)	5.4	3.8	3.4	5.5	9.6	7.4
Average head: Havg (inches)	9.3	10.1	10.3	9.2	19.2	20.3
Time Interval (minutes): Δt (minutes)	10	10	10	10	10	10
Radius of test hole: r (inches)	4	4	4	4	4	4
Tested Infiltration Rate: It (inches/hour)	5.7	3.8	3.3	5.9	5.4	4.0

The results of the infiltration testing indicate that infiltration at the locations tested ranged from 3.3 to 5.9 inches per hour.

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 on 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.

The infiltration feasibility per the *Water Quality Management Plan for the Santa Margarita Region of Riverside County* was evaluated for this site. Based on site typography and the lack of stream channels within or near the site, infiltration is not expected to negatively impact downstream water rights or other beneficial uses. The site is not located in an industrial area. Seasonal high ground water is expected to be more than 10 feet below the basin bottom elevations at the property. No water wells are known to be within 100 feet of the proposed infiltration basins. The site is likely not within a 2:1 (horizontal: vertical) projection of a septic leach line associated with the residence to the south or east. The soils in which the basins will be excavated are expected to have adequate physical and chemical properties for infiltration. The project civil engineer should review the infiltration rates and determine the storm water treatment structure most appropriate for this project.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

7.1.1 We opine that no soil or geologic conditions were encountered that would preclude the development of the property as proposed, provided the recommendations of this report are followed.

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

7.1.3 The surficial alluvium is considered unsuitable for the support of compacted fill or settlement-sensitive improvements based on the conditions as described on the test pit logs and moisture/density gauge test results. Remedial grading in the form of removal and compaction of these deposits will be required.

7.1.4 The test pits were loosely backfilled with the trench spoils generated from our field investigation. During grading operations, the test pit locations should locally be over-excavated 1 foot deeper than the recorded test pit depth and the soil replaced with engineered fill.

7.1.5 Following remedial grading as described herein, the planned residential structures can be supported on conventional shallow foundations with a slab-on-grade floor system.

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

7.1.7 Changes in the proposed rough grading, as outlined in this report, should be reviewed by this office. Once grading plans become available, they should be reviewed by this office to determine the necessity for review and possible revision of this report.

7.1.8 Geocon should review the grading and structural plans, and provide supplemental geotechnical recommendations as necessary.

7.2 Excavation and Soil Characteristics

7.2.1 Excavation of the undocumented fill, alluvium and very old alluvium should be possible with moderate effort using conventional heavy-duty equipment in proper functioning order. Excavation of the very old alluvium may require very heavy effort using conventional heavy-duty equipment during the grading operations. The grading and improvement contractors should review this report and evaluate the proper equipment to use for the planned excavations.

7.2.2 Laboratory test results indicate site soils exhibit Expansion Index test results of 0. The site soils are expected to be “non-expansive” (Expansion Index [EI] less than 20) as defined by 2019 California Building Code (CBC) Section 1803.5.3. Table 7.2.2 presents soil classifications based on the Expansion Index. Although unlikely, any medium to highly expansive soils encountered at the site should not be placed within 4 feet of the proposed foundations, flatwork or paving improvements.

TABLE 7.2.2
SOIL CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Expansion Classification	2016 CBC Expansion Classification
0 – 20	**Very Low**	**Non-Expansive**
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

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

7.2.4 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B and indicate that the on-site materials possess a sulfate exposure class of “S0” to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-19, Chapter 19.

7.2.5 Laboratory test results indicate resistivity of 5400, Ph of 7.7, chloride content of 40 ppm, and sulfate content of 0 ppm. Based on the laboratory test results, the site soils are not considered corrosive in accordance with the Caltrans *Corrosion Guidelines* (Version 3.2, May 2021) as shown in Table 7.2.5.

TABLE 7.2.5
CALTRANS CORROSION GUIDELINES

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

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

7.3 Grading

7.3.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in *Appendix C* and the grading ordinances of the City of Menifee.

7.3.2 Prior to commencing grading, a preconstruction conference should be held at the site with the City inspector, owner or developer, grading contractor, civil engineer, and geotechnical engineer in attendance. Special soil handling and/or the grading plans can be discussed at that time.

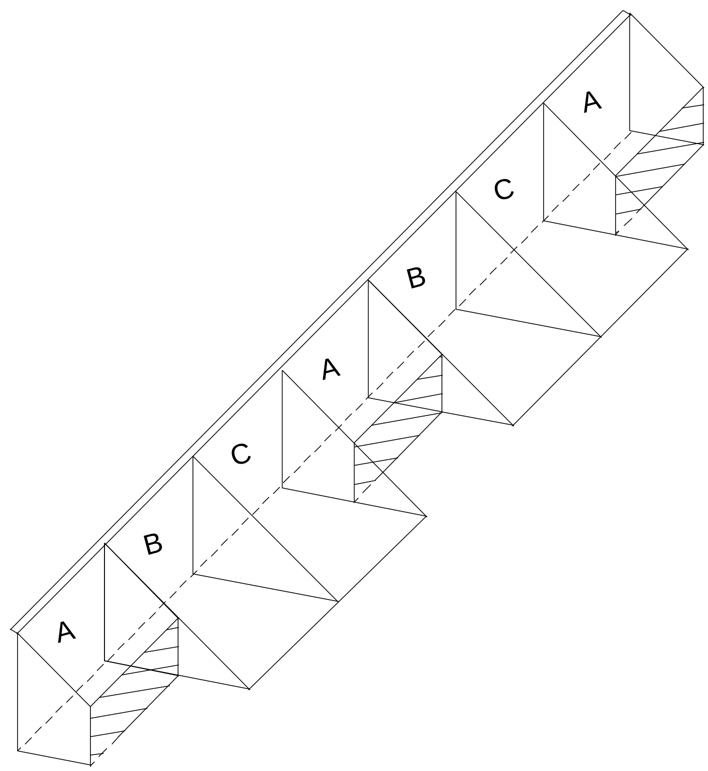
7.3.3 Site preparation should begin with the removal of deleterious material, debris and vegetation across the site and within the undocumented artificial fill end dumps. The depth of removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter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

7.3.4 The undocumented artificial fill end dumps and upper portions of alluvium, should be removed to expose very old alluvium which is non-porous and has an in-place relative compaction of at least 85 percent based on ASTM D1557. Remedial removal depths are expected to range between 6 to 9 feet. The actual depth of remedial grading should be evaluated by the engineering geologist during grading operations. The bottom of the excavations should be scarified to a depth of at least 1 foot, moisture conditioned, and compacted to 90 percent of the maximum dry density (ASTM D1557), prior to fill placement.

7.3.5 Remedial removals will be required adjacent to the existing housing development to the west where large excavation areas may not be possible without damage to existing block walls. Slot cutting may be necessary to perform the required removals in this area. Once site development and grading plans are prepared, Geocon should review the remedial grading in relation to the proposed structures.

7.3.6 Excavations adjacent to the existing block walls to the west that extend below a 1:1 (horizontal:vertical) projection downward and outward from the outside bottom edge of existing wall footings may utilize slot cutting to achieve remedial removals while maintaining support for the existing walls. Care should be taken by the grading contractor so that impact to existing improvements does not occur during slot-cut excavations. This may require reduced slot cut lengths if loose or otherwise unstable soil is encountered. The contractor should be aware that there is an inherent risk to slot-cutting as movement of near vertical excavations can cause stress relief features and vertical ground settlement outside of the excavation. The grading contractor should be prepared to take necessary steps to provide lateral stability/temporary buttressing if slot cut sidewalls experience instability.

7.3.7 We recommend that the initial temporary excavation along the property line be sloped back at a uniform 1:1 (horizontal to vertical) slope gradient or flatter for excavation of the existing soils to the necessary depth. The temporary slope may then be excavated using slot-cutting techniques (see illustration below).



7.3.8 The slot-cutting method employs the earth as a buttress and allows the earth excavation to proceed in phases. The initial excavation is made at a slope of 1:1. Alternate "A" slots should be worked first. Slots may be up to 4 feet in width. The backfill should be completed in the "A" slots before the "B" slots are excavated. After completing the backfill in the "B" slots, the "C" slots may be excavated. Slot-cutting is not recommended for vertical excavations greater than 9 feet in height or where surcharged by more than 1,000 pounds per linear foot. Where slot dimensions or surcharge loads exceed these amounts, Geocon should be contacted for additional recommendations.

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

7.3.10 The fill placed within 3 feet of proposed finish grade should possess a “low” expansion potential (EI of 50 or less), where practical.

7.3.11 Import fill (if necessary) should consist of granular materials with a “low” expansion potential (EI of 50 or less), generally free of deleterious material and rock fragments larger than 6 inches and should be compacted as recommended herein. Geocon should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to evaluate its suitability as fill material. Laboratory testing typically takes up to four working days to perform, therefore the grading contractor should plan for the laboratory testing in their schedule to provide sufficient time to allow for completion of testing prior to importing materials.

7.3.12 Fill slopes (if planned) should be overbuilt at least 2 feet and cut back to design grades for best performance. As an alternative, slopes should be compacted by back rolling with a loaded sheepsfoot roller at vertical intervals not to exceed 4 feet and should be track-walked at the completion of each slope such that the fill soils are uniformly compacted to at least 90 percent relative compaction to the face of the finished slope.

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

7.3.14 Infiltration basins should be excavated in native soil without compaction effort applied to the basin bottom. Basin maintenance should include the removal of silt from the basin bottom after each significant rain event for best performance.

7.4 Earthwork Grading Factors

7.4.1 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. Due to the variations in the actual shrinkage/bulking factors, a balance area should be provided to accommodate variations.

7.5 Utility Trench Backfill

7.5.1 Utility trenches should be properly backfilled in accordance with the requirements of the City of Menifee 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. The use of well graded crushed rock must be used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. 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. The use of 2-sack slurry and controlled low strength material (CLSM) are also acceptable. 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.

7.5.2 Utility excavation bottoms should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding materials, fill, gravel, concrete, or geogrid.

7.6 Seismic Design Criteria

7.6.1 Table 7.6.1 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 computer program *U.S. Seismic Design Maps*, provided by the Structural Engineers Association (SEA) to calculate the seismic design parameters. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.3.2 of the 2019 CBC and Table 20.3-1 of ASCE 7-16. The buildings and improvements should be designed using a Site Class D. The values presented on the following table are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 7.6.1
2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	D	Section 1613.2.2
Fill Thickness, T (feet)	T<20	--
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.415g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.522g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.200	Table 1613.2.3(1)
Site Coefficient, F_V	1.778	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.698g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.928g	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.132g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.619g	Section 1613.2.4 (Eqn 16-39)

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7.6.2 Table 7.6.2 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 7.6.2
2019 CBC SITE ACCELERATION DESIGN PARAMETERS

Parameter	Value	ASCE 7-16
Site Class	D	--
Fill Thickness, T (Feet)	$T \leq 20$	--
Mapped MCE_G Peak Ground Acceleration, PGA	0.513g	Figure 22-9
Site Coefficient, F_{PGA}	1.2	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.616g	Section 11.8.3 (Eqn 11.8-1)

7.6.3 Conformance to the criteria in Tables 6.7.1 and 6.7.2 for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur in the event of a large earthquake. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

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

7.6.5 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 6.84 magnitude event occurring at a hypocentral distance of 16.12 kilometers from the site.

7.6.6 Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.78 magnitude occurring at a hypocentral distance of 19.46 kilometers from the site.

7.6.7 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.

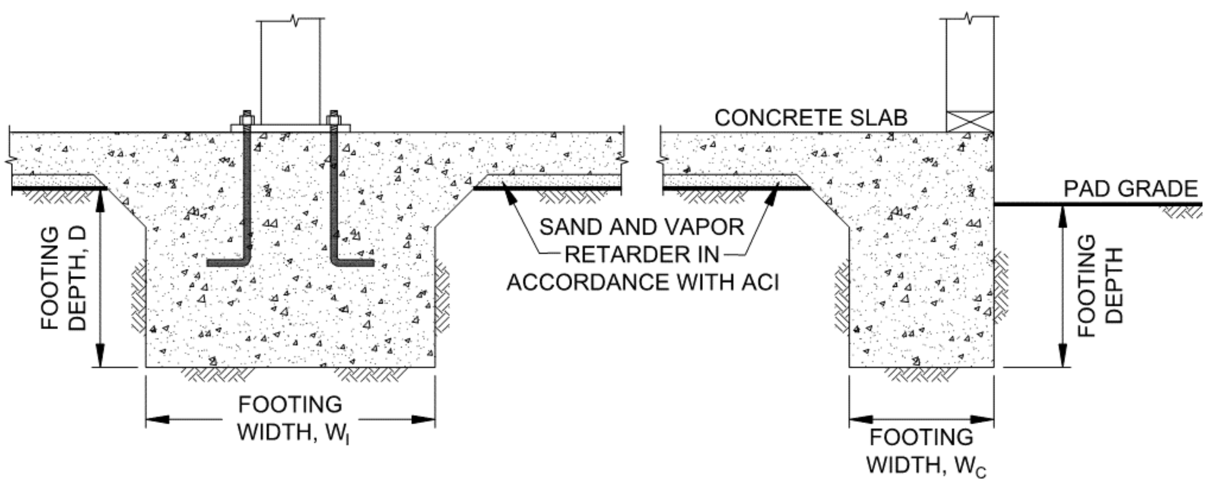
7.7 Foundation and Concrete Slabs-On-Grade Recommendations

7.7.1 The foundation recommendations presented herein are for the proposed residential buildings subsequent to the recommended grading. We understand that future buildings will be supported on conventional shallow foundations with concrete slabs-on-grade deriving support in newly placed engineered fill. Foundations for the structure should consist of continuous strip footings and/or isolated spread footings. Table 7.7.1 provides a summary of the foundation design recommendations.

TABLE 7.7.1
SUMMARY OF FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Continuous Foundation Width, W_C	12 inches
Minimum Isolated Foundation Width, W_I	24 inches
Minimum Foundation Depth, D	18 Inches Below Lowest Adjacent Grade
Minimum Steel Reinforcement	4 No. 4 Bars, 2 at the Top and 2 at the Bottom
Allowable Bearing Capacity	3,000 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	250 psf per Foot of Width
Maximum Allowable Bearing Capacity	4,000 psf
Estimated Total Settlement	½ Inch
Estimated Differential Settlement	¼ Inch in 40 Feet
Footing Size Used for Settlement	8-Foot Square
Design Expansion Index	50 or less

7.7.2 The foundations should be embedded in accordance with the recommendations herein and the following Wall/Column Footing Dimension Detail. The embedment depths should be measured from the lowest adjacent pad grade for both interior and exterior footings. Footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope (unless designed with a post-tensioned foundation system).



Wall/Column Footing Dimension Detail

7.7.3 The bearing capacity values presented herein are for dead plus live loads and may be increased by one-third when considering transient loads due to wind or seismic forces.

7.7.4 We should observe the foundation excavations prior to the placement of reinforcing steel and concrete to check that the exposed soil conditions are similar to those expected and that they have been extended to the appropriate bearing strata. Foundation modifications may be required if unexpected soil conditions are encountered.

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

7.8 Concrete Slabs-On-Grade

7.8.1 Concrete slabs-on-grade for the structures should be constructed in accordance with Table 7.8.1.

TABLE 7.8.1
MINIMUM CONCRETE SLAB-ON-GRADE RECOMMENDATIONS

Parameter	Value
Minimum Concrete Slab Thickness	4 inches
Minimum Steel Reinforcement	No. 3 Bars at 24 Inches on Center, Both Directions
Typical Slab Underlayment	3 to 4 Inches of Sand/Gravel/Base
Design Expansion Index	50 or less

7.8.2 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). In addition, the membrane should be installed in accordance with manufacturer’s recommendations and ASTM requirements and installed in a manner that prevents puncture. 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.

7.8.3 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. It is common to have 3 to 4 inches of sand for 5-inch and 4-inch thick slabs, respectively, in the southern California region. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 inches. The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing 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.

7.8.4 Concrete slabs should be provided with adequate crack-control joints, construction joints and/or expansion joints to reduce unsightly shrinkage cracking. The design of joints should consider criteria of the American Concrete Institute (ACI) when establishing crack-control spacing. Crack-control joints should be spaced at intervals no greater than 12 feet. Additional steel reinforcing, concrete admixtures and/or closer crack control joint spacing should be considered where concrete-exposed finished floors are planned.

7.8.5 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 as would be expected in any such concrete placement.

7.8.6 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 slabs for supporting residential-type loads.

7.8.7 Where exterior flatwork abuts the structure at entrant or exit areas, the exterior slab should be dowelled into the structure's foundation stemwall. 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.

7.8.8 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, stucco walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced 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.

7.9 Exterior Concrete Flatwork

7.9.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in Table 7.9.1. The recommended steel reinforcement would help reduce the potential for cracking.

TABLE 7.9.1
MINIMUM CONCRETE FLATWORK RECOMMENDATIONS

Expansion Index, EI	Minimum Steel Reinforcement* Options	Minimum Thickness
EI $\leq$ 50	6x6-W2.9/W2.9 (6x6-6/6) welded wire mesh	4 Inches
	No. 3 Bars 18 inches on center, Both Directions	

*In excess of 8 feet square.

7.9.2 The subgrade soil should be properly moisturized and compacted prior to the placement of steel and concrete. The subgrade soil should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content in accordance with ASTM D1557.

7.9.3 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. The steel reinforcement should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.

7.9.4 Concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the grading section prior to concrete placement. Subgrade soil should be properly compacted and the moisture content of subgrade soil should be verified prior to placing concrete. Base materials will not be required below concrete improvements.

7.9.5 Where exterior flatwork abuts the structure at entrant or exit points, the exterior slab should be dowelled into the structure's foundation stemwall. 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.

7.9.6 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.

7.10 Retaining Walls

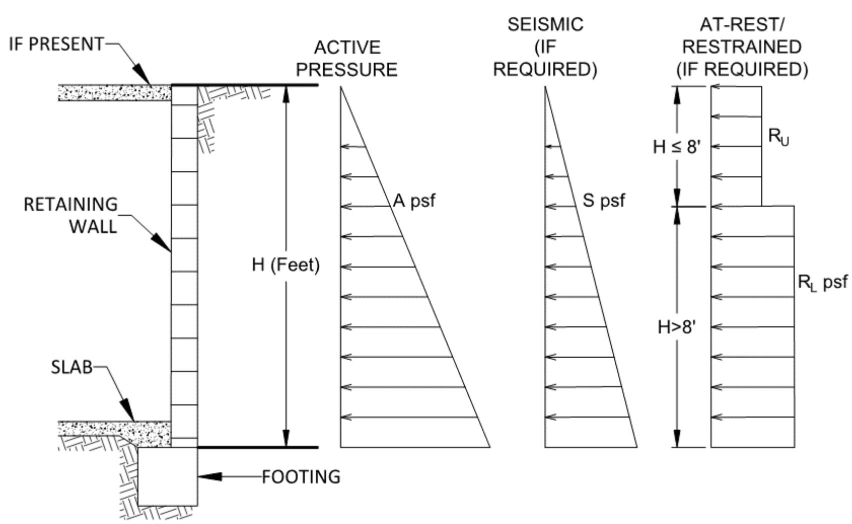
7.10.1 Retaining walls should be designed using the values presented in Table 7.10.1. Soil with an Expansion Index (EI) of greater than 50 should not be used as backfill material behind retaining walls.

TABLE 7.10.1
RETAINING WALL DESIGN RECOMMENDATIONS

Parameter	Value
Active Soil Pressure, A (Fluid Density, Level Backfill)	35 pcf
Active Soil Pressure, A (Fluid Density, 2:1 Sloping Backfill)	50 pcf
Seismic Pressure, S	15H psf
At-Rest/Restrained Walls Additional Uniform Pressure (0 to 8 Feet High)	7H psf
At-Rest/Restrained Walls Additional Uniform Pressure (8+ Feet High)	13H psf
Expected Expansion Index for the Subject Property	EI≤50

*H = height of the retaining portion of the wall

7.10.2 The project retaining walls should be designed as shown in the Retaining Wall Loading Diagram.



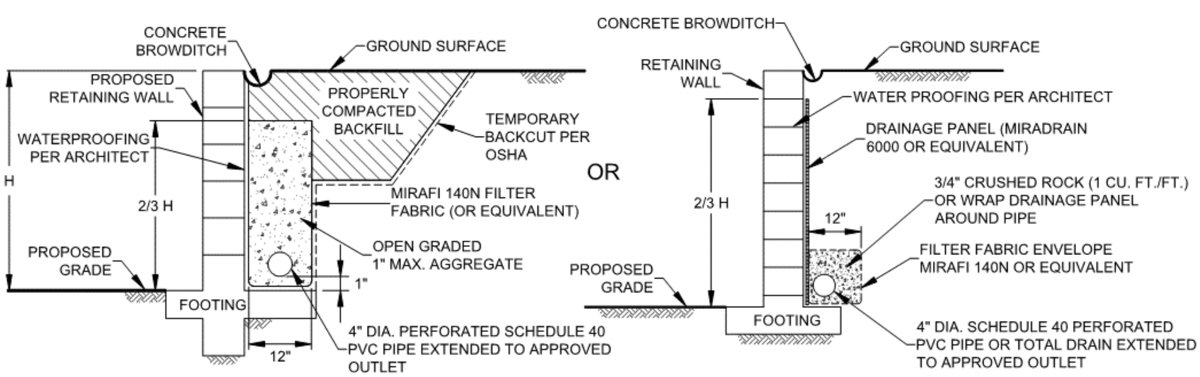
Retaining Wall Loading Diagram

7.10.3 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) at the top of the wall. Where walls are restrained from movement at the top (at-rest condition), an additional uniform pressure should be applied to the wall. For retaining walls subject to vehicular loads within a horizontal distance equal to two-thirds the wall height, a surcharge equivalent to 2 feet of fill soil should be added to the upper 10 feet of the retaining wall.

7.10.4 The structural engineer should determine the Seismic Design Category for the project in accordance with Section 1613.3.5 of the 2019 CBC or Section 11.6 of ASCE 7-16. For structures assigned to Seismic Design Category of D, E, or F, retaining walls that support more than 6 feet of backfill should be designed with seismic lateral pressure in accordance with Section 1803.5.12 of the 2019 CBC. The seismic load is dependent on the retained height where H is the height of the wall, in feet, and the calculated loads result in pounds per square foot (psf) exerted at the base of the wall and zero at the top of the wall.

7.10.5 Retaining walls should be designed to ensure stability against overturning sliding, and excessive foundation pressure. Where a keyway is extended below the wall base with the intent to engage passive pressure and enhance sliding stability, it is not necessary to consider active pressure on the keyway.

7.10.6 Drainage openings through the base of the wall (weep holes) should not be used 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 granular (EI of 50 or less) free-draining backfill material with no hydrostatic forces or imposed surcharge load. The retaining wall should be properly drained as shown in the Typical Retaining Wall Drainage Detail. If conditions different than those described are expected, or if specific drainage details are desired, Geocon should be contacted for additional recommendations.



Typical Retaining Wall Drainage Detail

7.10.7 The retaining walls may be designed using either the active and restrained (at-rest) loading condition or the active and seismic loading condition as suggested by the structural engineer. Typically, it appears the design of the restrained condition for retaining wall loading may be adequate for the seismic design of the retaining walls. However, the active earth pressure combined with the seismic design load should be reviewed and also considered in the design of the retaining walls.

7.10.8 In general, wall foundations should be designed in accordance with Table 7.10.8. The proximity of the foundation to the top of a slope steeper than 3:1 could impact the allowable soil bearing pressure. Therefore, retaining wall foundations should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.

TABLE 7.10.8
SUMMARY OF RETAINING WALL FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Retaining Wall Foundation Width	12 inches
Minimum Retaining Wall Foundation Depth	12 Inches
Minimum Steel Reinforcement	Per Structural Engineer
Allowable Bearing Capacity	2,500 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	250 psf per Foot of Width
Maximum Allowable Bearing Capacity	3,500 psf
Estimated Total Settlement	½ Inch
Estimated Differential Settlement	¼ Inch in 40 Feet

7.10.9 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls.

7.10.10 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.

7.10.11 Soil contemplated for use as retaining wall backfill, including import materials, should be identified in the field prior to backfill. At that time, Geocon should obtain samples for laboratory testing to evaluate its suitability. Modified lateral earth pressures may be necessary if the backfill soil does not meet the required expansion index or shear strength. City or regional standard wall designs, if used, are based on a specific active lateral earth pressure and/or soil friction angle. In this regard, on-site soil to be used as backfill may or may not meet the values for standard wall designs. Geocon should be consulted to assess the suitability of the on-site soil for use as wall backfill if standard wall designs will be used.

7.11 Lateral Loading

7.11.1 Table 7.11.1 should be used to help design the proposed structures and improvements to resist lateral loads for the design of footings or shear keys. 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.

TABLE 7.11.1
SUMMARY OF LATERAL LOAD DESIGN RECOMMENDATIONS

Parameter	Value
Passive Pressure Fluid Density	250 pcf
Coefficient of Friction (Concrete and Soil)	0.35
Coefficient of Friction (Along Vapor Barrier)	0.2 to 0.25*

*Per manufacturer's recommendations.

7.11.2 The passive and frictional resistant loads can be combined for design purposes. The lateral passive pressures may be increased by one-third when considering transient loads due to wind or seismic forces.

7.12 Preliminary Pavement Recommendations

7.12.1 The final pavement design should be based on R-value testing of soils at subgrade. Streets should be designed in accordance with the City of Menifee specifications and standards when final Traffic Indices (TI) and R-Value test results of subgrade soil are completed. For preliminary design purposes, we used an estimated R-value of 30 based on the soil classifications. Pavements should meet the minimum requirement for asphalt thickness in City of Menifee *Street Design Requirements* (Standard No. 80). Preliminary flexible pavement sections are presented in Table 7.12.1 for a range of applicable TI's. Geocon should be contacted if other roadway classifications and traffic indices are appropriate for the project.

TABLE 7.12.1
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Road Classification	Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Crushed Aggregate Base (inches)
General Local	5.5	30	4*	6*
Collector / Enhanced Local	8.0	30	6*	9
Secondary / Major / Arterial	10.0	30	6*	15

*Minimum Section per City of Menifee *Road Standard No. 80*

7.12.2 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 or slightly above optimum moisture content beneath pavement sections.

7.12.3 The crushed aggregated base and asphalt concrete materials should conform to Section 200-2.2 and Section 203-6, respectively, of the *Greenbook*. Base materials should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density at or slightly above optimum moisture content. Asphalt concrete should be compacted to a density of 95 percent of the laboratory Hveem density in accordance with ASTM D1561.

7.12.4 Where prefabricated concrete pavers (80 mm thick) will be used in site roadways and parking areas, it is acceptable from a geotechnical standpoint to construct the pavers over 1 inch of sand underlain by a properly prepared subgrade and aggregate base per the following table. The aggregate base should be compacted to at least 95 percent relative compaction as evaluated by ASTM D1557 (latest edition). Pavers should be constructed in accordance with the manufacture's guidelines. Preliminary paver design sections are presented in Table 7.12.4.

TABLE 7.12.4
PAVER DESIGN SECTIONS

Road Classification/Use	Traffic Index (TI)	Prefabricated Concrete Paver (inches)	Crushed Aggregate Base (inches)
General Local	5.5	3⅛	7½

7.12.5 Where concrete pavers will be placed in pedestrian walkway areas, and will not be subject to vehicle loading, the inclusion of a 4-inch layer of base over properly compacted subgrade underlying the pavers is acceptable from a geotechnical standpoint.

7.12.6 Where different pavement sections are to be constructed adjacent to each other, we recommend that consideration be given to the use of deepened base sections to maintain a uniform base thickness and avoid stepped cuts for placement of base material. This condition is expected to occur across the transition across the areas of asphalt paving and prefabricated pavers.

7.12.7 A rigid Portland cement concrete (PCC) pavement section should be placed in roadway aprons and cross gutters. We calculated the rigid pavement section in general conformance with the procedure recommended by the American Concrete Institute report ACI 330-21 *Commercial Concrete Parking Lots and Site Paving Design and Construction – Guide*. Table 7.12.7 provides the traffic categories and design parameters used for the calculations for 20-year design life.

TABLE 7.12.7
TRAFFIC CATEGORIES

Traffic Category	Description	Reliability (%)	Slabs Cracked at End of Design Life (%)
A	Car Parking Areas and Access Lanes	60	15
B	Entrance and Truck Service Lanes	60	15
C	School or City Buses (Excluding Large Articulated Buses)	75	15
D	Heavy Duty Trucks (Gross Weight of 80 Kips)	75	15
E	Garbage or Fire Truck Lane	75	15

7.12.8 We used the parameters presented in Table 7.12.8 to calculate the pavement design sections. We should be contacted to provide updated design sections, if necessary.

TABLE 7.12.8
RIGID PAVEMENT DESIGN PARAMETERS

Design Parameter	Design Value
Modulus of Subgrade Reaction, k	100 pci
Modulus of Rupture for Concrete, M_R	500 psi
Concrete Compressive Strength	3,000 psi
Concrete Modulus of Elasticity, E	3,150,000 psi

7.12.9 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 7.12.9.

TABLE 7.12.9
RIGID VEHICULAR PAVEMENT RECOMMENDATIONS

Traffic Category	Trucks Per Day	Portland Cement Concrete, T (Inches)
A = Car Parking Areas and Access Lanes	10	5½
B = Entrance and Truck Service Lanes	10	6
	50	6½
	100	6½
C = School or City Buses	50	9½
	100	9½
D = Heavy Duty Trucks	50	6½
	100	7
E = Garbage or Fire Truck Lanes	5	6½
	10	7

7.12.10 The PCC vehicular pavement should be placed over subgrade soil that is compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. The garbage truck pad should be large enough such that all wheels are on the concrete pad during the loading operations.

7.12.11 Adequate joint spacing should be incorporated into the design and construction of the rigid pavement in accordance with Table 7.12.11.

TABLE 7.12.11
MAXIMUM JOINT SPACING

Pavement Thickness, T (Inches)	Maximum Joint Spacing (Feet)
$4<T<5$	10
$5\leq T<6$	12.5
$6\leq T$	15

7.12.12 The rigid pavement should also be designed and constructed incorporating the parameters presented in Table 7.12.12.

TABLE 7.12.12
ADDITIONAL RIGID PAVEMENT RECOMMENDATIONS

Subject	Value
Thickened Edge	1.2 Times Slab Thickness Adjacent to Structures
	1.5 Times Slab Thickness Adjacent to Soil
	Minimum Increase of 2 Inches
	4 Feet Wide
Crack Control Joint Depth	Early Entry Sawn = T/6 to T/5, 1.25 Inch Minimum
	Conventional (Tooled or Conventional Sawing) = T/4 to T/3
Crack Control Joint Width	¼-Inch for Sealed Joints and Per Sealer Manufacturer's Recommendations
	$^{1}/_{16}$- to $^{1}/_{4}$-Inch is Common for Unsealed Joints

7.12.13 Reinforcing steel will not be necessary within the concrete for geotechnical purposes with the possible exception of dowels at construction joints as discussed herein.

7.12.14 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. Crack-control joints should be sealed with an appropriate sealant to prevent the migration of water through the control joint to the subgrade materials. The depth of the crack-control joints should be in accordance with the referenced ACI guide.

7.12.15 To provide load transfer between adjacent pavement slab sections, a butt-type construction joint should be constructed. The butt-type joint should be thickened by at least 20 percent at the edge and taper back at least 4 feet from the face of the slab.

7.12.16 Concrete curb/gutter should be placed on soil subgrade compacted to a dry density of at least 90 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Cross-gutters that receives vehicular should be placed on subgrade soil compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. Base materials should not be placed below the curb/gutter, or cross-gutters so water is not able to migrate from the adjacent parkways to the pavement sections. Where flatwork is located directly adjacent to the curb/gutter, the concrete flatwork should be structurally connected to the curbs to help reduce the potential for offsets between the curbs and the flatwork.

7.13 Excavation Slopes, Shoring and Tiebacks

7.13.1 The recommendations included herein are provided for stable excavations. It is the responsibility of the contractor and their competent person to ensure all excavations, temporary slopes and trenches are properly constructed and maintained in accordance with applicable OSHA guidelines in order to maintain safety and the stability of the excavations and adjacent improvements. These excavations should not be allowed to become saturated or to dry out. Surcharge loads should not be permitted to a distance equal to the height of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those recommended or closer than 15 feet from an existing surface improvement should be shored in accordance with applicable OSHA codes and regulations.

7.13.2 The stability of the excavations is dependent on the design and construction of the shoring system and site conditions. Therefore, Geocon cannot be responsible for site safety and the stability of the proposed excavations.

7.13.3 The design of temporary shoring is governed by soil and groundwater conditions, and by the depth and width of the excavated area. Continuous support of the excavation face can be provided by a system of soldier piles and wood lagging or other applicable techniques. Excavations exceeding 15 feet may require soil nails, tieback anchors or internal bracing to provide additional wall restraint.

7.13.4 The condition of existing buildings, streets, sidewalks, and other structures/improvements around the perimeter of the planned excavation should be documented prior to the start of shoring and excavation work. Special attention should be given to documenting existing cracks or other indications of differential settlement within these adjacent structures, pavements and other improvements. Underground utilities sensitive to settlement should be videorecorded (i.e. CCTV) prior to construction to check the integrity of pipes. In addition, monitoring points should be established indicating location and elevation around the excavation and upon existing buildings. These points should be monitored on a weekly basis during excavation work and on a monthly basis thereafter.

7.13.5 Temporary shoring should be designed using a lateral pressure envelope acting on the back of the shoring. The project shoring engineer should determine the applicable soil distribution for the design of the temporary shoring system. Additional lateral earth pressure due to the surcharging effects from construction equipment, sloping backfill, planned stockpiles, adjacent structures and/or traffic loads should be considered, where appropriate, during design of the shoring system.

7.14 Site Drainage and Moisture Protection

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

7.14.2 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 can infiltrate the soil for prolonged periods of time.

7.14.3 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 to collect excess irrigation water and transmit it to drainage structures or impervious above-grade planter boxes be used. In addition, where landscaping is planned adjacent to the pavement, we recommend construction of a cutoff wall or the use of an impermeable geosynthetic along the edge of the pavement that extends at least 6 inches below the bottom of the base material.

7.14.4 Infiltration systems should be located a minimum of 20 feet laterally from the outside edge of structural foundations, so that the percolation of water through the soil does not intersect a 1:1 (horizontal:vertical) structural load projection from the outside bottom edge of foundations.

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

7.15 Plan Review

7.15.1 Grading, shoring, and foundation plans should be reviewed by the Geotechnical Engineer (a representative of Geocon), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations, if necessary.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

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

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

The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to 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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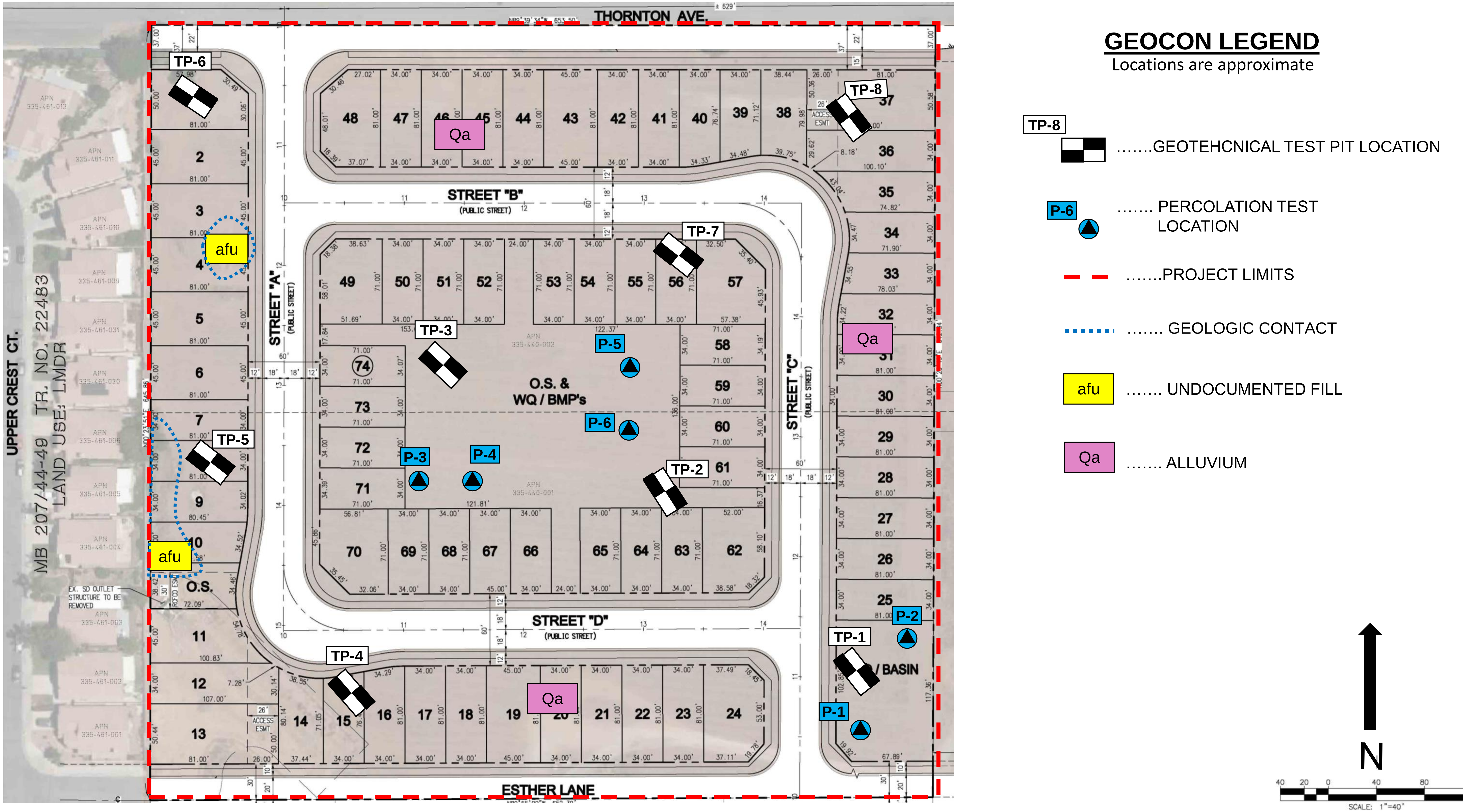
SOURCE: Google Earth 2022

VICINITY MAP

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

CORONADO RESIDENTIAL
THORNTON AVENUE
MENIFEE, CALIFORNIA

HD			MAY 2022	PROJECT NO. T2974-22-01	FIG. 1



Source: FM Civil Engineers, Inc., *City of Menifee Coronado Site Plan*, dated March 28, 2022.

GEOCON WEST, INC. GEOTECHNICAL, ENVIRONMENTAL, MATERIALS 41571 CORNING PLACE #101, MURRIETA, CALIFORNIA 92562 PHONE 951-304-2300 FAX 951-304-2392			GEOLOGIC MAP CORONADO RESIDENTIAL THORNTON AVENUE MENIFEE, CALIFORNIA		
HD			MAY 2022	PROJECT NO. T2974-22-01	FIG. 2

APPENDIX A

APPENDIX A

EXPLORATORY EXCAVATIONS

Geocon performed the field investigation on April 19 and 20, 2022. Our subsurface exploration consisted of excavating eight geotechnical test pits and six percolation test pits utilizing a Case 580 backhoe. Geotechnical test pits TP-1 through TP-8 were excavated to depths ranging from 8 and 18 feet. Percolation test pits were excavated to depths of 4 to 8 feet at the direction of the project civil engineer.

We collected bulk samples from the test pits and performed in place moisture and density testing with a nuclear density gage per ASTM D6938. We estimated elevations shown on the test pit logs using Google Earth. The soil conditions encountered in the test pits were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS).

Percolation testing was performed in accordance with Riverside County Flood Control and Water Conservation District *Low Impact Development Best Management Practices Handbook*.

Logs of the test pits are presented on Figures A-1 through A-14. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The percolation data sheets are presented on Figures A-15 through A-20. The approximate locations of the test pits and percolation tests are depicted on the *Geologic Map*, Figure 2.

TEST PIT TP-1

ELEV. (MSL.) **1448** DATE COMPLETED **4/19/2022**

EQUIPMENT **Case 580 Backhoe w/bucket 24"** BY: **L. 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	TP-1@0-5			SM	**ALLUVIUM (Qa)** Silty SAND, medium dense, dry, yellowish strong brown; fine to coarse sand; some porosity			
1							92.3	4.5
2								
3							93.1	2.5
4								
5					-Becomes slightly moist; less porosity			
6								
8				SM	**VERY OLD ALLUVIUM (Qvof)** Silty SAND, dense, slightly moist, red brown; fine to medium sand; some coarse sand; calcite stringers; clay development; slightly oxidized			
10								
12								
14					Total Depth = 14' No Groundwater encountered Backfilled with cuttings 4/19/2022			

Figure A-1,
Log of Test Pit TP-1, Page 1 of 1

T2974-22-01 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

TEST PIT TP-2

ELEV. (MSL.) **1455** DATE COMPLETED **4/19/2022**

EQUIPMENT **Case 580 Backhoe w/bucket 24"** BY: **L. 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	**ALLUVIUM (Qa)** Silty SAND, medium dense, dry, yellowish strong brown; fine to coarse sand; some porosity		104.7	1.7
2					-Becomes slightly moist; less porosity		106.3	5.5
6	TP-2@6-10'			SM	**VERY OLD ALLUVIUM (Qvof)** Silty SAND, dense, slightly moist, red brown; fine to medium sand; some coarse sand; calcite stringers; clay development; slightly oxidized			
18					Total Depth = 18' No Groundwater encountered Backfilled with cuttings 4/19/2022			

Figure A-2,
Log of Test Pit TP-2, Page 1 of 1

T2974-22-01 BORING LOGS.GPJ

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

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

PROJECT NO. T2974-22-01

TEST PIT TP-3

ELEV. (MSL.) **1455** DATE COMPLETED **4/19/2022**

EQUIPMENT **Case 580 Backhoe w/bucket 24"** BY: **L. 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	**ALLUVIUM (Qa)** Silty SAND, medium dense, dry, yellowish strong brown; fine to coarse sand; some porosity		94.2	3.3
2					-Becomes slightly moist; less porosity		91.1	1.6
4								
6								
8	TP-3@7-10'			SM	**VERY OLD ALLUVIUM (Qvof)** Silty SAND, dense, slightly moist, red brown; fine to medium sand; some coarse sand; calcite stringers; clay development; slightly oxidized			
10								
12								
14					-Becomes fine to coarse sand			
16								
					Total Depth = 17' No Groundwater encountered Backfilled with cuttings 4/19/2022			

Figure A-3,
Log of Test Pit TP-3, Page 1 of 1

T2974-22-01 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.

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TEST PIT TP-4

ELEV. (MSL.) **1454** DATE COMPLETED **4/19/2022**

EQUIPMENT **Case 580 Backhoe w/bucket 24"** BY: **L. 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	TP-4@0-5			SM	**ALLUVIUM (Qa)** Silty SAND, medium dense, dry, yellowish strong brown; fine to coarse sand; some porosity		94.2	1.8
2					-Becomes slightly moist; less porosity		95.0	4.9
4								
6				SM	**VERY OLD ALLUVIUM (Qvof)** Silty SAND, dense, slightly moist, red brown; fine to medium sand; some coarse sand; calcite stringers; clay development; slightly oxidized			
8					Total Depth = 8' No Groundwater encountered Backfilled with cuttings 4/19/2022			

Figure A-4,
Log of Test Pit TP-4, Page 1 of 1

T2974-22-01 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.

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PROJECT NO. T2974-22-01

TEST PIT TP-5

ELEV. (MSL.) **1461** DATE COMPLETED **4/19/2022**

EQUIPMENT **Case 580 Backhoe w/bucket 24"** BY: **L. 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	**ALLUVIUM (Qa)** Silty SAND, medium dense, dry, yellowish strong brown; fine to coarse sand; some porosity -Becomes slightly moist; less porosity		102.7 105.3	2.0 4.7
2								
4								
6				SM	**VERY OLD ALLUVIUM (Qvof)** Silty SAND, dense, slightly moist, red brown; fine to medium sand; some coarse sand; calcite stringers; clay development; slightly oxidized			
8					Total Depth = 8' No Groundwater encountered Backfilled with cuttings 4/19/2022			

Figure A-5,
Log of Test Pit TP-5, Page 1 of 1

T2974-22-01 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. T2974-22-01

TEST PIT TP-6

ELEV. (MSL.) **1459** DATE COMPLETED **4/19/2022**

EQUIPMENT **Case 580 Backhoe w/bucket 24"** BY: **L. 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	TP-6@0-5'			SM	**ALLUVIUM (Qa)** Silty SAND, medium dense, dry, yellowish strong brown; fine to coarse sand; some porosity		104.1	2.3
2					-Becomes slightly moist; less porosity		106.3	5.1
4								
6								
8				SM	**VERY OLD ALLUVIUM (Qvof)** Silty SAND, dense, slightly moist, red brown; fine to medium sand; some coarse sand; calcite stringers; clay development; slightly oxidized			
					Total Depth = 8' No Groundwater encountered Backfilled with cuttings 4/19/2022			

Figure A-6,
Log of Test Pit TP-6, Page 1 of 1

T2974-22-01 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.

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DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**TEST PIT TP-7** ELEV. (MSL.) **1455** DATE COMPLETED **4/19/2022** EQUIPMENT **Case 580 Backhoe w/bucket 24"** BY: **L. WEIDMAN**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	TP-7@0-3			SM	**ALLUVIUM (Qa)** Silty SAND, medium dense, dry, yellowish strong brown; fine to coarse sand; some porosity			
2					-Becomes slightly moist; less porosity			
4								
6								
8					-Becomes moist			
10				SM	**VERY OLD ALLUVIUM (Qvof)** Silty SAND, dense, slightly moist, red brown; fine to medium sand; some coarse sand; calcite stringers; clay development; slightly oxidized			

Total Depth = 8'
No Groundwater encountered
Backfilled with cuttings 4/19/2022

Figure A-7,
Log of Test Pit TP-7, Page 1 of 1

T2974-22-01 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. T2974-22-01

TEST PIT TP-8

ELEV. (MSL.) **1451** DATE COMPLETED **4/19/2022**

EQUIPMENT **Case 580 Backhoe w/bucket 24"** BY: **L. 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	**ALLUVIUM (Qa)** Silty SAND, medium dense, dry, yellowish strong brown; fine to coarse sand; some porosity		104.6	2.1
2					-Becomes slightly moist; less porosity		106.9	5.0
4								
6								
8								
10	TP-8@9-11			SM	**VERY OLD ALLUVIUM (Qvof)** Silty SAND, dense, slightly moist, red brown; fine to medium sand; some coarse sand; calcite stringers; clay development; slightly oxidized			

Total Depth = 8'
No Groundwater encountered
Backfilled with cuttings 4/19/2022

Figure A-8,
Log of Test Pit TP-8, Page 1 of 1

T2974-22-01 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. T2974-22-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	**BORING P-1** ELEV. (MSL.) **1448** DATE COMPLETED **4/20/2022** EQUIPMENT **Case 580 Backhoe w/bucket 24"** BY: **L. WEIDMAN**	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**ALLUVIUM (Qa)** Silty SAND, medium dense, dry, yellowish strong brown; fine to coarse sand; some porosity			
2					-Becomes slightly moist; less porosity			
4	P-1@4							
					Total Depth = 4' No Groundwater encountered Percolation Test Equipment set Presaturated with 5 gallons of water Backfilled with cuttings 4/20/2022			

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

T2974-22-01 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.

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DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P-2 ELEV. (MSL.) 1448 DATE COMPLETED 4/20/2022 EQUIPMENT Case 580 Backhoe w/bucket 24" BY: L. WEIDMAN	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0 2 4	P-2@4			SM	**ALLUVIUM (Qa)** Silty SAND, medium dense, dry, yellowish strong brown; fine to coarse sand; some porosity -Becomes slightly moist; less porosity			
					Total Depth = 4' No Groundwater encountered Percolation Test Equipment set Presaturated with 5 gallons of water Backfilled with cuttings 4/20/2022			

Figure A-10,
Log of Boring P-2, Page 1 of 1

T2974-22-01 BORING LOGS.GPJ

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

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

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P-3 ELEV. (MSL.) 1455 DATE COMPLETED 4/20/2022 EQUIPMENT Case 580 Backhoe w/bucket 24" BY: L. WEIDMAN	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**ALLUVIUM (Qa)** Silty SAND, medium dense, dry, yellowish strong brown; fine to coarse sand; some porosity			
2					-Becomes slightly moist; less porosity			
4								
6	P-3@7			SM	**VERY OLD ALLUVIUM (Qvof)** Silty SAND, dense, slightly moist, red brown; fine to medium sand; some coarse sand; calcite stringers; clay development; slightly oxidized			

Total Depth = 7'
No Groundwater encountered
Percolation Test Equipment set
Presaturated with 5 gallons of water
Backfilled with cuttings 4/20/2022

Figure A-11,
Log of Boring P-3, Page 1 of 1

T2974-22-01 BORING LOGS.GPJ

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

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

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P-4 ELEV. (MSL.) 1455 DATE COMPLETED 4/20/2022 EQUIPMENT Case 580 Backhoe w/bucket 24" BY: L. WEIDMAN	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**ALLUVIUM (Qa)** Silty SAND, medium dense, dry, yellowish strong brown; fine to coarse sand; some porosity			
2					-Becomes slightly moist; less porosity			
4								
6	P-4@7							
					Total Depth = 7' No Groundwater encountered Percolation Test Equipment set Presaturated with 5 gallons of water Backfilled with cuttings 4/20/2022			

Figure A-12,
Log of Boring P-4, Page 1 of 1

T2974-22-01 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-5

ELEV. (MSL.) **1455** DATE COMPLETED **4/20/2022**

EQUIPMENT **Case 580 Backhoe w/bucket 24"** BY: **L. 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	**ALLUVIUM (Qa)** Silty SAND, medium dense, dry, yellowish strong brown; fine to coarse sand; some porosity			
2					-Becomes slightly moist; less porosity			
4								
6								
8	P-5@8			ML	**VERY OLD ALLUVIUM (Qvof)** Sandy SILT, hard, slightly moist, red brown; fine to medium sand; some coarse sand; calcite stringers; clay development; slightly oxidized			

Total Depth = 8'
No Groundwater encountered
Percolation Test Equipment set
Presaturated with 5 gallons of water
Backfilled with cuttings 4/20/2022

Figure A-13,
Log of Boring P-5, Page 1 of 1

T2974-22-01 BORING LOGS.GPJ

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

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

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P-6 ELEV. (MSL.) 1455 DATE COMPLETED 4/20/2022 EQUIPMENT Case 580 Backhoe w/bucket 24" BY: L. WEIDMAN	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0				SM	**ALLUVIUM (Qa)** Silty SAND, medium dense, dry, yellowish strong brown; fine to coarse sand; some porosity			
2					-Becomes slightly moist; less porosity			
4								
6								
				ML	**VERY OLD ALLUVIUM (Qvof)**			
8	P-6@8				Sandy SILT, hard, slightly moist, red brown; fine to medium sand; some coarse sand; calcite stringers; clay development; slightly oxidized			

Total Depth = 8'
No Groundwater encountered
Percolation Test Equipment set
Presaturated with 5 gallons of water
Backfilled with cuttings 4/20/2022

Figure A-14,
Log of Boring P-6, Page 1 of 1

T2974-22-01 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.

PERCOLATION TEST REPORT

Project Name:	Coronado Condos	**Project No.:**	T2974-22-01
Test Hole No.:	P-1	**Date Excavated:**	4/19/2022
Length of Test Pipe:	60.0 inches	**Soil Classification:**	SM
Height of Pipe above Ground:	12.0 inches	**Presoak Date:**	4/19/2022
Depth of Test Hole:	60.0 inches	**Perc Test Date:**	4/20/2022
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	8:30 AM 8:55 AM	25	25	12.0	0.0	12.0	2.1
2	8:55 AM 9:20 AM	25	50	12.0	0.0	12.0	2.1

Soil Criteria: Sandy

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	9:00 AM 9:10 AM	10	10	12.0	5.9	6.1	1.6
2	9:10 AM 9:20 AM	10	20	12.0	5.6	6.4	1.6
3	9:20 AM 9:30 AM	10	30	12.0	6.6	5.4	1.9
4	9:30 AM 9:40 AM	10	40	12.0	6.7	5.3	1.9
5	9:40 AM 9:50 AM	10	50	12.0	6.8	5.2	1.9
6	9:50 AM 10:00 AM	10	60	12.0	6.6	5.4	1.9

Infiltration Rate (in/hr):	5.7
Radius of test hole (in):	4
Average Head (in):	9.3

Figure A-15

PERCOLATION TEST REPORT

Project Name:	Coronado Condos			**Project No.:**		T2974-22-01
Test Hole No.:	P-2			**Date Excavated:**		4/19/2022
Length of Test Pipe:		60.0	inches	**Soil Classification:**		SM
Height of Pipe above Ground:		12.0	inches	**Presoak Date:**		4/19/2022
Depth of Test Hole:		60.0	inches	**Perc Test Date:**		4/20/2022
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	8:31 AM 8:56 AM	25	25	12.0	0.0	12.0	2.1
2	8:56 AM 9:21 AM	25	50	12.0	0.0	12.0	2.1

Soil Criteria: Sandy

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	9:01 AM 9:11 AM	10	10	12.0	7.1	4.9	2.0
2	9:11 AM 9:21 AM	10	20	12.0	7.6	4.4	2.3
3	9:21 AM 9:31 AM	10	30	12.0	8.4	3.6	2.8
4	9:31 AM 9:41 AM	10	40	12.0	8.8	3.2	3.1
5	9:41 AM 9:51 AM	10	50	12.0	7.8	4.2	2.4
6	9:51 AM 10:01 AM	10	60	12.0	8.2	3.8	2.6

Infiltration Rate (in/hr):	3.8	
Radius of test hole (in):	4	**Figure A-16**
Average Head (in):	10.1	

PERCOLATION TEST REPORT

Project Name:	Coronado Condos	**Project No.:**	T2974-22-01
Test Hole No.:	P-3	**Date Excavated:**	4/19/2022
Length of Test Pipe:	120.0 inches	**Soil Classification:**	SM
Height of Pipe above Ground:	0.0 inches	**Presoak Date:**	4/19/2022
Depth of Test Hole:	120.0 inches	**Perc Test Date:**	4/20/2022
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	8:32 AM 8:57 AM	25	25	12.0	0.0	12.0	2.1
2	8:57 AM 9:22 AM	25	50	12.0	0.0	12.0	2.1

Soil Criteria: Sandy

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	10:15 AM 10:25 AM	10	10	12.0	6.6	5.4	1.9
2	10:25 AM 10:35 AM	10	20	12.0	6.7	5.3	1.9
3	10:35 AM 10:45 AM	10	30	12.0	7.8	4.2	2.4
4	10:45 AM 10:55 AM	10	40	12.0	7.8	4.2	2.4
5	10:55 AM 11:05 AM	10	50	12.0	8.8	3.2	3.1
6	11:05 AM 11:15 AM	10	60	12.0	8.6	3.4	3.0

Infiltration Rate (in/hr):	3.3
Radius of test hole (in):	4
Average Head (in):	10.3

Figure A-17

PERCOLATION TEST REPORT

Project Name:	Coronado Condos	**Project No.:**	T2974-22-01
Test Hole No.:	P-4	**Date Excavated:**	4/19/2022
Length of Test Pipe:	120.0 inches	**Soil Classification:**	SMg
Height of Pipe above Ground:	0.0 inches	**Presoak Date:**	4/19/2022
Depth of Test Hole:	120.0 inches	**Perc Test Date:**	4/20/2022
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	8:33 AM 8:58 AM	25	25	12.0	0.6	11.4	2.2
2	8:58 AM 9:23 AM	25	50	12.0	5.0	7.0	3.6

Soil Criteria: Sandy

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	10:16 AM 10:26 AM	10	10	12.0	5.9	6.1	1.6
2	10:26 AM 10:36 AM	10	20	12.0	6.6	5.4	1.9
3	10:36 AM 10:46 AM	10	30	12.0	6.4	5.6	1.8
4	10:46 AM 10:56 AM	10	40	12.0	6.0	6.0	1.7
5	10:56 AM 11:06 AM	10	50	12.0	6.0	6.0	1.7
6	11:06 AM 11:16 AM	10	60	12.0	6.5	5.5	1.8

Infiltration Rate (in/hr):	5.9
Radius of test hole (in):	4
Average Head (in):	9.2

Figure A-18

PERCOLATION TEST REPORT

Project Name:	Coronado Condos	**Project No.:**	T2974-22-01
Test Hole No.:	P-5	**Date Excavated:**	4/19/2022
Length of Test Pipe:	84.0 inches	**Soil Classification:**	ML
Height of Pipe above Ground:	0.0 inches	**Presoak Date:**	4/19/2022
Depth of Test Hole:	84.0 inches	**Perc Test Date:**	4/20/2022
Check for Sandy Soil Criteria Tested by:	Weidman	**Percolation Tested by:**	Weidman

Water level measured from BOTTOM of hole

Sandy Soil Criteria Test

Trial No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in)	Final Water Level (in)	∆ in Water Level (in)	Percolation Rate (min/inch)
1	8:33 AM 8:58 AM	25	25	24.0	0.0	24.0	1.0
2	8:58 AM 9:23 AM	25	50	24.0	0.0	24.0	1.0

Soil Criteria: Sandy

Percolation Test

Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	∆ in Water Level (in)	Percolation Rate (min/inch)
1	11:40 AM 11:50 AM	10	10	24.0	17.4	6.6	1.5
2	11:50 AM 12:00 PM	10	20	24.0	14.2	9.8	1.0
3	12:00 PM 12:10 PM	10	30	24.0	14.4	9.6	1.0
4	12:10 PM 12:20 PM	10	40	24.0	14.2	9.8	1.0
5	12:20 PM 12:30 PM	10	50	24.0	14.5	9.5	1.1
6	12:30 PM 12:40 PM	10	60	24.0	14.4	9.6	1.0

Infiltration Rate (in/hr):	5.4
Radius of test hole (in):	4
Average Head (in):	19.2

Figure A-19

PERCOLATION TEST REPORT

Project Name:		Coronado Condos			**Project No.:**		T2974-22-01
Test Hole No.:		P-6			**Date Excavated:**		4/19/2022
Length of Test Pipe:			96.0	inches	**Soil Classification:**		ML
Height of Pipe above Ground:			0.0	inches	**Presoak Date:**		4/19/2022
Depth of Test Hole:			96.0	inches	**Perc Test Date:**		4/20/2022
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	24.0	0.0	24.0	1.0
2	9:00 AM 9:25 AM	25	50	24.0	5.6	18.4	1.4

Soil Criteria: Sandy

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	11:42 AM 11:52 AM	10	10	24.0	15.6	8.4	1.2
2	11:52 AM 12:02 PM	10	20	24.0	16.0	8.0	1.2
3	12:02 PM 12:12 PM	10	30	24.0	15.6	8.4	1.2
4	12:12 PM 12:22 PM	10	40	24.0	15.6	8.4	1.2
5	12:22 PM 12:32 PM	10	50	24.0	16.3	7.7	1.3
6	12:32 PM 12:42 PM	10	60	24.0	16.6	7.4	1.3

Infiltration Rate (in/hr):	4.0	
Radius of test hole (in):	4	**Figure A-20**
Average Head (in):	20.3	

APPENDIX B

APPENDIX B

LABORATORY TESTING

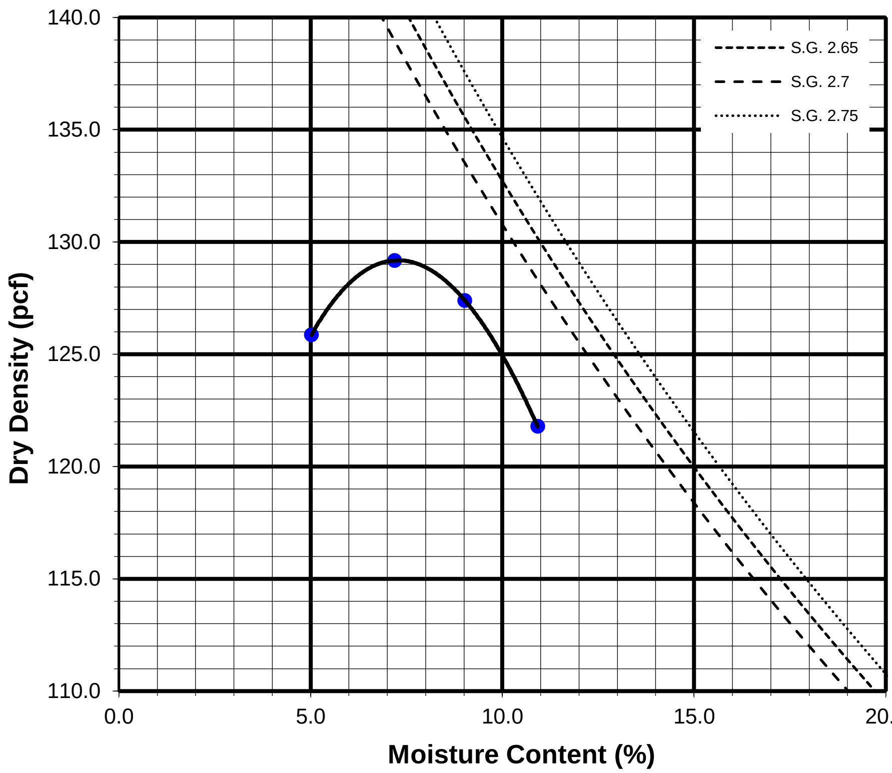
We performed laboratory tests in accordance with current, generally accepted test methods of ASTM International (ASTM) or other suggested procedures. We analyzed selected soil samples for maximum dry density and optimum moisture content, expansion potential, corrosion, grain size distribution, and direct shear. The results of the laboratory tests are presented on Figures B-1 through B-12. The in-place moisture content of the samples tested are presented on the boring logs in *Appendix A.*

Sample No:

TP-1@0-5	Silty SAND (SM), yellowish brown

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6358	6364	6306	6263		
Weight of Mold	(g)	4266	4266	4266	4266		
Net Weight of Soil	(g)	2092	2098	2041	1997		
Wet Weight of Soil + Cont.	(g)	716.8	691.9	721.9	721.7		730.5
Dry Weight of Soil + Cont.	(g)	685.9	655.9	676.2	699.6		725.4
Weight of Container	(g)	256.2	256.9	257.7	259.5		257.4
Moisture Content	(%)	7.2	9.0	10.9	5.0		1.1
Wet Density	(pcf)	138.5	138.9	135.1	132.2		
Dry Density	(pcf)	129.2	127.4	121.8	125.9		0.0

Maximum Dry Density (pcf)	**130.0**	**Optimum Moisture Content (%)**	**8.0**



Preparation Method: A

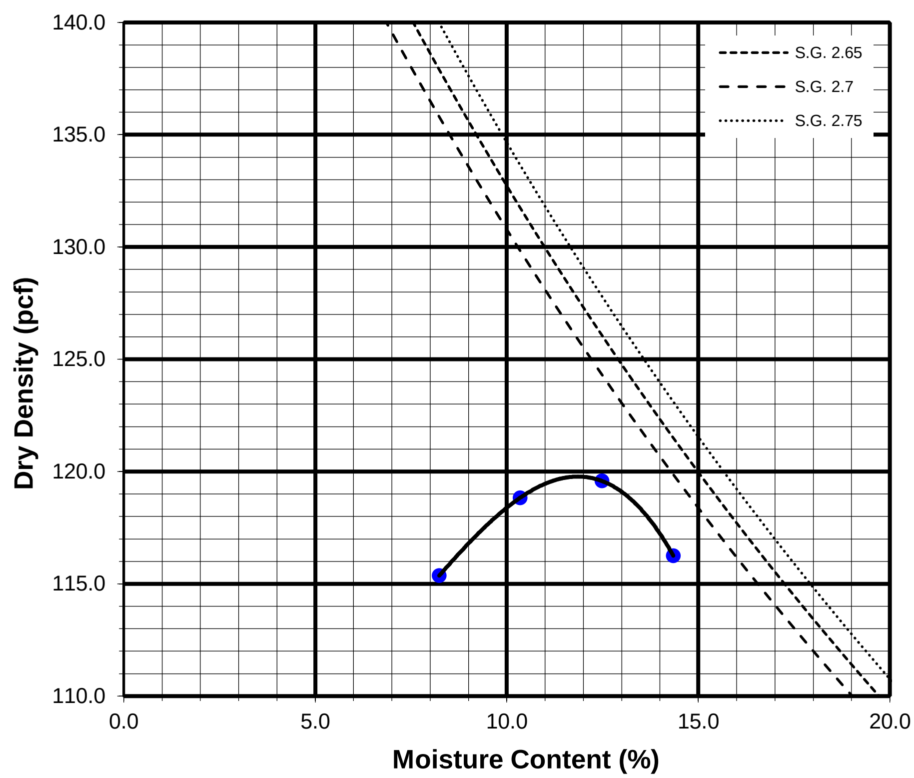
GEOCON	**COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS** ASTM D-1557	Project No.: T2974-22-01
		Coronado Residential Thornton Avenue Menifee, California
	Checked by:	May 22 Figure B1

Sample No:

TP-2@6-10	Silty SAND (SM), yellowish brown

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6298	6274	6247	6152		
Weight of Mold	(g)	4266	4266	4266	4266		
Net Weight of Soil	(g)	2032	2008	1981	1886		
Wet Weight of Soil + Cont.	(g)	712.6	717.4	720.2	712.1		723.4
Dry Weight of Soil + Cont.	(g)	662.1	659.5	676.8	677.6		686.9
Weight of Container	(g)	257.5	255.9	257.4	258.7		259.3
Moisture Content	(%)	12.5	14.3	10.3	8.2		8.5
Wet Density	(pcf)	134.5	132.9	131.1	124.9		
Dry Density	(pcf)	119.6	116.3	118.8	115.4		0.0

Maximum Dry Density (pcf)	**120.0**	**Optimum Moisture Content (%)**	**11.5**



Preparation Method: A

GEOCON	**COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS**	Project No.: T2974-22-01
	ASTM D-1557	Coronado Residential Thornton Avenue Menifee, California
	Checked by:	May 22 — Figure B2

TP-6@0-5'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	611.5	629.5
Wt. of Mold	(gm)	194.9	194.9
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	557.6	629.5
Dry Wt. of Soil + Cont.	(gm)	534.1	384.0
Wt. of Container	(gm)	257.6	194.9
Moisture Content	(%)	8.5	13.2
Wet Density	(pcf)	125.7	130.9
Dry Density	(pcf)	115.8	115.7
Void Ratio		0.5	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	64.8	64.6
Degree of Saturation	(%) [S_{meas}]	50.8	78.3

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
5/3/2022	10:00	1.0	0	0.3581
5/3/2022	10:10	1.0	10	0.3579
Add Distilled Water to the Specimen				
5/4/2022	10:00	1.0	1430	0.3572
5/4/2022	11:00	1.0	1490	0.3572

Expansion Index (EI meas) =	-0.7
Expansion Index (Report) =	**0**

Expansion Index, EI_{50}	CBC CLASSIFICATION *	UBC CLASSIFICATION **
0-20	Non-Expansive	Very Low
21-50	Expansive	Low
51-90	Expansive	Medium
91-130	Expansive	High
>130	Expansive	Very High

* Reference: 2019 California Building Code, Section 1803.5.3
** Reference: 1997 Uniform Building Code, Table 18-I-B.

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EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by:

Project No.: T2974-22-01

Coronado Residential
Thornton Avenue
Menifee, California

May 22

Figure B3

SUMMARY OF LABORATORY
POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
AASHTO T289 ASTM D4972 and AASHTO T288 ASTM G187

Sample No.	pH	Resistivity (ohm centimeters)
TP-4@0-5	7.7	5400

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
AASHTO T291 ASTM C1218

Sample No.	Chloride Ion Content (%)
TP-4@0-5	0.004

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
AASHTO T290 ASTM C1580

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure
TP-4@0-5	0.000	S0

GEOCON

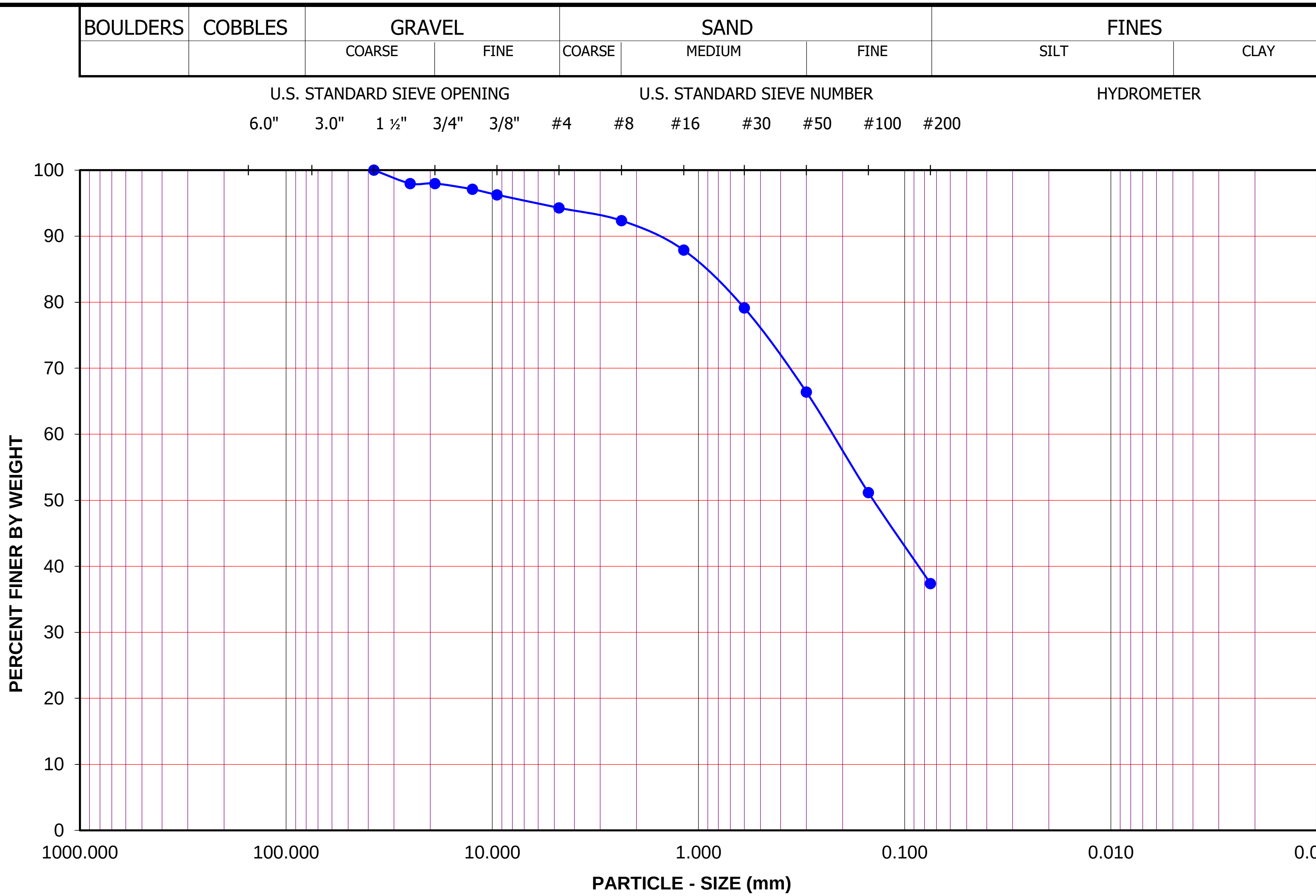
CORROSIVITY TEST RESULTS

Checked by:

Project No.: T2974-22-01

Coronado Residential
Thornton Avenue
Menifee, California

May 22



GEOCON WEST, INC.

PARTICLE - SIZE DISTRIBUTION

ASTM C-136

Checked by:

Project No.: T2974-22-01

Coronado Residential
Thornton Avenue
Menifee, California

May-22 | Figure B5

Exploration No.: P-1

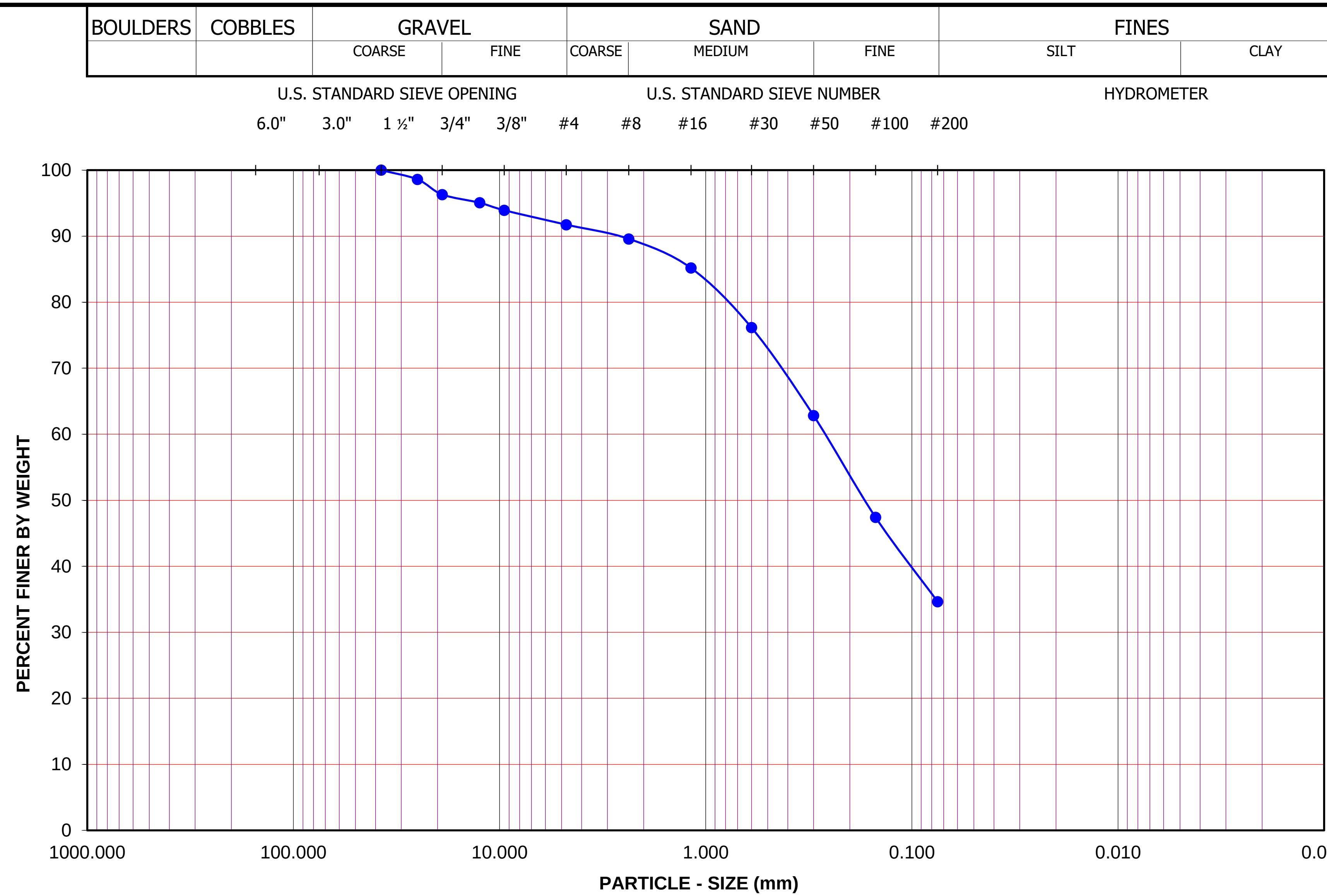
Sample No.: P-1@4

Depth (feet): 4

Soil Type: SM

Soil Identification: Silty SAND (SM), yellowish brown

GR:SA:FI : (%) 6 : 57 : 37



Exploration No.:	P-2
Sample No.:	P-2@4
Depth (feet):	4
Soil Type:	SM
Soil Identification:	Silty SAND (SM), yellowish brown

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PARTICLE - SIZE DISTRIBUTION

ASTM C-136

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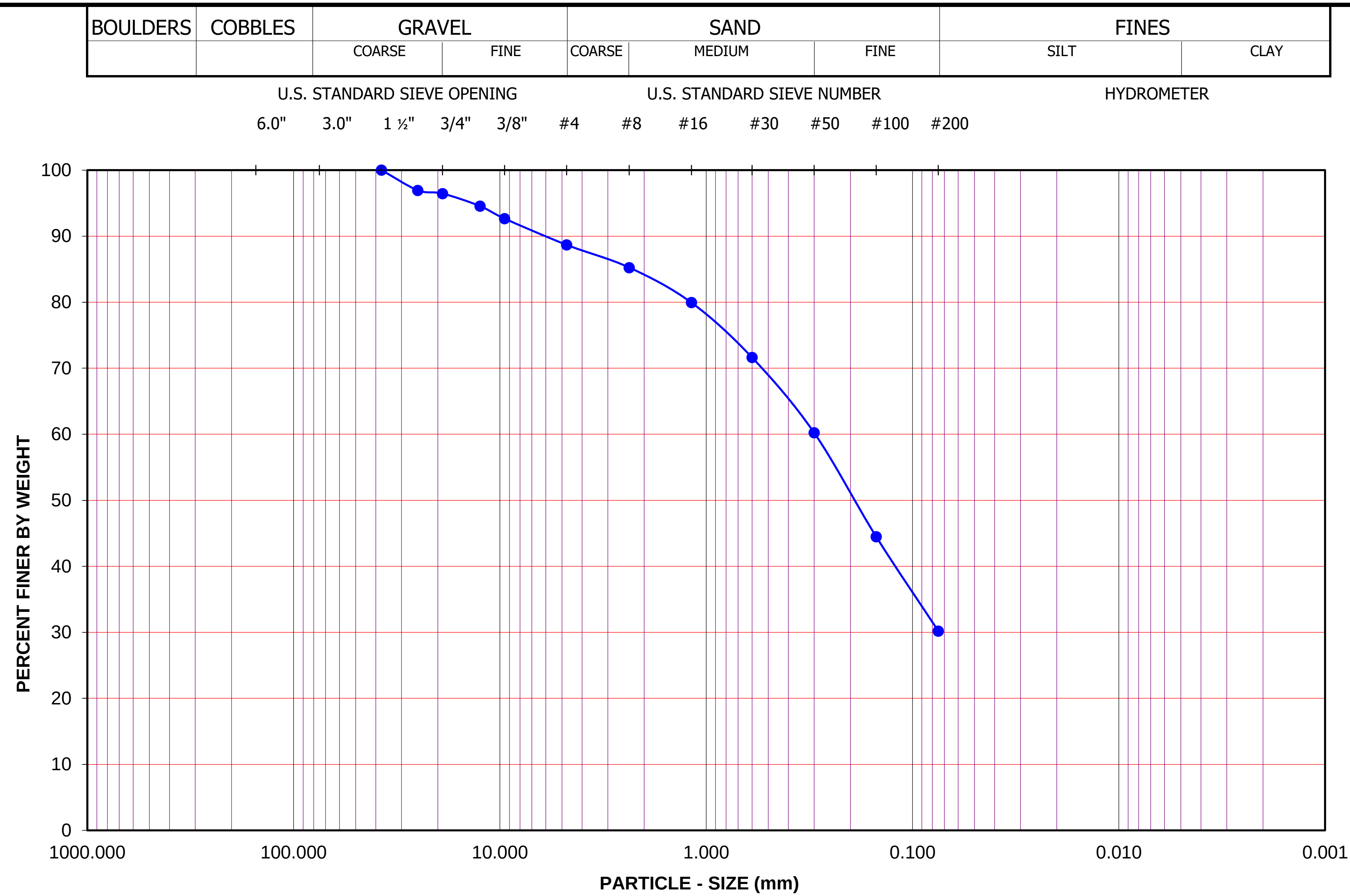
Project No.: T2974-22-01

Coronado Residential
Thornton Avenue
Menifee, California

May-22 Figure B6

GR:SA:FI : (%) 8 : 57 : 35



Exploration No.:	P-3
Sample No.:	P-3@7
Depth (feet):	7
Soil Type:	SM
Soil Identification:	Silty SAND (SM), yellowish brown

GR:SA:FI : (%) 11 : 58 : 30

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PARTICLE - SIZE DISTRIBUTION

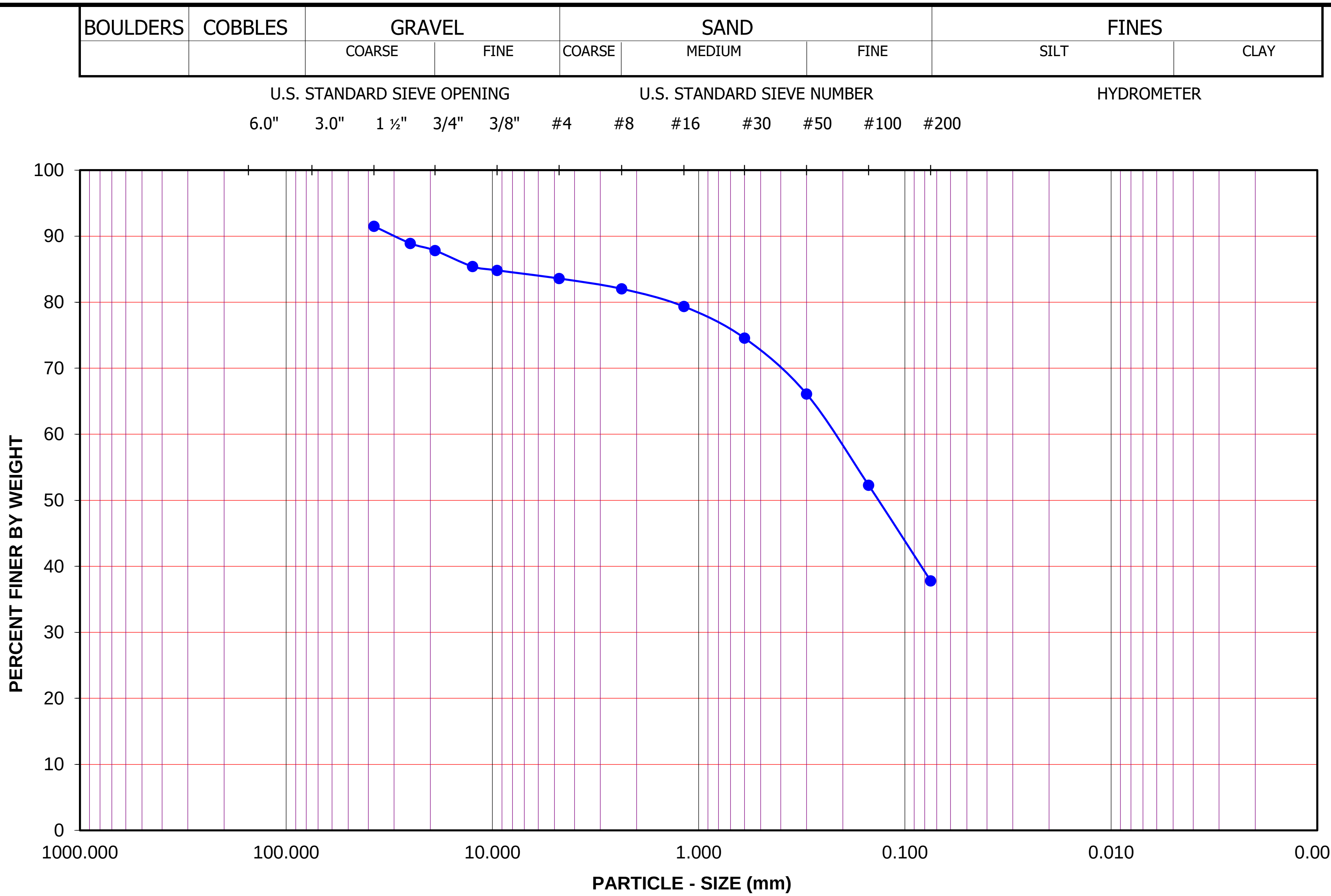
ASTM C-136

Checked by:

Project No.: T2974-22-01

Coronado Residential
Thornton Avenue
Menifee, California

May-22 Figure B7



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PARTICLE - SIZE DISTRIBUTION

ASTM C-136

Checked by:

Project No.: T2974-22-01

Coronado Residential
Thornton Avenue
Menifee, California

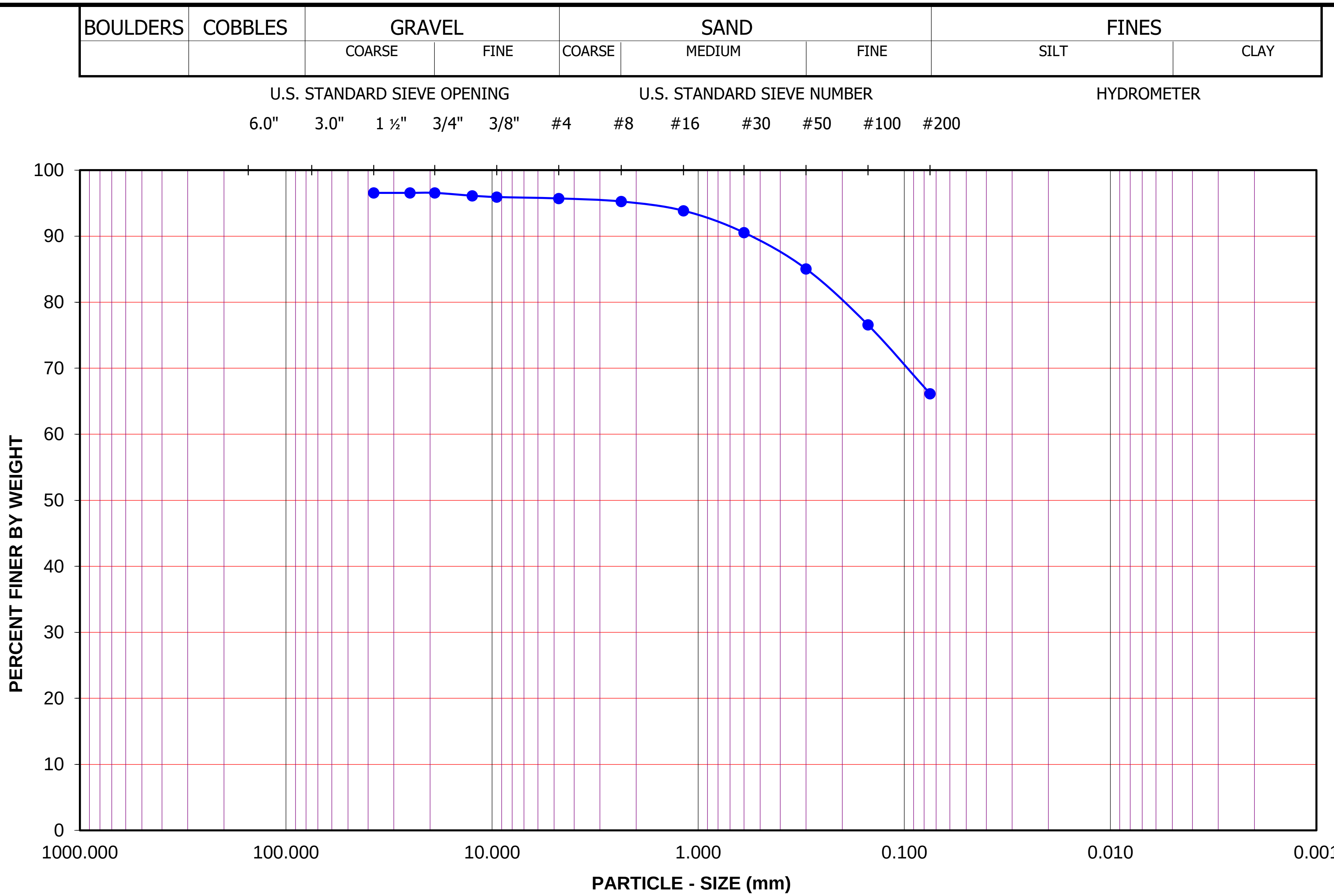
May-22 Figure B8

Exploration No.: P-4 Sample No.: P-4@7

Depth (feet): 7 Soil Type: SMg

Soil Identification: Silty SAND with Gravel (SM)g, yellowish brown

GR:SA:FI : (%) 16 : 46 : 38



Exploration No.:	P-5
Sample No.:	P-5@8
Depth (feet):	8
Soil Type:	ML
Soil Identification:	SILT with Sand (ML), brown

GEOCON WEST, INC.

PARTICLE - SIZE DISTRIBUTION

ASTM C-136

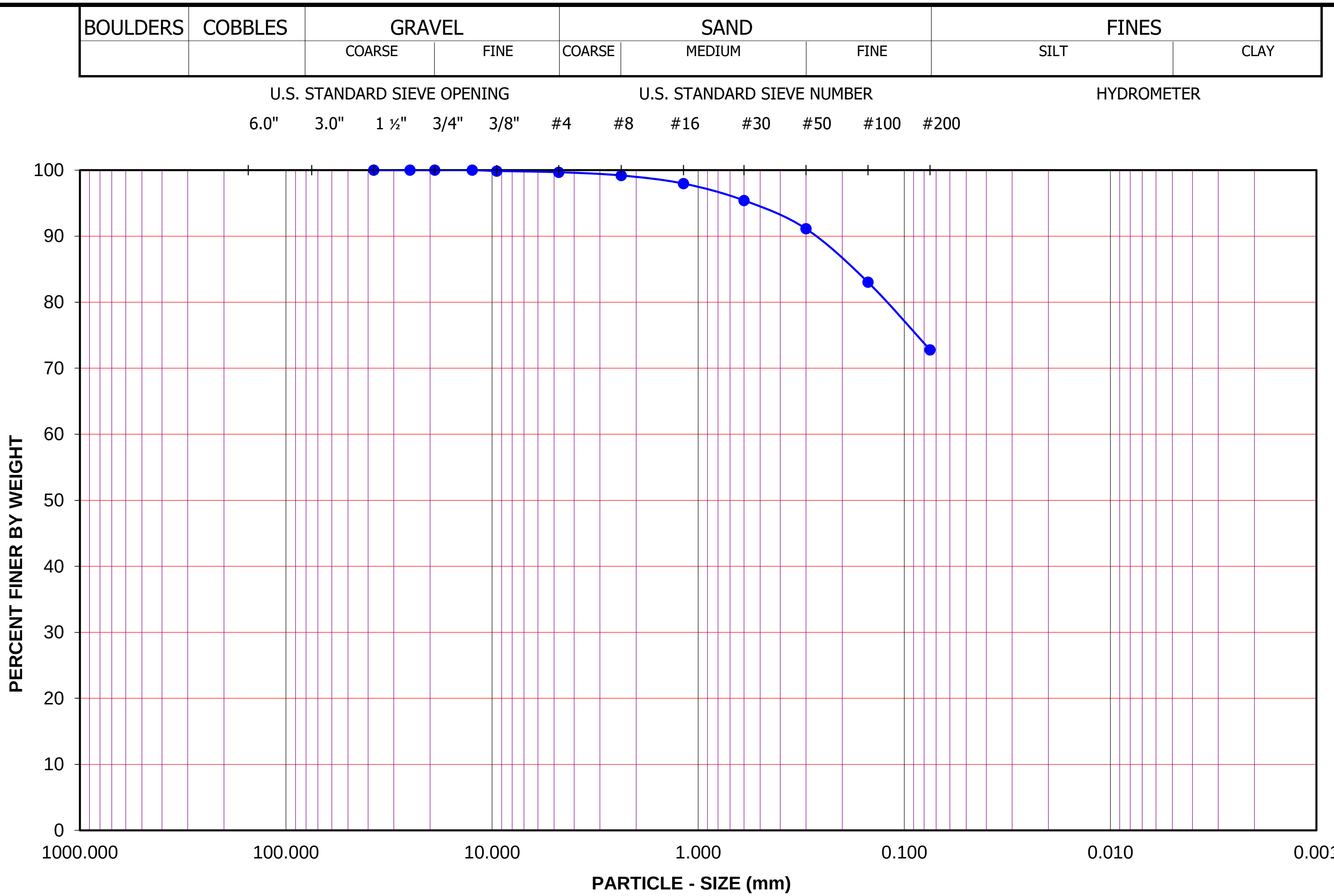
Checked by:

Project No.: T2974-22-01

Coronado Residential
Thornton Avenue
Menifee, California

May-22 Figure B9

GR:SA:FI : (%) 4 : 30 : 66



Exploration No.:	P-6	Sample No.:	P-6@8
Depth (feet):	8	Soil Type:	ML
Soil Identification:	SILT with Sand (ML), brown		

GEOCON WEST, INC.

PARTICLE - SIZE DISTRIBUTION

ASTM C-136

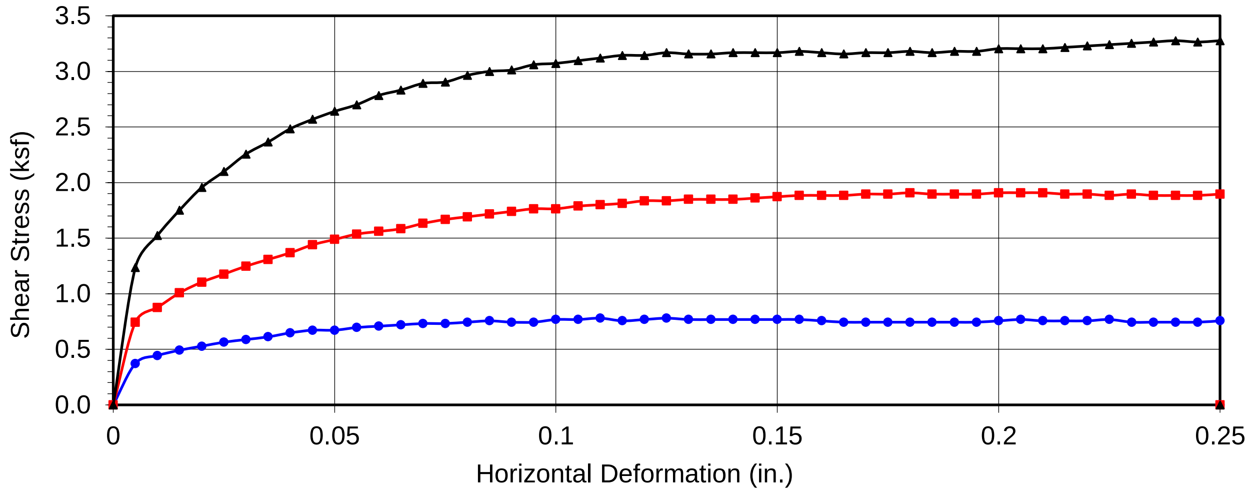
Checked by:

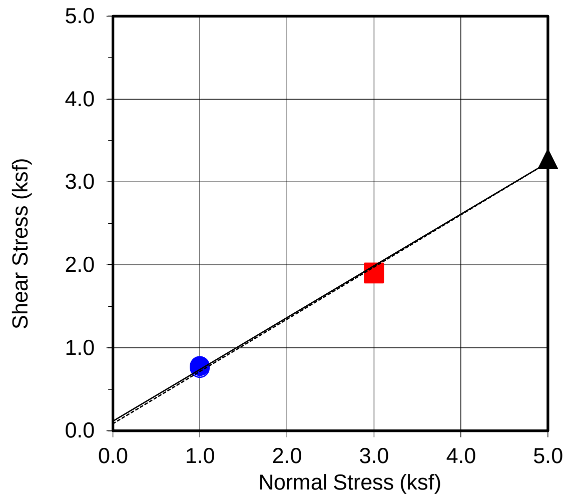
Project No.: T2974-22-01

Coronado Residential
Thornton Avenue
Menifee, California

May-22 Figure B10

GR:SA:FI : (%) 0 : 27 : 73





Boring No.	**TP-1**
Sample No.	**TP-1@0-5**
Depth (ft)	**Bulk**
Sample Type:	0-5

Soil Identification:		
Silty SAND (SM), yellowish brown		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	116	32.0
Ultimate	86	32.2

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.78	■ 1.91	▲ 3.28
Shear Stress @ End of Test (ksf)	○ 0.76	□ 1.90	△ 3.28
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.1	8.1	8.1
Initial Dry Density (pcf)	117.0	117.0	117.0
Initial Degree of Saturation (%)	49.4	49.5	49.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	11.9	10.9	12.5

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

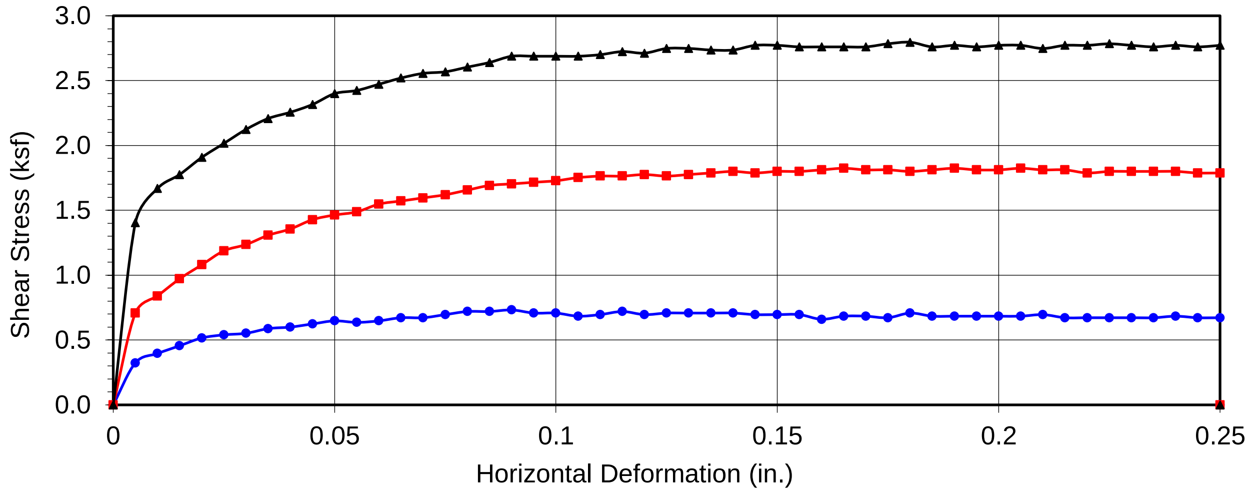
Checked by:

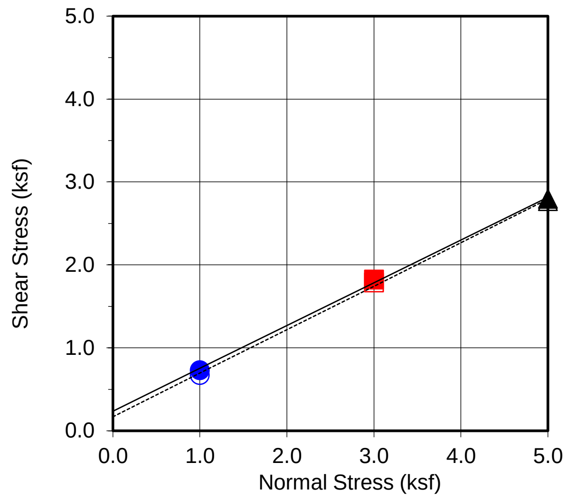
Project No.: T2974-22-01

Coronado Residential
Thornton Avenue
Menifee, California

May 22

Figure B11





Boring No.	**TP-2**
Sample No.	**TP-2@6-10**
Depth (ft)	**Bulk**
Sample Type:	6-10

Soil Identification:		
Silty SAND (SM), yellowish brown		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	236	27.3
Ultimate	169	27.7

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.73	■ 1.82	▲ 2.80
Shear Stress @ End of Test (ksf)	○ 0.67	□ 1.79	△ 2.77
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	11.4	11.5	11.6
Initial Dry Density (pcf)	107.9	108.0	108.1
Initial Degree of Saturation (%)	54.8	55.1	56.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.0	18.1	16.0

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by:

Project No.: T2974-22-01

Coronado Residential
Thornton Avenue
Menifee, California

May 22

Figure B12

APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

CORONADO RESIDENTIAL
THORNTON AVENUE
MENIFEE, CALIFORNIA

PROJECT NO. T2974-22-01

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

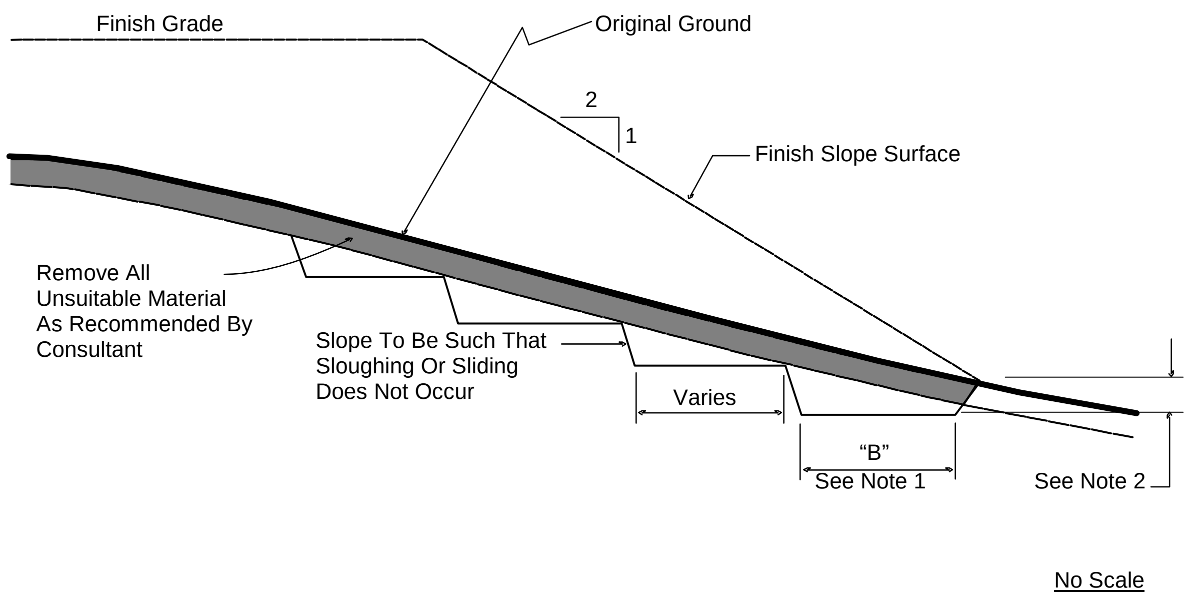
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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



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.

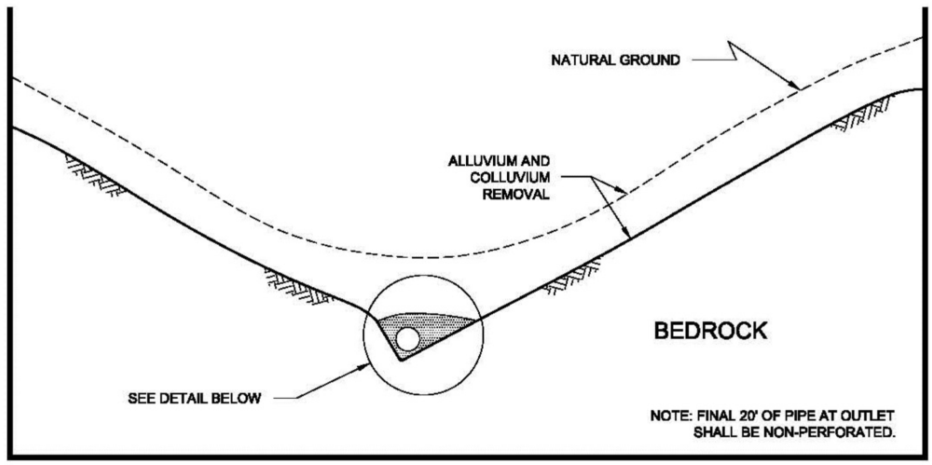
6.3.6 To reduce the potential for “piping” of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

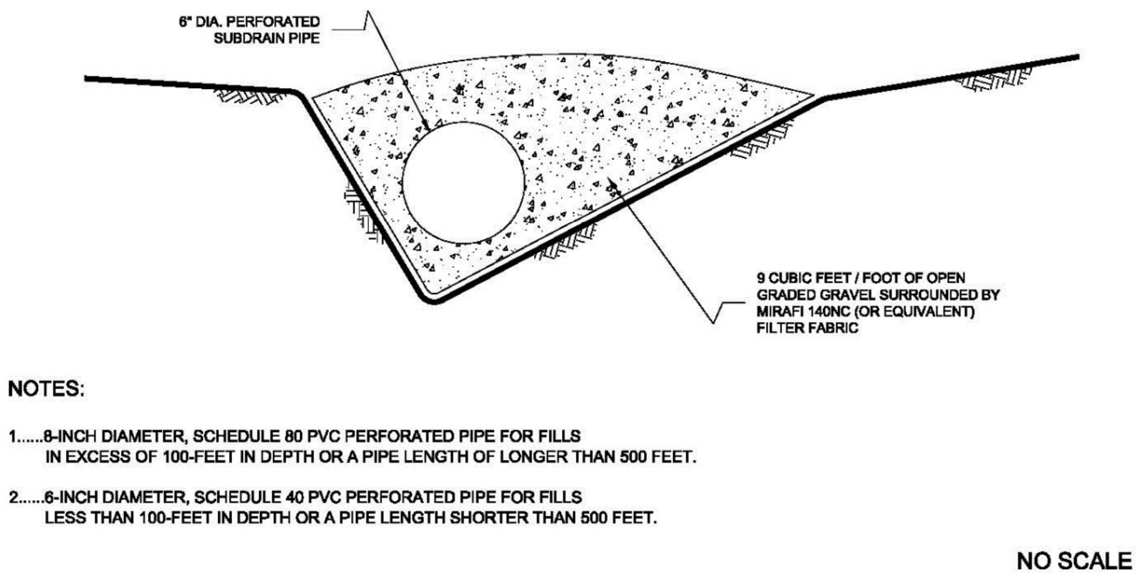
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

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.

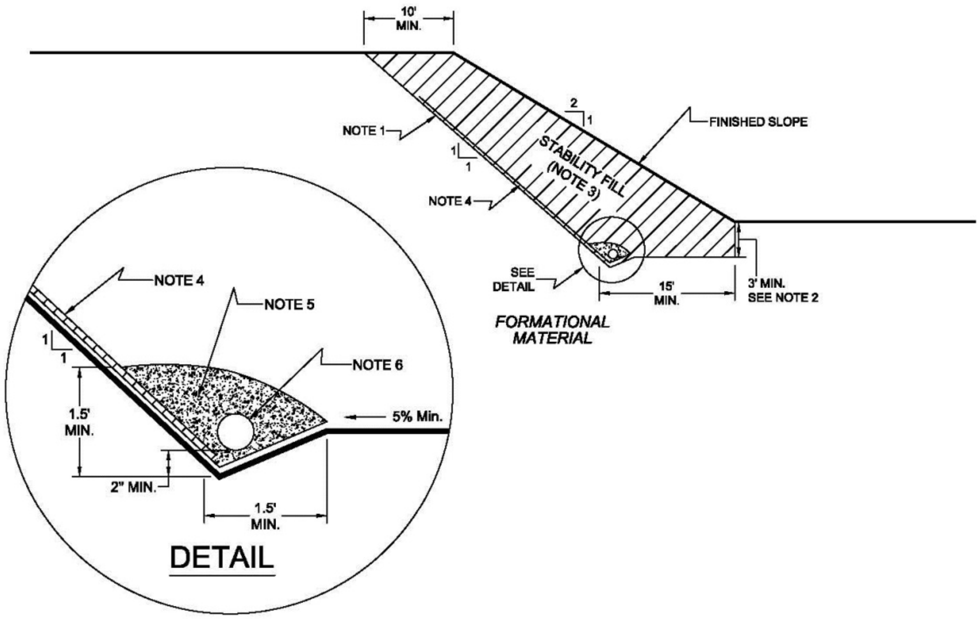
TYPICAL CANYON DRAIN DETAIL





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

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

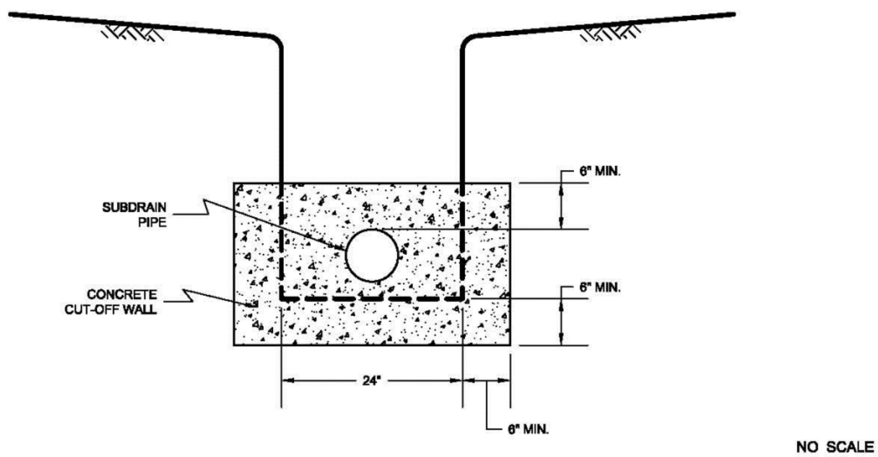
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

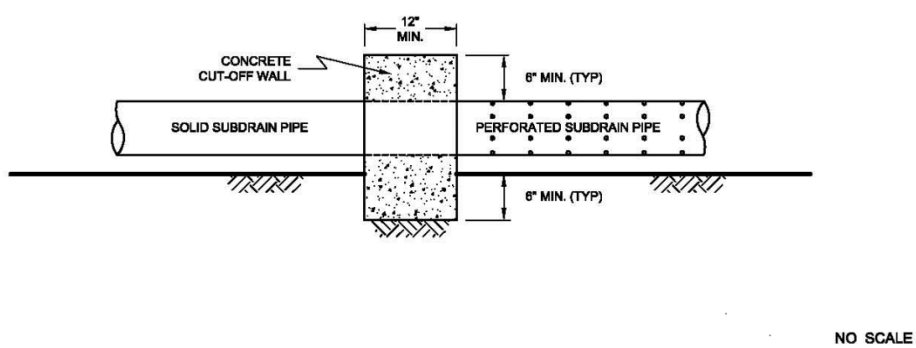
7.5 Prior to outletting, the final 20-foot segment of a subdrain that will not be extended during future development should consist of non-perforated drainpipe. At the non-perforated/ perforated interface, a seepage cutoff wall should be constructed on the downslope side of the pipe.

TYPICAL CUT OFF WALL DETAIL

FRONT VIEW

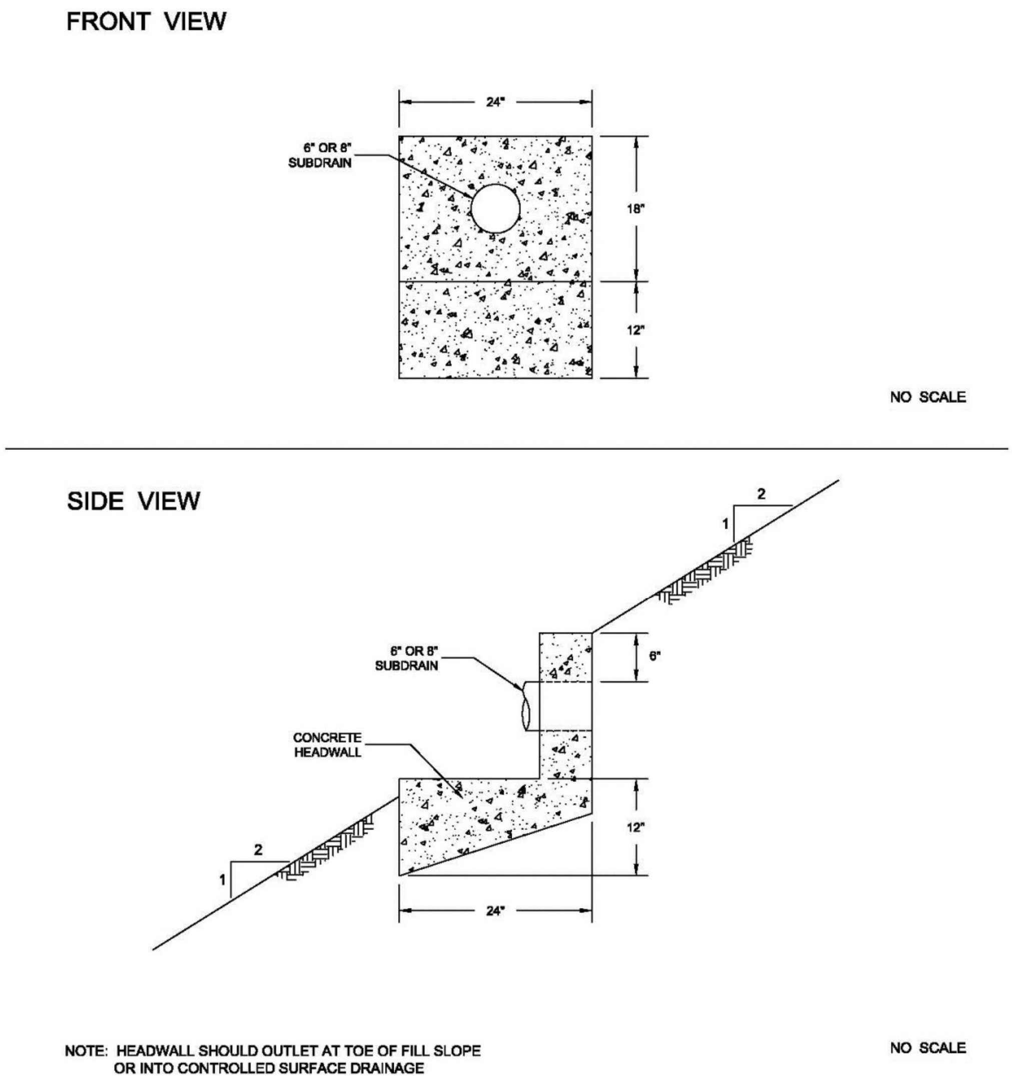


SIDE VIEW



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

TYPICAL HEADWALL DETAIL



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

8. OBSERVATION AND TESTING

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

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

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

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

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

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

8.6.1 Soil and Soil-Rock Fills:

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

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

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

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

9. PROTECTION OF WORK

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

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

10. CERTIFICATIONS AND FINAL REPORTS

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

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