

Appendix 5

**Preliminary Geotechnical Investigation for Proposed Cherry
Outpost**

PRELIMINARY GEOTECHNICAL INVESTIGATION

PROPOSED CHERRY OUTPOST ASSESSOR'S PARCELS 366-290-007 & -008 NWC OF BUNDY CANYON AND CHERRY STREET WILDOMAR, CALIFORNIA



GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

CHERRY OUTPOST, LP
CORONA, CALIFORNIA

JANUARY 6, 2023
PROJECT NO. T3010-22-01



Project No. T3010-22-01
January 6, 2023

Cherry Outpost, LP.
139 Radio Road
Corona, California 92879

Attention: Ms. Claudia Grajeda, Project Manager

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED CHERRY OUTPOST
ASSESSOR'S PARCELS 366-290-007 & -008
NWC OF BUNDY CANYON AND CHERRY STREET
WILDOMAR, CALIFORNIA

Dear Ms. Grajeda:

In accordance with your authorization of Geocon's Proposal IE-4018 dated November 8, 2022, Geocon West, Inc. (Geocon) herein submits the results of our preliminary geotechnical investigation for the proposed development. The accompanying report presents the results of our study, conclusions, and recommendations pertaining to the geotechnical aspects of the proposed development. The site is considered suitable for the proposed development provided the recommendations of this report are followed.

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

Very truly yours,

GEOCON WEST, INC.

A handwritten signature in green ink that reads "Alireza Kazem".

Alireza Kazem
EIT 160387



Neal D. Berliner
GE 2576

A handwritten signature in green ink that reads "Lisa A. Battiato".

Lisa A. Battiato
CEG 2316



ARK:LB:NDB:hd

Distribution: Addressee (email)

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PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of our preliminary geotechnical investigation for the planning and design of Cherry Outpost planned for 6.65 acres located northwest of the intersection of Bundy Canyon Road and Cherry Street in Wildomar, California (see *Vicinity Map*, Figure 1). The purpose of the investigation is to evaluate subsurface soil and geologic conditions at the site, and based on the conditions encountered, provide preliminary recommendations pertaining to the geotechnical aspects of the proposed construction.

The scope of our investigation included review of published geotechnical information, site reconnaissance, Underground Service Alert mark out and notification, private utility location, drilling and logging of geotechnical and percolation borings, collection of soil samples, percolation testing, laboratory testing, engineering analyses, and preparation of this report. A summary of the information and documentation reviewed for this study is presented in the *List of References*.

Our field investigation was performed on November 30 and December 1, 2022 by drilling of eight geotechnical borings (B-1 through B-8) and four percolation borings (P-1 through P-4) with a CME-75 truck mounted drill rig, in accessible areas, to depths ranging between approximately 21 and 26 feet (geotechnical borings) and 5 feet (percolation borings) below existing ground surface. The borings were drilled to observe the subsurface geotechnical conditions and collect soil samples for laboratory testing. The percolation testing was performed in the lowest elevation areas, within the southern area of the property and in accordance with Riverside County Flood Control and Water Conservation District's (RCFC) *Low Impact Development Best Management Practices Handbook*. The approximate locations of the exploratory and percolation borings are depicted on the *Geologic Map* (Figure 2).

A detailed discussion of the field investigation, logs of the geotechnical and percolation borings, and the percolation test results are presented in *Appendix A*. Laboratory tests were performed on select soil samples obtained to evaluate pertinent physical and chemical soil properties for use in engineering analysis. *Appendix B* presents a summary of the laboratory test results.

2. SITE AND PROJECT DESCRIPTION

The roughly triangular site is located immediately northwest of Bundy Canyon Road and Cherry Street and encompasses APNs 366-290-007 and -008. The site was previously occupied by a single-family residence and a palm tree nursery. The residence was demolished at the end of 2013; however, young palms are still located within the northern half of the site. The site is bounded by Cherry Street to the east, Bundy Canyon to the south, Interstate 15 to the west, and rural residences to the north. The latitude and longitude of the site are 33.6280 degrees and -117.2717 degrees, respectively.

The elevation of the site varies from 1,406 feet above mean sea level (MSL) in the south to the 1,437 feet MSL in the northeast corner. Drainage is directed to a south flowing valley along the east side of the site.

Based the *Conceptual Site Plan* prepared by KTGy, dated September 30, 2022, the site development will include a 4-story hotel, convenience store, gas station, drive through restaurant, and carwash. Approximately 70 feet of the site's southern area will be utilized to widen Bundy Canyon Road. We also expect Cherry Street will be improved on the east side of the site. Finished grade elevations are not known at the time of this report; however, based on the current site elevations and surrounding grades we expect maximum proposed cuts and fills to be 10 feet or less. Based on our understanding of the general plan for the site, we do not expect slopes to be constructed; however, retaining walls up to 5 feet high may be required.

We expect the structures will be lightly loaded two- to four-story buildings with spread foundations and slab-on-grade floors. Although structural plans and loading information is unavailable at this time, we expect column loads will not exceed 200 kips and wall loads will not exceed 4 kips per linear foot. Preliminary geotechnical recommendations for design of the structure are based on these assumptions and provided herein. If actual plans differ, supplementary geotechnical recommendations may be necessary.

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

3. GEOLOGIC SETTING

The site is located along the northeastern margin of the Elsinore Trough within the Peninsular Ranges Geomorphic Province. The Peninsular Ranges are bounded on the north by the Transverse Ranges and the Cucamonga/Sierra Madre faults, the east by the San Jacinto fault, the west by the Elsinore fault and the Santa Ana Mountains. The Peninsular Ranges extend southward into Mexico. They are characterized by granitic highlands of low to moderate relief surrounded by alluvial plains and valleys. Locally, the Elsinore Trough is the dominant geomorphic feature of the area and is a graben that formed as a result of a left step over from the Wildomar to the Glen Ivy North branches of the Elsinore fault. Locally, the Glen Ivy North fault is geologically mapped one quarter mile southwest of the site. No faulting is mapped within or projecting on to the site. Faulting is discussed in detail in the *Surface Fault Rupture* section, below.

4. GEOLOGIC MATERIALS

4.1 General

The primary geologic units at the site consist of young alluvial fan deposits and Pauba Sandstone. Localized areas of undocumented fill are present near the previous residence, the residential driveway, and within the occasional end dump piles. Topsoil was encountered up to a depth of approximately 2½ feet within B-3 and is expected throughout the palm tree area. The geologic nomenclature follows that of Morton, D.M. and Weber, F.H. (2003). The soil and geologic units encountered at the site are summarized below and described on the boring logs in *Appendix A*.

4.2 Undocumented Artificial Fill (afu)

Undocumented artificial fill is present at the vicinity of the previous residence and within the driveway to the residence. Potential utility conflicts prevented excavation of a borings in these areas and the condition and extent of the fill is not known at this time. However, we expect the fill was derived from local sources and is similar to the young alluvial fan deposits and Pauba sandstone described below.

4.3 Young Alluvial Fan Deposits (Qyf)

Young alluvial fan deposits were encountered within the percolation test borings and within geotechnical borings B-1, B-2, and B-4 to depths between approximately 2½ and 6 feet below existing ground surface. It consists of silty sand that can be characterized as loose to medium dense, dry to moist, and strong brown to dark brown.

4.4 Pauba Formation (Qps)

Pleistocene-age Pauba Sandstone was observed near the surface and below the younger alluvial fan deposits to the maximum depth explored of approximately 26 feet. It is described herein as a soil because, although it is considered bedrock, it behaves like a soil and its engineering properties are closer to a soil than a rock. The Pauba, as encountered, can be characterized as silty sand which is dense to very dense, dry to moist, and shades of brown, orange-brown, and grayish brown.

5. GROUNDWATER

Groundwater was not encountered in the borings excavated for this study to depths of 26 feet. Well data from the California Department of Water Resources was available for 2019. Well 336251N1172930W001 located west of the site reported groundwater depths of between 357 and 389 feet below existing ground surface. Well 336200N1172734W001 located south of the site reported a groundwater depth of 44 feet below existing ground surface. Groundwater and seepage are dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. It is not uncommon for seepage conditions to develop where none previously existed. Proper surface drainage will be important to future performance of the planned improvements.

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in southern California include active, potentially active, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS), formerly known as California Division of Mines and Geology (CDMG), for the Alquist-Priolo Earthquake Fault Zone Program (Byrant and Hart, 2007). By definition, an active fault is one that has had surface displacement within Holocene time (about the last 11,700 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 Glen Ivy North branch of the Elsinore fault zone located approximately 1/4 mile southwest. Faults within a 50-mile radius of the site are listed in Table 6.1. Historic earthquakes in southern California of magnitude 6.0 and greater, their magnitude, distance, and direction from the site are listed in Table 6.2.

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

Fault Name	Maximum Magnitude (Mw)	Geometry (Slip Character)	Slip Rate (mm/yr)	Distance from Site (mi)	Direction from Site
Elsinore Fault (Glen Ivy)	6.8	RL-SS	5.0	1/4	SW
Elsinore (Wildomar)	6.8	RL-SS	5.0	2	S
San Jacinto (Casa Loma)	6.9	RL-SS	12.0	20	NE
San Jacinto (Claremont)	6.7	RL-SS	12.0	23	NE
Chino	6.7	RL-R-O	1.0	25	NW
Whittier	6.8	RL-R-O	2.5	30	NW
San Andreas (San Bernardino)	7.5	RL-SS	24.0	35	NE
San Gorgonio Pass	n/a	THRUST	n/a	35	NE
San Jacinto (Clark)	7.2	RL-SS	12.0	35	E
Cucamonga	6.9	R	5.0	40	NW
San Jacinto (Glen Helen)	6.7	RL-SS	12.0	40	N
Pinto Mountain	7.2	LL-SS	2.5	45	NE
San Jacinto (Coyote Creek)	6.8	RL-SS	4.0	45	SE
Morongo Valley	7.2	LL-SS	2.5	47	NE
S Branch San Andreas Fault	7.5	RL-SS	24.0	48	NE
N Branch San Andreas Fault	7.4	RL-SS	30.0	50	NE

6.2 Seismicity

As with all southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. A number of earthquakes of moderate to major magnitude have occurred in the southern California area within the last 100 years. A partial list of these earthquakes is included in the following table.

**TABLE 6.2
LIST OF HISTORIC EARTHQUAKES**

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Near Redlands	July 23, 1923	6.3	26	N
Long Beach	March 10, 1933	6.4	40	W
Tehachapi	July 21, 1952	7.5	137	NW
San Fernando	February 9, 1971	6.6	84	NW
Whittier Narrows	October 1, 1987	5.9	55	WNW
Sierra Madre	June 28, 1991	5.8	60	NW
Landers	June 28, 1992	7.3	62	NE
Big Bear	June 28, 1992	6.4	47	NE
Northridge	January 17, 1994	6.7	83	WNW
Hector Mine	October 16, 1999	7.1	88	NE
Ridgecrest China Lake Fault	July 5, 2019	7.1	149	N

6.3 Liquefaction

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

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

According to the *Map My County* GIS system (RCIT, 2022), the site is located within an area mapped as having a moderate potential for liquefaction. However, due to the presence of Pauba Sandstone at or near the surface, liquefaction is not considered a design consideration for this site.

6.4 Expansive Soil

The on-site surficial soils generally consist of silty sands. Laboratory test results on a select sample indicate that the expansion potential of site soils is 19 which is considered “very low” per ASTM D4829.

6.5 Landslides

We did not observe evidence of previous or incipient slope instability within the site or adjacent hillsides during our site investigation. Further, no landslides have been geologically mapped on or adjacent to the site (Morton & Weber, 2003). Therefore, landslide hazard to the site is not a design consideration.

6.6 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 more than 24 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 located almost 3.5 miles from the shore of Lake Elsinore and is up slope from the lake, therefore, a seiche hazard from Lake Elsinore is not a design consideration.

6.7 Slope Stability

A grading plan was not available at the time of this report. In general, it is our opinion that permanent, graded fill slopes constructed of on-site soils and free of any surcharge with gradients of 2:1 (horizontal to vertical) or flatter and vertical heights of 15 feet or less will possess adequate factors of safety for global and surficial stability in static conditions. However, evaluation of the stability of proposed slopes, if any, may need to be performed once detailed grading plans depicting proposed slopes are available.

7. PERCOLATION TESTING

Percolation testing was conducted in accordance with the procedures in the *Riverside County Flood Control and Water Conservation District LID BMP, Appendix A* (Handbook), at the proposed location of the proposed site BMPs. The percolation test locations are depicted on the *Geologic Map* (see Figure 2).

Four percolation test holes were excavated to depths of 5 feet (P-1 through P-4) below the existing ground surface using an 8-inch diameter auger. The geologic conditions at the percolation test locations consist of Alluvium (Silty Sand) over Pauba Sandstone. The deep geotechnical boring (B-1) did not encounter groundwater or impenetrable bedrock within 10 feet of the bottom of the percolation test depths. Approximately two inches of gravel was placed at the bottom of each test hole and a perforated

pipe was placed at top the gravel to keep the test hole open. Gravel was placed around the bottom of the test hole to support the test pipe. The test locations were pre-saturated prior to testing. Infiltration test results are included as Figures A-13 through A-16. Results of the converted percolation test rates to infiltration test rates are presented in Table 7.0 below. The *Handbook* requires a factor of safety of 3 be applied to the values below based on the test method used.

**TABLE 7.0
INFILTRATION TEST RATES**

Parameter	P-1	P-2	P-3	P-4
Depth (inches)	60	60	60	60
Test Type	Normal	Normal	Normal	Normal
Change in head over time: ΔH (inches)	2.5	3	5.5	5
Average head: H_{avg} (in)	39	40.3	37.5	38
Time Interval (minutes): Δt (minutes)	30	30	30	30
Radius of test hole: r (inches)	4	4	4	4
Tested Infiltration Rate: I_t (inches/hour)	0.2	0.3	0.6	0.5

The results of the infiltration testing indicate that infiltration at the locations tested ranges from 0.2 to 0.6 inches per hour. These rates are the calculated infiltration rates from the tested percolation rates. The *Handbook* requires a factor of safety of 3 be applied to the values above based on the test method used.

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

8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

- 8.1.1 Soil or geologic conditions were not encountered during the investigation that would preclude the construction of the proposed development, provided the recommendations presented herein are followed and implemented during design and construction.
- 8.1.2 Potential geologic hazards at the site include seismic shaking, and compressible near surface soils.
- 8.1.3 The existing undocumented artificial fill, topsoil, young alluvial fan deposits, and upper few feet of Pauba Sandstone are not suitable for the support of engineered fill or settlement-sensitive improvements. The earth materials are suitable for re-use as engineered fill provided the recommendations in this geotechnical investigation report are followed. Remedial grading of the uppermost earth materials will be required as discussed herein.
- 8.1.4 The *in-situ* moisture content of upper site soils is below optimum moisture content. Therefore, moisture conditioning site soils to be used as fill material throughout the grading and backfill processes will be required.
- 8.1.5 Proper surface drainage should be maintained to prevent ponding and saturation of the fill in pad areas. Recommendations for site drainage are provided herein.
- 8.1.6 An evaluation of seismically induced “dry-sand” settlement during earthquake indicates that the existing young alluvial fan deposits could be prone to minor settlement during a high-magnitude earthquake; however, the recommended site grading will minimize or eliminate this risk since these soils will be properly compacted.
- 8.1.7 Once the design and civil grading plans are made available, the recommendations within this report should be reviewed and revised, as necessary. Additionally, as the project design progresses toward a final design, changes in the design, location, or elevation of the proposed improvements should be reviewed by this office. Geocon should be contacted to evaluate the necessity for review and possible revision of this report.

8.2 Soil Characteristics

8.2.1 Based on the earth materials encountered in the field, the soils are expected to be “non-expansive” (Expansion Index [EI] 20 or less) as defined by 2019 California Building Code (CBC) Section 1803.5.3. The existing soil possesses a “very low” expansion potential (expansion index of 19) in accordance with ASTM D 4829. Table 8.2.1 presents soil classifications based on the expansion index.

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

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

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

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

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

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

8.2.4 Laboratory testing has been performed on two representative bulk samples of soil obtained in the upper five feet in Borings B-3 and B-6. Boring locations are depicted in Figure 2 and results are presented in the following table.

**TABLE 8.2.4
LABORATORY RESULTS**

Sample	Depth (ft)	Resistivity (OHM CM)	Chloride (PPM)	Sulfate (PPM)	pH
B-3	0-5	1900	121	1	9.3
B-6	0-5	4400	75	0	8.8

8.2.5 Test results for B-3 and B-6 show Resistivity of 1,900 and 4,400 ohm-cm, pH of 9.3 and 8.8, chloride content of 121 and 75 ppm. The site soils are not classified as corrosive to metal improvements in accordance with Caltrans Corrosion Guidelines, provided in Table 8.2.5 below. (Caltrans, 2021).

**TABLE 8.2.5
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

8.2.6 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.

8.3 Consolidation and Settlement

8.3.1 Undocumented artificial fill, topsoil, alluvium, and the upper few feet of Pauba Sandstone at the site are subject to consolidation settlement (densification by the removal of water within the soil) under loads imposed by placement of fill or structure loads. Appendix B presents the laboratory consolidation test results.

8.4 Hydrocollapse

8.4.1 The Hydrocollapse is the tendency of unsaturated soil structure to collapse upon saturation resulting in the overall settlement of the effected soil and overlying foundations or improvements supported thereon. Recommended removals to Pauba Sandstone and proper compaction of the upper younger, compressible surficial earth materials will minimize the potential for settlement due to hydrocollapse.

8.5 Grading

- 8.5.1 Grading should be performed in accordance with the *Recommended Grading Specifications of Appendix C* and the grading ordinances of the City of Wildomar.
- 8.5.2 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.
- 8.5.3 Site preparation should begin with the removal of existing improvements, deleterious material, debris and vegetation. The depth of removal should be such that material exposed in cut areas or soil to be used as fill is relatively free of organic matter. Material generated during stripping and/or site demolition should be exported from the site.
- 8.5.4 The undocumented artificial fill, topsoil, alluvium and upper few feet of Pauba Sandstone within a 1:1 (h:v) projection of the limits of grading should be removed to expose competent Pauba Sandstone bedrock. Areas of loose, dry, or compressible soils will require deeper excavation and processing prior to fill placement. The approximate remedial removal depths are depicted on the *Geologic Map*, Figure 2. The excavation bottoms should expose dense Pauba Sandstone. The engineering geologist should evaluate the actual depth of removal during grading operations. Where over-excavation and compaction is to be conducted, the excavations should be extended laterally a minimum distance of 5 feet beyond the foundation footprint or for a distance equal to the depth of removal plus additional fill, whichever is greater. Patios and building appurtenances should be considered as part of the building footprint when determining the limits of lateral excavation. Grading should be performed under the full-time observation and testing of Geocon.
- 8.5.5 Proposed spread foundations for each building may derive support in either Pauba Sandstone, or engineered fill, but not a combination of both. Foundations should maintain a minimum depth of 18 inches below the ground surface. Foundations deriving support in Pauba Sandstone should be deepened as necessary to maintain a minimum embedment of 12-inches into the competent Pauba Sandstone. Foundations deriving support in newly placed engineered fill should be underlain by a minimum of 3 feet of engineered fill.
- 8.5.6 Proposed building slabs-on-grade may derive support directly in newly placed engineered fill, approved Pauba Sandstone, or a combination of both if exposed at the building pad elevation.

- 8.5.7 The site should be brought to finish grade elevations with engineered fill compacted in layers. Layers of fill should be no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density at or slightly above optimum moisture content (as determined by ASTM D1557). Fill materials placed below optimum moisture content may require additional moisture conditioning prior to placing additional fill.
- 8.5.8 The fill placed within 4 feet of proposed finish grade should possess a “very low” to “low” expansion potential (EI of 51 or less), where practical.
- 8.5.9 Import fill (if necessary) should consist of granular materials with a “very low” expansion potential (EI of 20 or less), generally free of deleterious material and rock fragments larger than 6 inches and should be compacted as recommended herein. Geocon should be notified of the import soil source and should perform laboratory testing of import soil prior to its arrival at the site to evaluate its suitability as fill material.

8.6 Utility Trench Backfill

- 8.6.1 Utility trenches should be properly backfilled in accordance with the requirements of the City of Wildomar and the latest edition of the *Standard Specifications for Public Works Construction* (Greenbook). The pipes should be bedded with well-graded crushed rock or clean sand (Sand Equivalent greater than 30) to a depth of at least one foot over the pipe. If open graded rock is used it should be wrapped in filter fabric to prevent finer soils from migrating into the rock voids. The remainder of the trench backfill may be derived from onsite soil or approved import soil. Backfill of utility trenches should not contain rocks greater than 3 inches in diameter. The use of 2-sack slurry and controlled low strength material (CLSM) are also acceptable as backfill. However, consideration should be given to the possibility of differential settlement where the slurry ends and earthen backfill begins. These transitions should be minimized, and additional stabilization should be considered at these transitions.
- 8.6.2 Utility trench backfill should be placed in layers no thicker than will allow for adequate bonding and compaction. Utility backfill should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density and moisture conditioned at or slightly above optimum moisture content (as determined by ASTM D1557). Backfill at the finish subgrade elevation of new pavements should be compacted to at least 95 percent of the maximum dry density. Backfill materials placed below the recommended moisture content may require additional moisture conditioning prior to placing additional fill.

8.7 Seismic Design Criteria

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

**TABLE 8.7.1
2019 CBC SEISMIC DESIGN PARAMETERS**

Parameter	Value	2019 CBC Reference
Site Class	C	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.669g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.618g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.2	Table 1613.2.3(1)
Site Coefficient, F_V	1.4	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	2.012g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.866g	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.341g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.578	Section 1613.2.4 (Eqn 16-39)

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

**TABLE 8.7.2
ASCE 7-16 PEAK GROUND ACCELERATION**

Parameter	Value	ASCE 7-10 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.725g	Figure 22-7
Site Coefficient, F_{PGA}	1.2	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.87g	Section 11.8.3 (Eqn 11.8-1)

- 8.7.3 The Maximum Considered Earthquake Ground Motion (MCE) is the level of ground motion that has a 2 percent chance of exceedance in 50 years, with a statistical return period of 2,475 years. According to the 2019 California Building Code and ASCE 7-16, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building Code is to maintain “Life Safety” during an MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.
- 8.7.4 Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition. The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 7.71 magnitude event occurring at a hypo central distance of 2.37 kilometers from the site.
- 8.7.5 Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.8 magnitude occurring at a hypocentral distance of 12.58 kilometers from the site.
- 8.7.6 Conformance to the criteria for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

8.8 Foundation and Concrete Slabs-On-Grade Recommendations

- 8.8.1 The foundation recommendations presented herein are intended for all structures after completion of the recommended grading. Spread foundations are recommended for support of proposed site structures and improvements.
- 8.8.2 The foundation recommendations have been separated into two categories based on either the maximum and differential fill thickness or Expansion Index, as provided in Table 8.8.2. Structures will be Category I; however, the category may be increased to II where maximum fill dictates. Final foundation categories will be evaluated once site grading has been completed.

**TABLE 8.8.2
FOUNDATION CATEGORY CRITERIA**

Foundation Category	Maximum Fill Thickness, T (Feet)	Differential Fill Thickness, D (Feet)	Expansion Index (EI)
I	$T < 20$	$D < 10$	$EI \leq 50$
II	$20 \leq T < 50$	$10 \leq D < 20$	$50 < EI \leq 90$

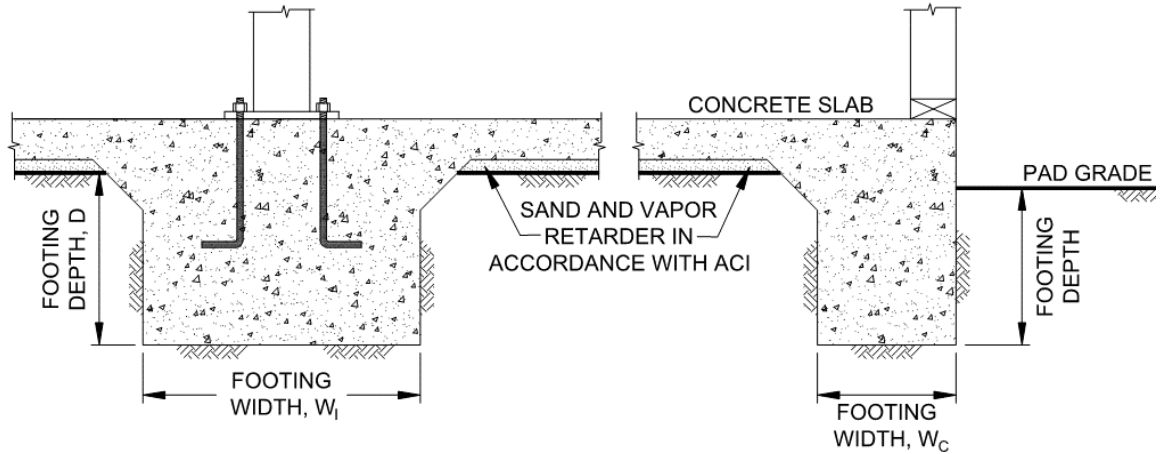
8.8.3 Proposed spread foundations for each building may derive support in either Pauba Sandstone, or engineered fill, but not a combination of both. Foundations should maintain a minimum depth of 18 inches below the ground surface (24 inches for Category II). Foundations deriving support in Pauba Sandstone should be deepened as necessary to maintain a minimum embedment of 18-inches into the Pauba Sandstone. Foundations deriving support in newly placed engineered fill should be underlain by a minimum of 3 feet of engineered fill.

8.8.4 Table 8.8.4 provides a summary of the foundation design recommendations.

**TABLE 8.8.4
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	3,500 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	250 psf per Foot of Width
Maximum Allowable Bearing Capacity	4,500 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 30 Feet (One-Story Buildings)
Footing Size Used for Settlement	6-Foot Square
Design Expansion Index	20 or less

8.8.5 The foundations should be embedded in accordance with the recommendations herein and the Wall/Column Footing Dimension Detail. The embedment depths should be measured from the lowest adjacent pad grade for both interior and exterior footings. If there is any slope designed 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 as discussed herein).



Wall/Column Footing Dimension Detail

- 8.8.6 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.
- 8.8.7 We should observe the foundation excavations prior to the placement of reinforcing steel and concrete to verify 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.
- 8.8.8 Proposed building slabs-on-grade may derive support directly in newly placed engineered fill, approved Pauba Sandstone, or a combination of both if exposed at the building pad elevation.
- 8.8.9 Table 8.8.9 presents minimum foundation and interior concrete slab design criteria for conventional shallow foundation systems.

**TABLE 8.8.9
CONVENTIONAL FOUNDATION RECOMMENDATIONS BY CATEGORY**

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

- 8.8.10 The concrete slab-on-grade should be a minimum of 4 inches thick for Foundation Category I, and 5 inches thick for Foundation Category II.
- 8.8.11 Slabs-on-grade that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06) and should be installed in general conformance with ASTM E1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. The vapor retarder should be puncture resistant and durable, so as to avoid perforations during its installation.
- 8.8.12 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. However, we should be contacted to provide recommendations if the bedding sand is thicker than 4 inches. Placement of 3 inches and 4 inches of sand is common practice in southern California. The foundation engineer should provide appropriate concrete mix design criteria and curing measures that may be utilized to assure proper curing of the slab to reduce the potential for rapid moisture loss and subsequent cracking and/or slab curl.
- 8.8.13 As an alternative to the conventional foundation recommendations, consideration should be given to the use of post-tensioned concrete slab and foundation systems for the support of the proposed structures. The post-tensioned systems should be designed by a structural engineer experienced in post-tensioned slab design and design criteria of the Post-Tensioning Institute (PTI) DC 10.5-12 *Standard Requirements for Design and Analysis of Shallow Post-Tensioned Concrete Foundations on Expansive Soils* or *WRI/CRSI Design of Slab-on-Ground Foundations*, as required by the 2019 California Building Code (CBC Section 1808.6.2). Although this procedure was developed for expansive soil conditions, it can also be used to reduce the potential for foundation distress due to differential fill settlement. The post-tensioned design should incorporate the geotechnical parameters presented in Table 8.8.9 for the Foundation Category designated. The parameters presented in Table 8.8.13 are based on the guidelines presented in the PTI DC 10.5 design manual.

**TABLE 8.8.13
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS**

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

8.8.14 The foundations for the post-tensioned slabs should be embedded in accordance with the recommendations of the structural engineer. If a post-tensioned mat foundation system is planned, the slab should possess a thickened edge with a minimum width of 18 inches and extend below the clean sand or crushed rock layer.

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

- The deflection criteria presented in Table 8.8.9 are still applicable.
- Interior stiffener beams should be used for Foundation Category II.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 18 inches, and 24 inches for foundation categories I, and II, respectively. The embedment depths should be measured from the lowest adjacent pad grade.

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

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

- 8.8.18 Foundations may be designed for an allowable soil bearing pressure of 3,500 pounds per square foot (psf) (dead plus live load) for the site. This bearing pressure may be increased by one-third for transient loads due to wind or seismic forces.
- 8.8.19 The maximum expected static settlement for the planned structures, supported on conventional foundation systems with the above allowable bearing pressures, and deriving support in engineered fill is estimated to be 1 inch and to occur below the heaviest loaded structural element. Settlement of the foundation system is expected to occur on initial application of loading. Differential settlement due to foundation loads is not expected to exceed ½ inch over a horizontal distance of 30 feet.
- 8.8.20 Isolated footings outside of the slab area, if present, should have the minimum embedment depth and width recommended for conventional foundations for a Foundation Category. In addition, consideration should be given to connecting patio slabs that exceed 5 feet in width to the building foundation, to reduce the potential for future separation to occur.
- 8.8.21 Interior stiffening beams should be incorporated into the design of the foundation system in accordance with the PTI design procedures.
- 8.8.22 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisture conditioned at or slightly above optimum moisture content.
- 8.8.23 The recommendations of this report are intended to reduce the potential for cracking of slabs and foundations. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade placed on such conditions may still exhibit some cracking due to soil movement and/or shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.
- 8.8.24 Concrete slabs should be provided with adequate crack-control joints, construction joints and/or expansion joints to reduce unsightly shrinkage cracking. The design of joints should consider criteria of the American Concrete Institute when establishing crack-control spacing. Additional reinforcing steel, concrete admixtures and/or closer crack control joint spacing should be considered where concrete-exposed finished floors are planned.
- 8.8.25 Geocon should be consulted to provide additional design parameters as required by the structural engineer.

8.9 Exterior Concrete Flatwork

- 8.9.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations herein assuming the subgrade materials possess an Expansion Index of 30 or less. Subgrade soils should be compacted to 90 percent relative compaction at or slightly above optimum moisture content. Slab panels should be a minimum of 4 inches thick and when in excess of 8 feet square should be reinforced with No. 3 reinforcing bars spaced 24 inches center-to-center in both directions to reduce the potential for cracking. In addition, concrete flatwork should be provided with crack control joints to reduce and/or control shrinkage cracking. Crack control spacing should be determined by the project structural engineer based upon the slab thickness and intended usage. Criteria of the American Concrete Institute (ACI) should be taken into consideration when establishing crack control spacing. Subgrade soil for exterior slabs not subjected to vehicle loads should be compacted in accordance with criteria presented in the *Grading* section prior to concrete placement. Subgrade soil should be properly compacted, and the moisture content of subgrade soil should be verified prior to placing concrete. Base materials will not be required below concrete improvements.
- 8.9.2 The reinforcement steel should overlap continuously in flatwork to reduce the potential for vertical offsets within flatwork. Additionally, flatwork should be structurally connected to the curbs, where possible, to reduce the potential for offsets between the curbs and the flatwork.
- 8.9.3 Where exterior flatwork abuts the structure at entrant or exit points, the exterior slab should be dowelled into the structure's foundation stem wall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer.
- 8.9.4 The recommendations presented herein are intended to reduce the potential for cracking of exterior slabs as a result of differential movement. However, even with the incorporation of the recommendations presented herein, slabs-on-grade will still crack. The occurrence of concrete shrinkage cracks is independent of the soil supporting characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, the use of crack control joints and proper concrete placement and curing. Crack control joints should be spaced at intervals no greater than 12 feet. Literature provided by the Portland Concrete Association (PCA) and American Concrete Institute (ACI) present recommendations for proper concrete mix, construction, and curing practices, and should be incorporated into project construction.

8.10 Conventional Retaining Walls

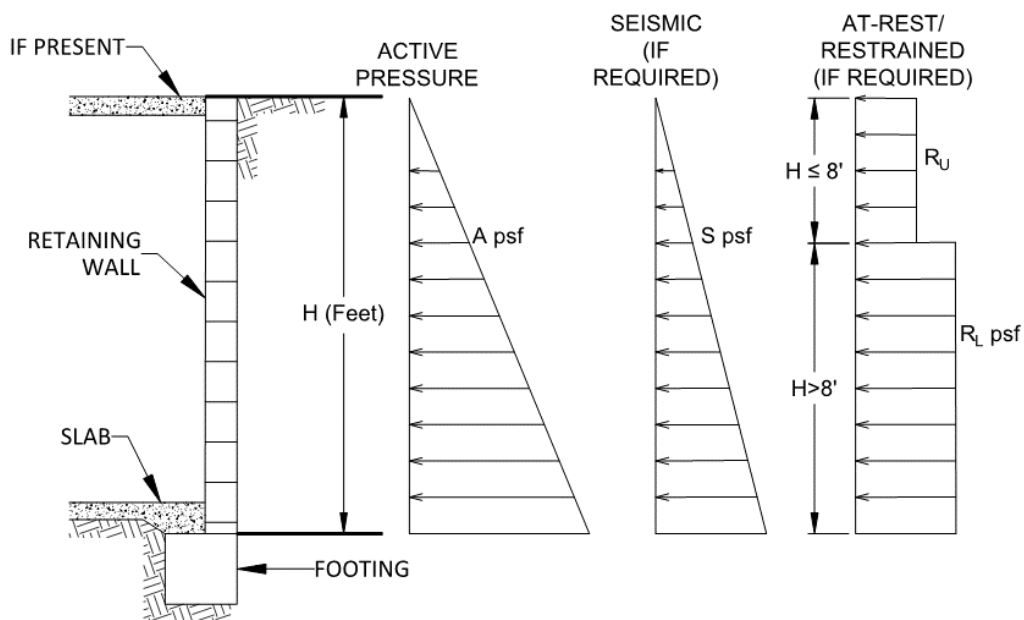
- 8.10.1 Spread foundations are recommended for support of proposed retaining walls. Soil with an expansion index (EI) of greater than 30 should not be used as backfill material behind retaining walls.

**TABLE 8.10.1
RETAINING WALL DESIGN RECOMMENDATIONS**

Parameter	Value
Active Soil Pressure, A (Fluid Density, Level Backfill)	30 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, R_U (0 to 8 Feet High)	7H psf
At-Rest/Restrained Walls Additional Uniform Pressure, R_L (8+ Feet High)	13H psf
Expected Expansion Index for the Subject Property	$EI \leq 30$

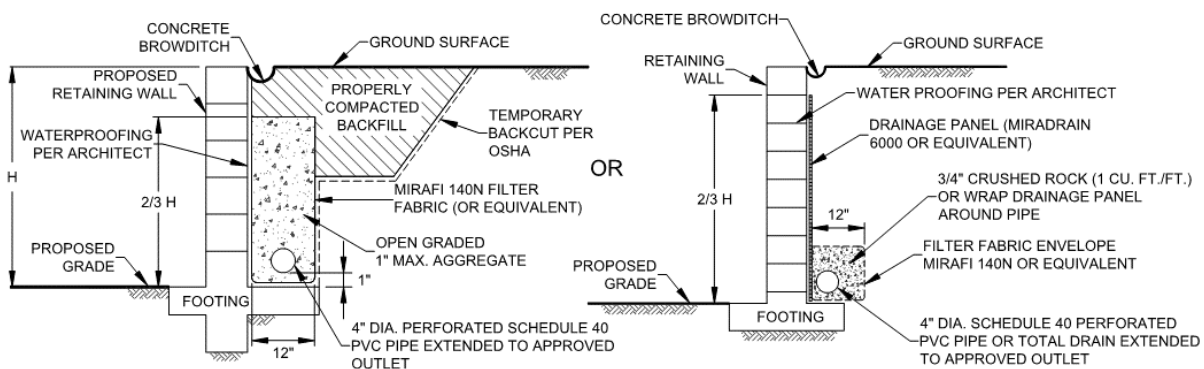
H equals the height of the retaining portion of the wall

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



Retaining Wall Loading Diagram

- 8.10.3 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.
- 8.10.4 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.
- 8.10.5 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.
- 8.10.6 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 90 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

- 8.10.7 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.
- 8.10.8 In general, wall foundations should be designed in accordance with Table 8.10.8. 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 8.10.8
SUMMARY OF RETAINING WALL FOUNDATION RECOMMENDATIONS**

Parameter	Value
Minimum Retaining Wall Foundation Width	12 inches
Minimum Retaining Wall Foundation Depth	18 Inches
Minimum Steel Reinforcement	Per Structural Engineer
Allowable Bearing Capacity	3,500 psf
Bearing Capacity Increase	500 psf per Foot of Depth
	300 psf per Foot of Width
Maximum Allowable Bearing Capacity	4,500 psf
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch in 30 Feet

- 8.10.9 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls. In the event that other types of walls (such as mechanically stabilized earth [MSE] walls, soil nail walls, or soldier pile walls) are planned, Geocon should be consulted for additional recommendations.
- 8.10.10 It is common to see retaining walls constructed in the areas of the elevator pits. The retaining walls should be properly drained and designed in accordance with the recommendations presented herein. If the elevator pit walls are not drained, the walls should be designed with an increased active pressure with an equivalent fluid density of 90 pcf. It is also common to see seepage and water collection within the elevator pit. The pit should be designed and properly waterproofed to prevent seepage and water migration into the elevator pit.

- 8.10.11 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.
- 8.10.12 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.
- 8.10.13 Retaining walls up to 6 feet high and not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 30 pounds per cubic foot (pcf). Where the backfill will be inclined at no steeper than 2:1 (horizontal to vertical), an active soil pressure of 50 pcf is recommended. These soil pressures assume that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall possess an EI of 30 or less. For walls where backfill materials do not conform to the criteria herein, Geocon should be consulted for additional recommendations.
- 8.10.14 Unrestrained walls are those that are allowed to rotate more than $0.001H$ (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls with a level backfill surface should be designed for a soil pressure equivalent to the pressure exerted by a fluid density of 50 pcf.
- 8.10.15 Unrestrained walls will move laterally when backfilled and loading is applied. The amount of lateral deflection is dependent on the wall height, the type of soil used for backfill, and loads acting on the wall. The retaining walls and improvements above the retaining walls should be designed to incorporate an appropriate amount of lateral deflection as determined by the structural engineer.
- 8.10.16 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and waterproofed as required by the project architect. The soil immediately adjacent to the backfilled retaining wall should be composed of free draining material completely wrapped in Mirafi 140N (or equivalent) filter fabric for a lateral

distance of 1 foot for the bottom two-thirds of the height of the retaining wall. The upper one-third should be backfilled with less permeable compacted fill to reduce water infiltration. Alternatively, a drainage panel, such as a Miradrain 6000 or equivalent, can be placed along the back of the wall. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted backfill (EI of 20 or less) with no hydrostatic forces or imposed surcharge load. If conditions different than those described are expected or if specific drainage details are desired, Geocon should be contacted for additional recommendations. A graphic depicting typical retaining wall drainage is provided below.

8.11 Lateral Loading

- 8.11.1 To resist lateral loads, a passive pressure exerted by an equivalent fluid weight of 250 pounds per cubic foot (pcf) should be used for the design of footings or shear keys poured neat against compacted fill or Pauba Sandstone. The allowable passive pressure assumes a horizontal surface extending at least 5 feet or 3 times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by floor slabs or pavement should not be included in design for passive resistance.
- 8.11.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between compacted fill or Pauba Sandstone and concrete of 0.35 should be used for design.

8.12 Preliminary Pavement Recommendations

- 8.12.1 The final pavement design should be based on R-value testing of soils at road subgrade elevation. Roadways should be designed in accordance with the County of Riverside Ordinance No. 461, *Road Improvement Standards & Specifications*, Section 8.07 and Standard Drawing No. 114, *Roadway Design Requirements*, when final Traffic Indices (TI) and R-value test results of subgrade soils are completed.
- 8.12.2 For preliminary design purposes, we used an assumed R-value test result of 30 based on the material observed in the field. A value of 78 was considered for aggregate base materials for the purposes of this preliminary analysis. Pavements should meet the minimum requirement for pavement thickness per County of Riverside Ordinance No. 461, *Road Improvement Standards & Specifications*, Section 8.07 and Standard Drawing No. 114, *Roadway Design Requirements*. Preliminary flexible pavement sections are presented in Table 8.12.2. Geocon should be contacted if other roadway classifications and traffic indices are appropriate for the project.

**TABLE 8.12.2
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS**

Road Classification	Assumed Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Local Street/Access Road	5.5	30	3	7
Enhanced Local Street at School or Park	6.5	30	4	8
Collector	7.0	30	4	10

- 8.12.3 The upper 12 inches of the roadway subgrade soil should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density at or slightly above optimum moisture content (as determined by ASTM D1557).
- 8.12.4 The asphalt concrete materials should conform to Section 203-6, of the latest edition of the California *Greenbook*. Class 2 Aggregate Base should conform to Section 26 of the Caltrans *Standard Specifications*. 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 (as determined by ASTM D1557). Asphalt concrete should be compacted to a density of 95 percent of the laboratory Hveem density (as determined by ASTM D1561).
- 8.12.5 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 8.12.5 provides the traffic categories and design parameters used for the calculations for 20-year design life.

**TABLE 8.12.5
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

8.12.6 We used the parameters presented in Table 8.12.6 to calculate the pavement design sections. We should be contacted to provide updated design sections, if necessary.

**TABLE 8.12.6
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

8.12.7 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 8.12.7.

**TABLE 8.12.7
RIGID VEHICULAR PAVEMENT RECOMMENDATIONS**

Traffic Category	Trucks Per Day	Portland Cement Concrete, T (Inches)
A = Car Parking Areas and Access Lanes	10	6
B = Entrance and Truck Service Lanes	10	6½
B = Entrance and Truck Service Lanes	50	7
B = Entrance and Truck Service Lanes	100	7
C = School or City Buses	50	10
C = School or City Buses	100	10½
D = Heavy Duty Trucks	50	7½
D = Heavy Duty Trucks	100	8½
E = Garbage or Fire Truck Lanes	5	7½
E = Garbage or Fire Truck Lanes	10	7½

8.12.8 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.

8.12.9 Adequate joint spacing should be incorporated into the design and construction of the rigid pavement in accordance with Table 8.12.9.

**TABLE 8.12.9
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

8.12.10 The rigid pavement should also be designed and constructed incorporating the parameters presented in Table 8.12.10.

**TABLE 8.12.10
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	$1/4$ -Inch for Sealed Joints and Per Sealer Manufacturer's Recommendations
	$1/16$ - to $1/4$ -Inch is Common for Unsealed Joints

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

8.12.12 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.

8.12.13 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.

- 8.12.14 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.

8.13 Temporary Excavations

- 8.13.1 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. If there is any improvement designed, 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.
- 8.13.2 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.
- 8.13.3 Where there is insufficient space for sloped excavations, shoring or trench shields should be used to support excavations. Shoring may also be necessary where sloped excavation could remove vertical or lateral support of existing improvements, including existing utilities and adjacent structures. Recommendations for temporary shoring are provided in the following section.
- 8.13.4 Where sloped embankments are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction embankments are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The contractor's competent person should inspect the soils exposed in the cut slopes during excavation in accordance with OSHA regulations so that modifications of the slopes can be made if variations in the soil conditions occur.

8.14 Site Drainage and Moisture Protection

- 8.14.1 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.
- 8.14.2 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.
- 8.14.3 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.
- 8.14.4 Proposed infiltration systems should be offset from the outside edge of planned foundations a minimum lateral distance of 20 feet to reduce the occurrence of water migrating below the load projection of planned structures.
- 8.14.5 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. Downgradient and adjacent structures may be subjected to seeps, movement of foundations and slabs, or other impacts as a result of water infiltration.

8.15 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in this 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.

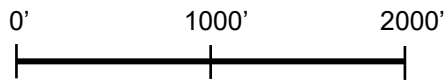
The requirements for concrete and reinforcing steel presented in this report are preliminary recommendations from a geotechnical perspective. The Structural Engineer should provide the final recommendations for structural design of concrete and reinforcing steel for foundation systems, floor slabs, exterior concrete, or other systems where concrete and reinforcing steel are utilized, in accordance with the latest version of applicable codes.

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

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

LIST OF REFERENCES

1. American Concrete Institute, 2019, *Building Code Requirements for Structural Concrete*, Report by ACI Committee 318.
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3. ASCE 7-16, 2019, *Minimum Design Loads for Buildings and Other Structures*.
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6. California Department of Transportation (Caltrans), Division of Engineering Services, Materials Engineering and Testing Services, 2021, *Corrosion Guidelines, Version 3.2*, dated March.
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8. Jennings, C. W. and Bryant, W. A., 2010, *Fault Activity Map of California*, California Geological Survey Geologic Data Map No. 6.
9. Kennedy, Michael P., 1977, *Recency and Character of Faulting Along the Elsinore Fault Zone in Southern Riverside County, California*, California Division of Mines and Geology, Special Report 131.
10. KTGy, 2022, *Conceptual Site Plan*, www.ktgy.com, dated September 30.
11. Legg, M.R., J.C. Borrero, and C.E. Synolakis, 2003, *Evaluation of Tsunami Risk to Southern California Coastal Cities*, EERI, NEHRP Professional Fellowship Report PF2002-11, dated January.
12. Morton, D.M. and Weber, F.H., 2003, *Preliminary Geologic Map of the Elsinore 7.5' Quadrangle, Riverside County, California*, U.S. Geological Survey, Open-File Report OF-2003-281, 1:24,000. BedGIS In View
13. OSHPD Seismic Design Maps Web Application, <https://seismicmaps.org/>, accessed December 2022.
14. Riverside County GIS (RC GIS), *Map My County* website, http://mmc.rivcoit.org/MMC_Public/Custom/disclaimer/Default.htm; accessed December 2022.
15. Riverside County Flood Control and Water Conservation District, 2011, *Design Handbook for Low Impact Development Best Management Practices*, dated September.
16. United States Geological Survey (USGS), *Unified Hazard Tool*, <https://earthquake.usgs.gov/hazards/interactive/>; accessed December 2022.



SOURCE: Google Earth, 2022

SCALE: 1" = 1000'

VICINITY MAP

GEOCON
WEST, INC.



GEOTECHNICAL, ENVIRONMENTAL, MATERIALS
41571 CORNING PLACE #101, MURRIETA, CALIFORNIA 92562
PHONE 951-304-2300 www.geoconinc.com

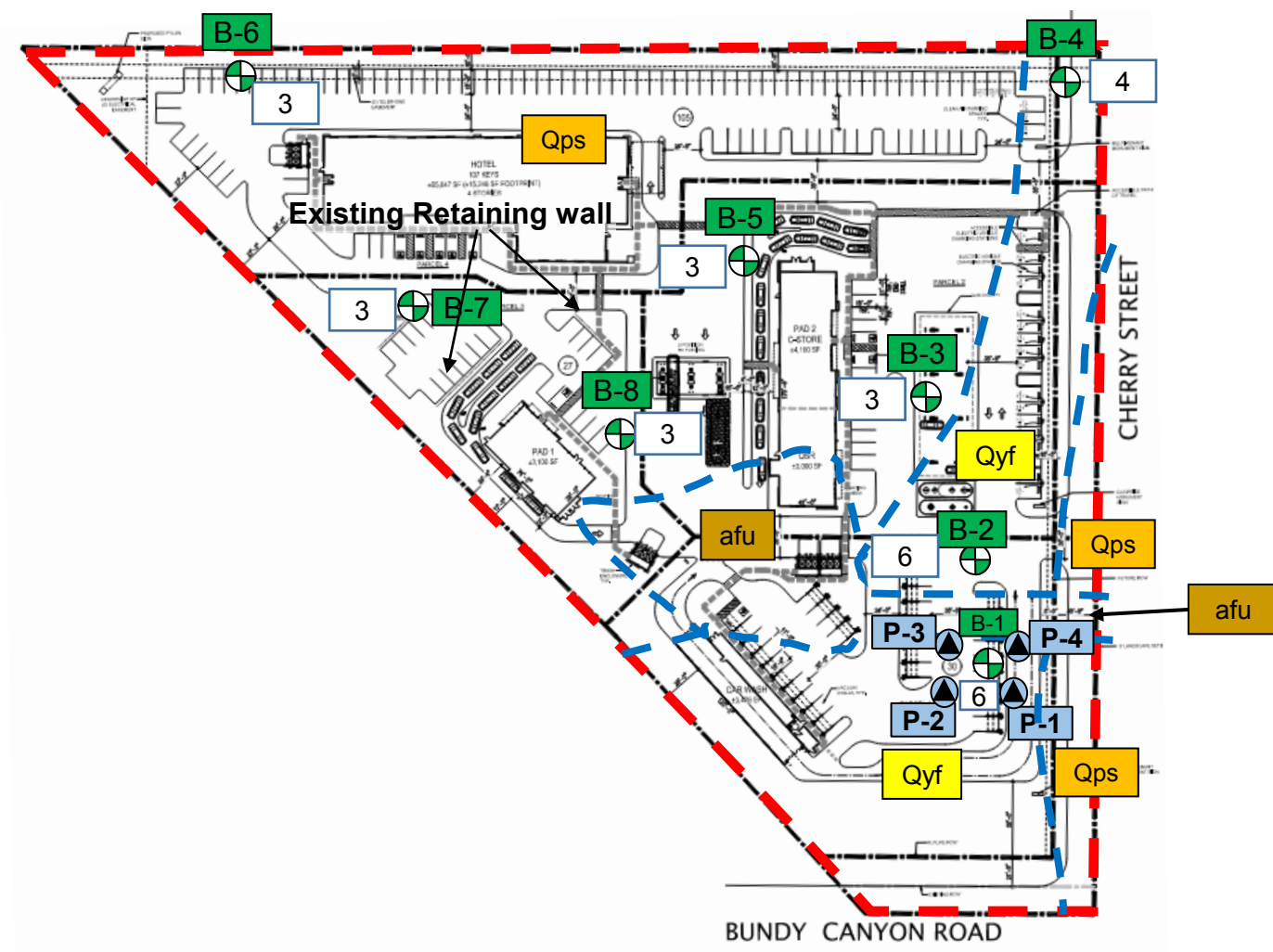
PROPOSED CHERRY OUTPOST
CHERRY ST & BUNDY CANYON RD
WILDOMAR, CALIFORNIA

HD

JANUARY 2023

PROJECT NO. T3010-22-01

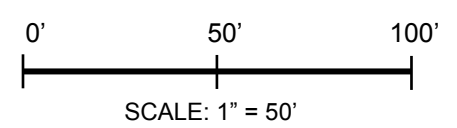
FIG. 1



GEOCON LEGEND

Locations are approximate

- B-8 HOLLOW STEM AUGER BORING LOCATION
- 6 DEPTH OF EXPECTED REMEDIAL REMOVAL BELOW EXISTING GRADE (ft)
- P-4 PERCOLATION TEST LOCATION
- LIMIT OF THE PROJECT
- afu UNDOCUMENTED ARTIFICIAL FILL
- Qyf YOUNG ALLUVIUM
- Qps POTATO SANDSTONE
- GEOLOGIC CONTACT



Source: KTG Architecture and Planning, 2022, *Conceptual Site Plan*, dated September 30.

GEOCON WEST, INC. <small>GEOTECHNICAL, ENVIRONMENTAL, MATERIALS 41571 CORNING PLACE #101, MURRIETA, CALIFORNIA 92562 PHONE 951-304-2300 www.geoconinc.com</small>	GEOLOGIC MAP		
	PROPOSED CHERRY OUTPOST CHERRY ST & BUNDY CANYON RD WILDOMAR, CALIFORNIA		
HD	JANUARY 2023	PROJECT NO. T3010-22-01	FIG. 2

APPENDIX

A

APPENDIX A

EXPLORATORY EXCAVATIONS

Eight exploratory borings and four percolation test borings were drilled on November 30, 2022. Percolation testing was conducted on December 1, 2022. The borings and percolation holes were drilled with a CME-75 truck mounted drill rig. Borings B-1 through B-8 were drilled to depths between 20½ and 26 feet. Percolation test borings were drilled to depths of 5 feet. Representative and relatively undisturbed samples were obtained by driving a 3-inch O. D., California Modified Sampler into the “undisturbed” soil mass with blows from an above-ground auto-hammer. The sampler was equipped with 1-inch by 2¾-inch brass sampler rings to facilitate removal and testing. Bulk samples were also obtained. The soil conditions encountered in the excavations were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Logs of the geotechnical borings are presented on Figures A-1 through A-8, with logs of the percolation test borings presented on Figures A-9 and A-12. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate locations of the borings are indicated on the *Geotechnical Map* (see Figure 2).

Percolation testing was performed in accordance with *Riverside County Flood Control and Water Conservation District, LID BMP Manual, Appendix A*. The percolation tests were run in general accordance with *Section 2.3, Shallow Percolation Test*. Percolation test result data are presented on Figures A-13 and A-16.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B-1		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	
					ELEV. (MSL.) 1410	DATE COMPLETED 11/30/2022				
					EQUIPMENT CME 75 HSA		BY: A. KAZEM			
MATERIAL DESCRIPTION										
0	B-1@0-5'			SM	Young ALLUVIAL Fan Deposits (Qall) Silty SAND, loose to medium dense, moist, orange brown; fine to coarse sand; abundant small pores					
2										
4										
6	B-1@5'				PAUBA SANDSTONE (Qps) Silty SAND, very dense, dry, orange brown; fine to coarse sand; less pores	55	101.5	5.2		
8										
10	B-1@10'					75/11"	121.0	7.8		
12										
14										
16	B-1@15'				-Becomes cemented	50/6"	98.1	7.2		
18										
20	B-1@20'				-Becomes moist; locally massive; driller added H2O to facilitate drilling	50/6"	122.0	5.3		
22										
24	B-1@25'				-Becomes dry, strong brown to orange brown.	50/5"	119.7	6.2		
					Total Depth = 25'5" Groundwater not encountered Penetration resistance for 140-lb hammer falling 30 inches by auto hammer Backfilled with cuttings 11/30/2022					

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

T3010-22-02 BORING LOGS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL		... 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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B-2		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) 1418	DATE COMPLETED 11/30/2022			
					EQUIPMENT CME 75 HSA BY: A. KAZEM				
					MATERIAL DESCRIPTION				
0	B-2@0-5'			SM	Young ALLUVIAL Fan Deposits (Quall) Silty SAND, loose to dense, dry, dark brown; fine to coarse sand				
2	B-2@2.5'				PAUBA SANDSTONE (Qps) Moist, Orange Brown, weathered, Cemented		59	123.6	3.9
4	B-2@5'						85	127.8	6.0
6	B-2@7.5'				-Becomes dry, Very dense		90/10"	125.7	12.0
8	B-2@10'				-Becomes very dense		50/6"	125.3	6.8
10	B-2@15'						83/12"	122.9	4.6
12	B-2@20'				-Becomes dense, light brown to strong brown; more cemented trace Clay		59	114.5	6.3
					Total Depth = 21'6" Groundwater not encountered Penetration resistance for 140-lb hammer falling 30 inches by auto hammer Backfilled with cuttings 11/30/2022				

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

T3010-22-02 BORING LOGS.GPJ







SAMPLE SYMBOLS		
	... SAMPLING UNSUCCESSFUL	
	... DISTURBED OR BAG SAMPLE	

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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B-3		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) 1423	DATE COMPLETED 11/30/2022			
					EQUIPMENT CME 75 HSA BY: A. KAZEM				
					MATERIAL DESCRIPTION				
0				SM	TOPSOIL Silty SAND, loose, strong brown				
2	B-3@2.5'				PAUBA SANDSTONE (Qps) Silty SAND, very dense, dry, light to orange brown; trace porosity		76/12"	110.5	5.2
4					-Becomes cemented				
6	B-3@5' B-3@0-5'						95/12"	116.1	10.9
8	B-3@7.5'				-Becomes very dense		77/12"	119.7	8.8
10	B-3@10'				Less dense		80	120.4	11.8
12									
14									
16	B-3@15'				Difficult Drilling		77/12"	119.5	9.9
18									
20	B-3@20'				Driller added water to facilitate drilling, very dense		80/12"	122.1	4.9
22									
24									
	B-3@25'						95/11"	127.6	4.4
					Total Depth = 25'11" Groundwater not encountered Penetration resistance for 140-lb hammer falling 30 inches by auto hammer Backfilled with cuttings 11/30/2022				

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

T3010-22-02 BORING LOGS.GPJ

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

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B-4		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) 1431	DATE COMPLETED 11/30/2022			
					EQUIPMENT CME 75 HSA BY: A. KAZEM				
					MATERIAL DESCRIPTION				
0	B-4@0-5'			SM	Young ALLUVIAL Fan Deposits (Qall) Silty SAND, loose, moist, strong brown				
2									
4					PAUBA SANDSTONE (Qps) Silty SAND, dense, dry, light brown; cemented				
6	B-4@5'						85/11"	124.7	4.4
8									
10	B-4@10'						72	127.3	7.4
12									
14									
16	B-4@15'				-Becomes Clayey		64	126.5	5.7
18									
20	B-4@20'				-Increase in sand		63	126.6	3
					Total Depth = 21'6" Groundwater not encountered Penetration resistance for 140-lb hammer falling 30 inches by auto hammer Backfilled with cuttings 11/30/2022				

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

T3010-22-02 BORING LOGS.GPJ







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

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B-5 ELEV. (MSL.) <u>1428</u> DATE COMPLETED <u>11/30/2022</u> EQUIPMENT <u>CME 75 HSA</u> BY: <u>A. KAZEM</u>	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					MATERIAL DESCRIPTION			
0				SM	TOPSOIL Silty SAND, loose, strong brown			
2					PAUBA SANDSTONE (Qps) Silty SAND, dense, dry, light orange brown			
6	B-5@5' B-5@0-5'					50	117.8	4.5
10	B-5@10'				-gray to brown	49	122.7	2.1
16	B-5@15'				-Becomes very dense, dry; cemented	85/12"	129.9	4.8
20	B-5@20'				-Becomes Clayey	78/12"	116.5	5.2
					Total Depth = 21' Groundwater not encountered Penetration resistance for 140-lb hammer falling 30 inches by auto hammer Backfilled with cuttings 11/30/2022			

Figure A-5,
Log of Boring B-5, Page 1 of 1

T3010-22-02 BORING LOGS.GPJ







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

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B-6		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) 1419	DATE COMPLETED 11/30/2022			
					EQUIPMENT CME 75 HSA BY: A. KAZEM				
					MATERIAL DESCRIPTION				
0				SM	TOPSOIL Silty SAND, loose, dry, strong brown				
2									
4					PAUBA SANDSTONE (Qps) Silty SAND, dense, dry, strong brown; cemented				
6	B-6@5' B-6@0-5'					90	116.1	6.6	
8									
10	B-6@10'				-Becomes very dense, light brown	75/12"	118.8	5.7	
12									
14									
16	B-6@15'				-Becomes dense, dry	82			
18									
20	B-6@20'				-Becomes lighter in color; increase in sand	71	121.8	3.0	
					Total Depth = 21'6" Groundwater not encountered Penetration resistance for 140-lb hammer falling 30 inches by auto hammer Backfilled with cuttings 11/30/2022				

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

T3010-22-02 BORING LOGS.GPJ







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

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B-7		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	
					ELEV. (MSL.) 1419	DATE COMPLETED 11/30/2022				
					EQUIPMENT CME 75 HSA		BY: A. KAZEM			
MATERIAL DESCRIPTION										
0				SM	TOPSOIL Silty SAND, loose, dry, strong brown					
2										
4					PAUBA SANDSTONE (Qps) Silty SAND, medium dense, dry, light brown to strong brown					
6	B-7@5' B-7@0-5'					48	129.5	5.3		
8										
10	B-7@10'				-Becomes cemented	49				
12										
14										
16	B-7@15'				-density increases with depth	75	132.5	8.4		
18										
20	B-7@20'				-Becomes very dense	78/12"	129.8	7.7		
					Total Depth = 21' Groundwater not encountered Penetration resistance for 140-lb hammer falling 30 inches by auto hammer Backfilled with cuttings 11/30/2022					

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

T3010-22-02 BORING LOGS.GPJ

SAMPLE SYMBOLS			
	... SAMPLING UNSUCCESSFUL		... STANDARD PENETRATION TEST
	... DISTURBED OR BAG SAMPLE		... DRIVE SAMPLE (UNDISTURBED)
	... 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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING B-8		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)	
					ELEV. (MSL.) 1421	DATE COMPLETED 11/30/2022				
					EQUIPMENT CME 75 HSA		BY: A. KAZEM			
MATERIAL DESCRIPTION										
0	B-8@0-5'			SM	TOPSOIL Silty SAND, loose, dry, strong brown					
2	B-8@2.5'				PAUBA SANDSTONE (Qps) Silty SAND, dense, dry; strong brown Becomes medium dense	69	123.0	3.3		
4	B-8@5'					28	118.0	3.5		
8	B-8@7.5'				-Becomes cemented	21	121.0	2.9		
10	B-8@10'					44	120.1	2.7		
16	B-8@15'				-Becomes very dense	88	127.2	5.5		
20	B-8@20'				-Difficult drilling	66	130.2	12.0		
					Total Depth = 21'6" Groundwater not encountered Penetration resistance for 140-lb hammer falling 30 inches by auto hammer Backfilled with cuttings 11/30/2022					

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

T3010-22-02 BORING LOGS.GPJ

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

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

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P-1		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) 1410	DATE COMPLETED 11/30/2022			
					EQUIPMENT _____ BY: A. KAZEM				
0					MATERIAL DESCRIPTION				
2				SM	ALLUVIUM (Qyf) Silty SAND, medium dense, dry, orange brown; fine to coarse sand; abundant small pores				
4					Total Depth = 5' Groundwater not encountered Percolation Test Equipment set Presaturated with 5 gallons of water Backfilled with cuttings 12/1/2022				

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

T3010-22-02 BORING LOGS.GPJ

SAMPLE SYMBOLS	<input type="checkbox"/>	... SAMPLING UNSUCCESSFUL	<input type="checkbox"/>	... STANDARD PENETRATION TEST	<input checked="" type="checkbox"/>	... DRIVE SAMPLE (UNDISTURBED)
	<input checked="" type="checkbox"/>	... DISTURBED OR BAG SAMPLE	<input checked="" type="checkbox"/>	... CHUNK SAMPLE	<input checked="" type="checkbox"/>	... 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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P-2		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) 1410	DATE COMPLETED 11/30/2022			
					EQUIPMENT _____ BY: A. KAZEM				
0					MATERIAL DESCRIPTION				
2				SM	ALLUVIUM (Qyf) Silty SAND, medium dense, dry, orange brown; fine to coarse sand; abundant small pores				
4					Total Depth = 5' Groundwater not encountered Percolation Test Equipment set Presaturated with 5 gallons of water Backfilled with cuttings 12/1/2022				

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

T3010-22-02 BORING LOGS.GPJ

SAMPLE SYMBOLS	<input type="checkbox"/> ... SAMPLING UNSUCCESSFUL	<input type="checkbox"/> ... STANDARD PENETRATION TEST	<input type="checkbox"/> ... DRIVE SAMPLE (UNDISTURBED)
	<input checked="" type="checkbox"/> ... DISTURBED OR BAG SAMPLE	<input checked="" type="checkbox"/> ... CHUNK SAMPLE	<input type="checkbox"/> ... 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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P-3		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) 1410	DATE COMPLETED 11/30/2022			
					EQUIPMENT _____ BY: A. KAZEM				
					MATERIAL DESCRIPTION				
0									
2									
4				SM	ALLUVIUM (Qyf) Silty SAND, medium dense, dry, orange brown; fine to coarse sand; abundant small pores				
					Total Depth = 5' Groundwater not encountered Percolation Test Equipment set Presaturated with 5 gallons of water Backfilled with cuttings 12/1/2022				

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

T3010-22-02 BORING LOGS.GPJ

SAMPLE SYMBOLS	<input type="checkbox"/>	... SAMPLING UNSUCCESSFUL	<input type="checkbox"/>	... STANDARD PENETRATION TEST	<input checked="" type="checkbox"/>	... DRIVE SAMPLE (UNDISTURBED)
	<input checked="" type="checkbox"/>	... DISTURBED OR BAG SAMPLE	<input checked="" type="checkbox"/>	... CHUNK SAMPLE	<input type="checkbox"/>	... 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.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING P-4		PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) 1410	DATE COMPLETED 11/30/2022			
					EQUIPMENT _____ BY: A. KAZEM				
					MATERIAL DESCRIPTION				
0				SM	ALLUVIUM (Qyf) Silty SAND, medium dense, dry, orange brown; fine to coarse sand; abundant small pores				
2									
4									
					Total Depth = 5' Groundwater not encountered Percolation Test Equipment set Presaturated with 5 gallons of water Backfilled with cuttings 12/1/2022				

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

T3010-22-02 BORING LOGS.GPJ

SAMPLE SYMBOLS	<input type="checkbox"/> ... SAMPLING UNSUCCESSFUL	<input type="checkbox"/> ... STANDARD PENETRATION TEST	<input type="checkbox"/> ... DRIVE SAMPLE (UNDISTURBED)
	<input checked="" type="checkbox"/> ... DISTURBED OR BAG SAMPLE	<input checked="" type="checkbox"/> ... CHUNK SAMPLE	<input type="checkbox"/> ... 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.

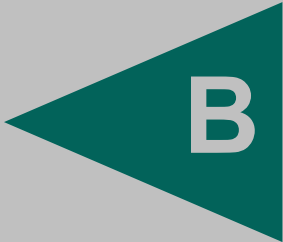
PERCOLATION TEST REPORT							
Project Name:		Cherry Outpost GI			Project No.:		T3010-22-01
Test Hole No.:		P-1			Date Excavated:		11/30/2022
Length of Test Pipe:		60.0 inches			Soil Classification:		0
Height of Pipe above Ground:		6.0 inches			Presoak Date:		11/30/2022
Depth of Test Hole:		54.0 inches			Perc Test Date:		12/1/2022
Check for Sandy Soil Criteria Tested by:				AK		Percolation Tested by: AK	
Water level measured from BOTTOM of hole							
Sandy Soil Criteria Test							
Trial No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in)	Final Water Level (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	9:00 AM	25	25	39.0	37.0	2.0	12.5
	9:25 AM						
2	9:25 AM	25	50	40.0	35.5	4.5	5.6
	9:50 AM						
Soil Criteria: Normal							
Percolation Test							
Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	10:00 AM	30	30	40.8	37.5	3.3	9.2
	10:30 AM						
2	10:30 AM	30	60	41.0	38.0	3.0	10.0
	11:00 AM						
3	11:00 AM	30	90	38.0	35.0	3.0	10.0
	11:30 AM						
4	11:30 AM	30	120	41.0	38.3	2.8	10.9
	12:00 PM						
5	12:00 PM	30	150	38.3	35.5	2.8	10.9
	12:30 PM						
6	12:30 PM	30	180	40.0	37.3	2.8	10.9
	1:00 PM						
7	1:00 PM	30	210	37.3	34.8	2.5	12.0
	1:30 PM						
8	1:30 PM	30	240	41.5	39.0	2.5	12.0
	2:00 PM						
9	2:00 PM	30	270	39.0	36.5	2.5	12.0
	2:30 PM						
10	2:30 PM	30	300	40.8	38.3	2.5	12.0
	3:00 PM						
11	3:00 PM	30	330	38.3	35.8	2.5	12.0
	3:30 PM						
12	3:30 PM	30	360	40.3	37.8	2.5	12.0
	4:00 PM						
Infiltration Rate (in/hr):			0.2				
Radius of test hole (in):			4	Figure A-13			
Average Head (in):			39.0				

PERCOLATION TEST REPORT							
Project Name:		Cherry Outpost GI			Project No.:		T3010-22-01
Test Hole No.:		P-2			Date Excavated:		11/30/2022
Length of Test Pipe:		60.0 inches			Soil Classification:		SM
Height of Pipe above Ground:		5.0 inches			Presoak Date:		11/30/2022
Depth of Test Hole:		55.0 inches			Perc Test Date:		12/1/2022
Check for Sandy Soil Criteria Tested by:				AK		Percolation Tested by: AK	
Water level measured from BOTTOM of hole							
Sandy Soil Criteria Test							
Trial No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in)	Final Water Level (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	9:01 AM	25	25	41.5	38.0	3.5	7.1
	9:26 AM						
2	9:26 AM	25	50	43.0	39.0	4.0	6.2
	9:51 AM						
Soil Criteria: Normal							
Percolation Test							
Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	10:00 AM	30	30	43.8	39.0	4.8	6.3
	10:30 AM						
2	10:30 AM	30	60	45.0	40.8	4.3	7.1
	11:00 AM						
3	11:00 AM	30	90	40.8	36.8	4.0	7.5
	11:30 AM						
4	11:30 AM	30	120	44.5	40.8	3.8	8.0
	12:00 PM						
5	12:00 PM	30	150	40.8	37.0	3.8	8.0
	12:30 PM						
6	12:30 PM	30	180	44.5	41.0	3.5	8.6
	1:00 PM						
7	1:00 PM	30	210	41.0	37.5	3.5	8.6
	1:30 PM						
8	1:30 PM	30	240	44.8	41.5	3.3	9.2
	2:00 PM						
9	2:00 PM	30	270	41.5	38.3	3.3	9.2
	2:30 PM						
10	2:30 PM	30	300	43.8	40.8	3.0	10.0
	3:00 PM						
11	3:00 PM	30	330	40.8	37.8	3.0	10.0
	3:30 PM						
12	3:30 PM	30	360	41.8	38.8	3.0	10.0
	4:00 PM						
Infiltration Rate (in/hr):			0.3				
Radius of test hole (in):			4	Figure A-14			
Average Head (in):			40.3				

PERCOLATION TEST REPORT							
Project Name:		Cherry Outpost GI			Project No.:		T3010-22-01
Test Hole No.:		P-3			Date Excavated:		11/30/2022
Length of Test Pipe:		63.0 inches			Soil Classification:		SM
Height of Pipe above Ground:		6.0 inches			Presoak Date:		11/30/2022
Depth of Test Hole:		57.0 inches			Perc Test Date:		12/1/2022
Check for Sandy Soil Criteria Tested by:				AK		Percolation Tested by: AK	
Water level measured from BOTTOM of hole							
Sandy Soil Criteria Test							
Trial No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Level (in)	Final Water Level (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	9:02 AM	25	25	46.0	41.0	5.0	5.0
	9:27 AM						
2	9:27 AM	25	50	43.5	38.0	5.5	4.5
	9:52 AM						
Soil Criteria: Normal							
Percolation Test							
Reading No.	Time	Time Interval (min)	Total Elapsed Time (min)	Initial Water Head (in)	Final Water Head (in)	Δ in Water Level (in)	Percolation Rate (min/inch)
1	10:02 AM	30	30	44.0	38.3	5.8	5.2
	10:32 AM						
2	10:32 AM	30	60	45.0	38.8	6.3	4.8
	11:02 AM						
3	11:02 AM	30	90	38.8	32.8	6.0	5.0
	11:32 AM						
4	11:32 AM	30	120	45.0	38.8	6.3	4.8
	12:02 PM						
5	12:02 PM	30	150	38.8	32.3	6.5	4.6
	12:32 PM						
6	12:32 PM	30	180	44.8	38.8	6.0	5.0
	1:02 PM						
7	1:02 PM	30	210	38.8	32.8	6.0	5.0
	1:32 PM						
8	1:32 PM	30	240	42.8	37.0	5.8	5.2
	2:02 PM						
9	2:02 PM	30	270	37.0	31.3	5.8	5.2
	2:32 PM						
10	2:32 PM	30	300	43.5	38.0	5.5	5.5
	3:02 PM						
11	3:02 PM	30	330	45.8	40.3	5.5	5.5
	3:32 PM						
12	3:32 PM	30	360	40.3	34.8	5.5	5.5
	4:02 PM						
Infiltration Rate (in/hr):			0.6				
Radius of test hole (in):			4	Figure A-15			
Average Head (in):			37.5				

PERCOLATION TEST REPORT							
Project Name:		Cherry Outpost GI			Project No.:		T3010-22-01
Test Hole No.:		P-4			Date Excavated:		11/30/2022
Length of Test Pipe:		60.0 inches			Soil Classification:		SM
Height of Pipe above Ground:		6.0 inches			Presoak Date:		11/30/2022
Depth of Test Hole:		54.0 inches			Perc Test Date:		12/1/2022
Check for Sandy Soil Criteria Tested by:				AK		Percolation Tested by: AK	
Water level measured from BOTTOM of hole							
Sandy Soil Criteria Test							
Trial No.	Time	Time Interval	Total Elapsed Time	Initial Water Level	Final Water Level	Δ in Water Level	Percolation Rate
		(min)	(min)	(in)	(in)	(in)	(min/inch)
1	9:03 AM	25	25	40.5	35.0	5.5	4.5
	9:28 AM						
2	9:28 AM	25	50	39.0	33.3	5.8	4.3
	9:53 AM						
Soil Criteria: Normal							
Percolation Test							
Reading No.	Time	Time Interval	Total Elapsed Time	Initial Water Head	Final Water Head	Δ in Water Level	Percolation Rate
		(min)	(min)	(in)	(in)	(in)	(min/inch)
1	10:03 AM	30	30	38.0	31.8	6.3	4.8
	10:33 AM						
2	10:33 AM	30	60	40.0	33.8	6.3	4.8
	11:03 AM						
3	11:03 AM	30	90	33.8	27.5	6.3	4.8
	11:33 AM						
4	11:33 AM	30	120	34.5	28.8	5.8	5.2
	12:03 PM						
5	12:03 PM	30	150	28.8	23.0	5.8	5.2
	12:33 PM						
6	12:33 PM	30	180	39.5	34.0	5.5	5.5
	1:03 PM						
7	1:03 PM	30	210	34.0	28.5	5.5	5.5
	1:33 PM						
8	1:33 PM	30	240	37.5	32.3	5.3	5.7
	2:03 PM						
9	2:03 PM	30	270	32.3	27.0	5.3	5.7
	2:33 PM						
10	2:33 PM	30	300	42.8	37.8	5.0	6.0
	3:03 PM						
11	3:03 PM	30	330	37.8	32.8	5.0	6.0
	3:33 PM						
12	3:33 PM	30	360	40.5	35.5	5.0	6.0
	4:03 PM						
Infiltration Rate (in/hr):			0.5				
Radius of test hole (in):			4	Figure A-16			
Average Head (in):			38.0				

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 in-place dry density/moisture content, maximum dry density/optimum moisture content, expansion potential, corrosivity, grain size analysis, consolidation potential, and direct shear strength. The results of the laboratory tests are presented on Figures B-1 through B-13 and on the logs presented in *Appendix A*.

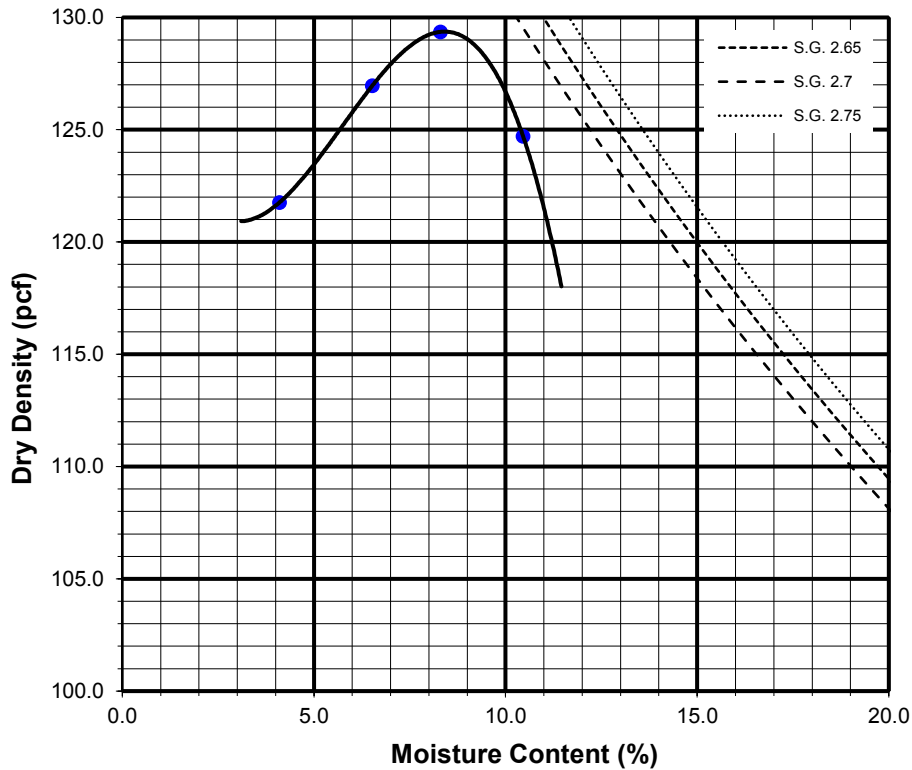
Sample No:

B1@0-5	Silty SAND (SM), brown
---------------	------------------------

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6174	6302	6375	6340		
Weight of Mold	(g)	4259	4259	4259	4259		
Net Weight of Soil	(g)	1915	2043	2116	2081		
Wet Weight of Soil + Cont.	(g)	654.9	642.7	761.8	717.5		
Dry Weight of Soil + Cont.	(g)	639.2	619.1	723.3	674.1		
Weight of Container	(g)	256.4	257.2	259.4	259.1		
Moisture Content	(%)	4.1	6.5	8.3	10.5		
Wet Density	(pcf)	126.8	135.2	140.1	137.8		
Dry Density	(pcf)	121.8	127.0	129.4	124.7		

Maximum Dry Density (pcf) 129.5

Optimum Moisture Content (%) 8.5



Preparation Method: A



COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS

ASTM D-1557

Checked by: ARK

Project No.: T3010-22-01

PROPOSED CHERRY OUTPOST
NWC OF BUNDY CANYON AND CHERRY STREET
WILDOMAR, CALIFORNIA

Jan 23

Figure B-1

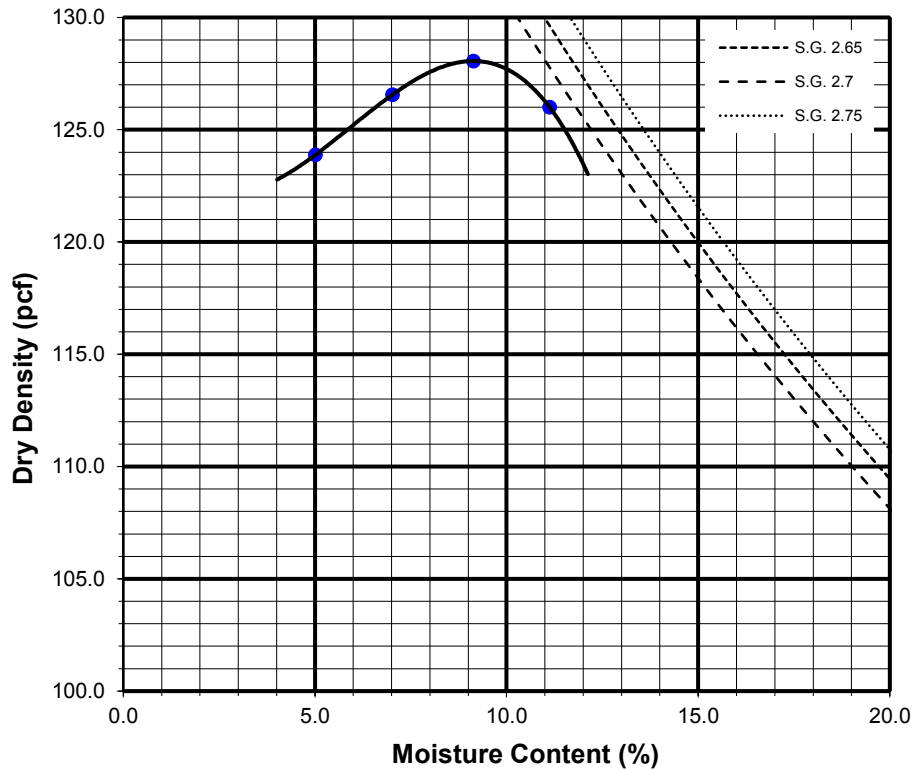
Sample No:

B3@0-5'	Silty SAND (SM), brown
----------------	------------------------

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6333	6337	6268	6187		
Weight of Mold	(g)	4222	4222	4222	4222		
Net Weight of Soil	(g)	2111	2115	2046	1965		
Wet Weight of Soil + Cont.	(g)	767.8	730.0	699.2	674.6		
Dry Weight of Soil + Cont.	(g)	719.0	675.4	665.4	651.2		
Weight of Container	(g)	184.8	184.4	184.2	184.4		
Moisture Content	(%)	9.1	11.1	7.0	5.0		
Wet Density	(pcf)	139.8	140.0	135.5	130.1		
Dry Density	(pcf)	128.1	126.0	126.6	123.9		

Maximum Dry Density (pcf) 128.0

Optimum Moisture Content (%) 9.0



Preparation Method: A



COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS

ASTM D-1557

Checked by: ARK

Project No.: T3010-22-01

PROPOSED CHERRY OUTPOST
NWC OF BUNDY CANYON AND CHERRY STREET
WILDOMAR, CALIFORNIA

Jan 23

Figure B-2

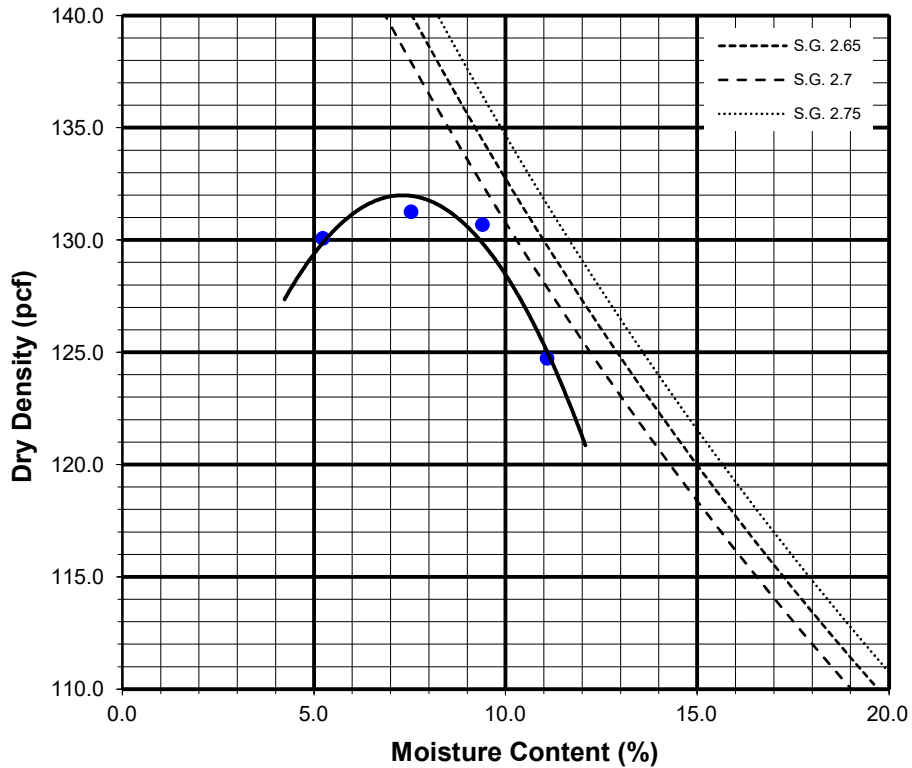
Sample No:

B6@0-5	Silty SAND (SM), brown
---------------	------------------------

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6327	6391	6419	6352		
Weight of Mold	(g)	4259	4259	4259	4259		
Net Weight of Soil	(g)	2068	2132	2160	2093		
Wet Weight of Soil + Cont.	(g)	674.3	707.6	776.2	730.5		
Dry Weight of Soil + Cont.	(g)	653.5	676.2	731.5	683.4		
Weight of Container	(g)	256.0	259.2	255.8	258.4		
Moisture Content	(%)	5.2	7.5	9.4	11.1		
Wet Density	(pcf)	136.9	141.1	143.0	138.6		
Dry Density	(pcf)	130.1	131.3	130.7	124.7		

Maximum Dry Density (pcf) 131.5

Optimum Moisture Content (%) 8.5



Preparation Method: A



**COMPACTION CHARACTERISTICS USING
MODIFIED EFFORT TEST RESULTS**

ASTM D-1557

Checked by: ARK

Project No.: T3010-22-01

PROPOSED CHERRY OUTPOST
NWC OF BUNDY CANYON AND CHERRY STREET
WILDOMAR, CALIFORNIA

Jan 23

Figure B-3

B3@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)	630.3	660.5
Wt. of Mold	(gm)	200.3	200.3
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	473.2	660.5
Dry Wt. of Soil + Cont.	(gm)	452.3	400.0
Wt. of Container	(gm)	173.2	200.3
Moisture Content	(%)	7.5	15.1
Wet Density	(pcf)	129.7	138.6
Dry Density	(pcf)	120.7	120.5
Void Ratio		0.4	0.4
Total Porosity		0.3	0.3
Pore Volume	(cc)	58.8	62.8
Degree of Saturation	(%) [S_{meas}]	51.4	95.8

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
12/14/2022	10:00	1.0	0	0.4069
12/14/2022	10:10	1.0	10	0.4066
Add Distilled Water to the Specimen				
12/15/2022	10:00	1.0	1430	0.4258
12/15/2022	11:00	1.0	1490	0.4258

Expansion Index (EI meas) =	19.2
Expansion Index (Report) =	19

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 18-1-B.



EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: ARK

Project No.: T3010-22-01

PROPOSED CHERRY OUTPOST
NWC OF BUNDY CANYON AND CHERRY STREET
WILDOMAR, CALIFORNIA

Jan 23

Figure B-4

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)
B3@0-5	9.3	1900 (Corrosive)
B6@0-5	8.8	4400 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
 AASHTO T291 ASTM C1218

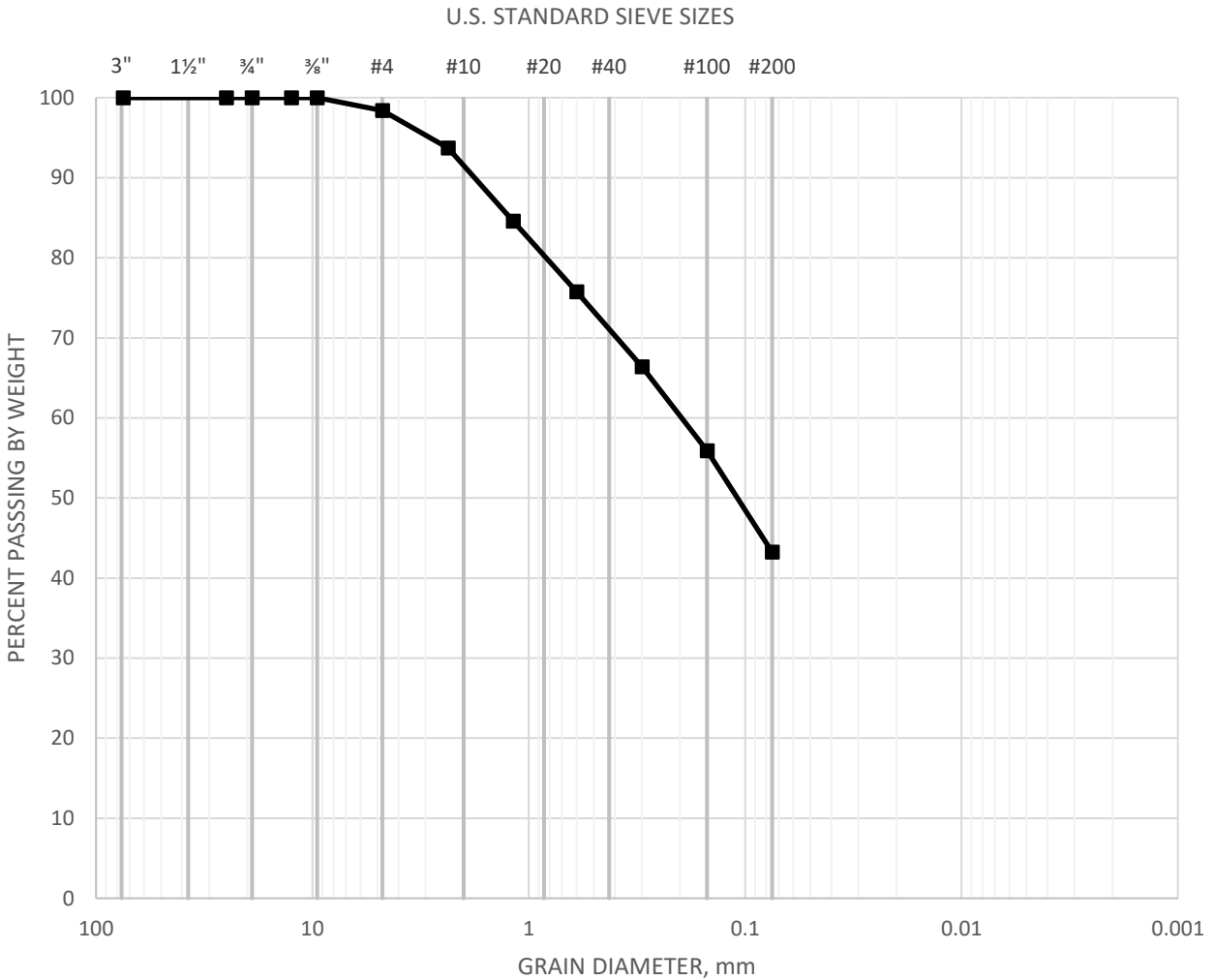
Sample No.	Chloride Ion Content (%)
B3@0-5	0.012
B6@0-5	0.008

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

Sample No.	Water Soluble Sulfate (% SO ₄)	Sulfate Exposure
B3@0-5	0.000	S0
B6@0-5	0.000	S0

 GEOCON	CORROSIVITY TEST RESULTS	Project No.: T3010-22-01
		PROPOSED CHERRY OUTPOST NWC OF BUNDY CANYON AND CHERRY STREET WILDOMAR, CALIFORNIA
	Checked by: ARK	Jan 23 Figure B-5

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



SAMPLE	CLASSIFICATION	D60	D30	D10
B1@0-5	Silty SAND (SM), brown	0.2	0.075	0.075



GRAIN SIZE DISTRIBUTION

ASTM D 6913

Checked by: ARK

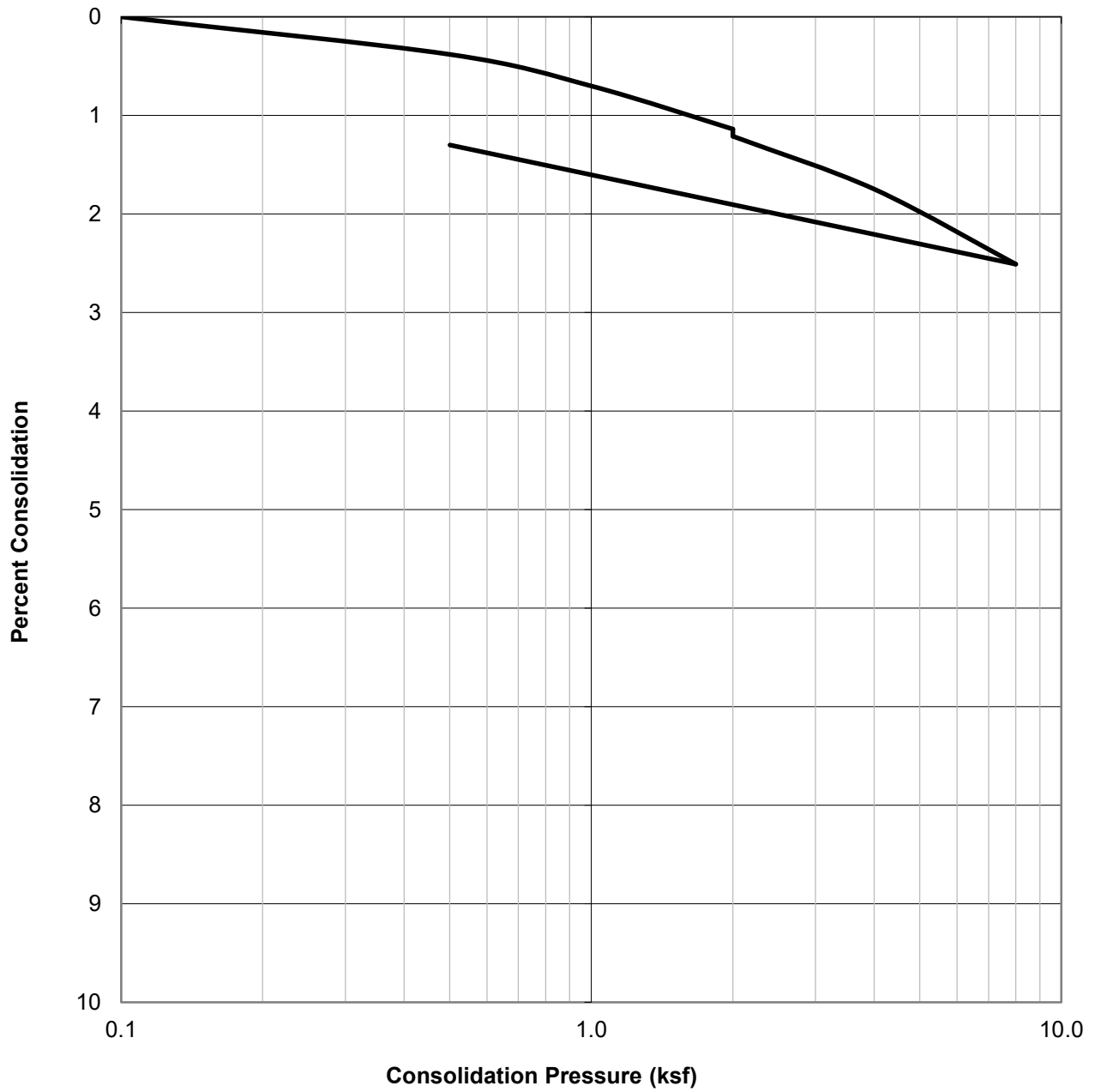
Project No.: T3010-22-01

PROPOSED CHERRY OUTPOST
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WILDOMAR, CALIFORNIA

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Figure B-6

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@0-5	Silty SAND (SM), brown	122.2	8.9	13.6



CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: ARK

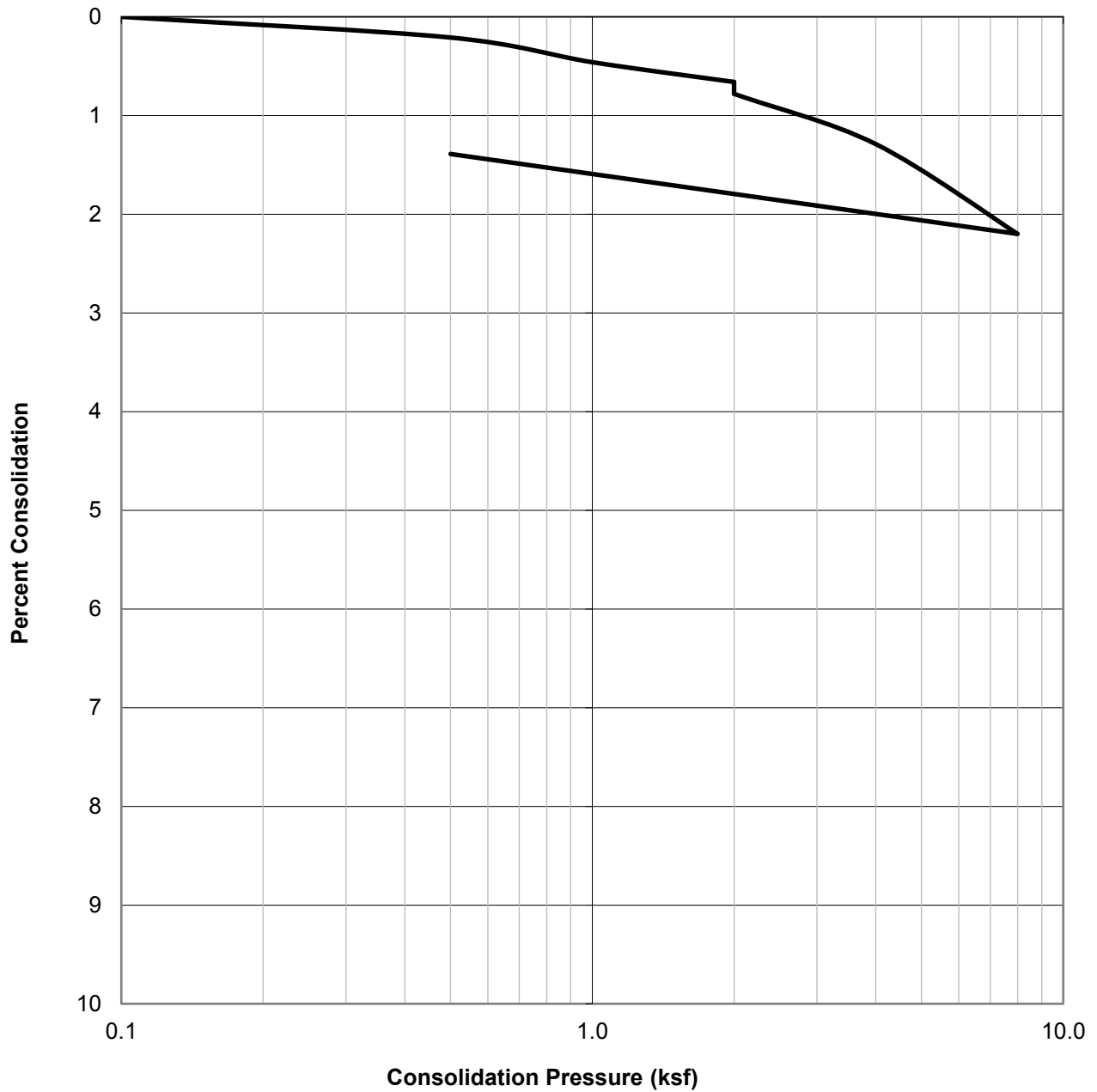
Project No.: T3010-22-01

PROPOSED CHERRY OUTPOST
NWC OF BUNDY CANYON AND CHERRY STREET
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Figure B-7

WATER ADDED AT 2.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B6@0-5	Silty SAND (SM), brown	117.4	9.0	14.3



CONSOLIDATION TEST RESULTS

ASTM D-2435

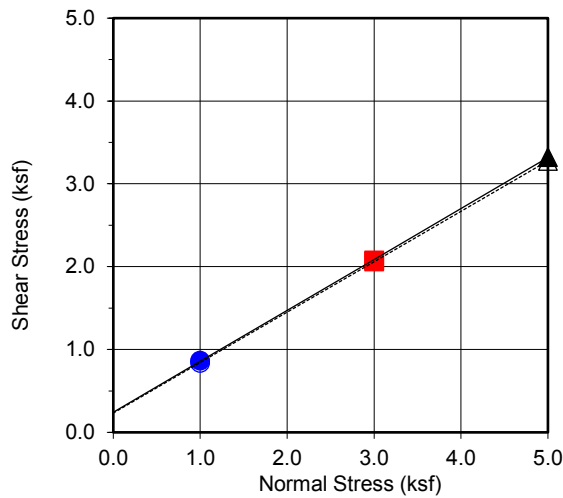
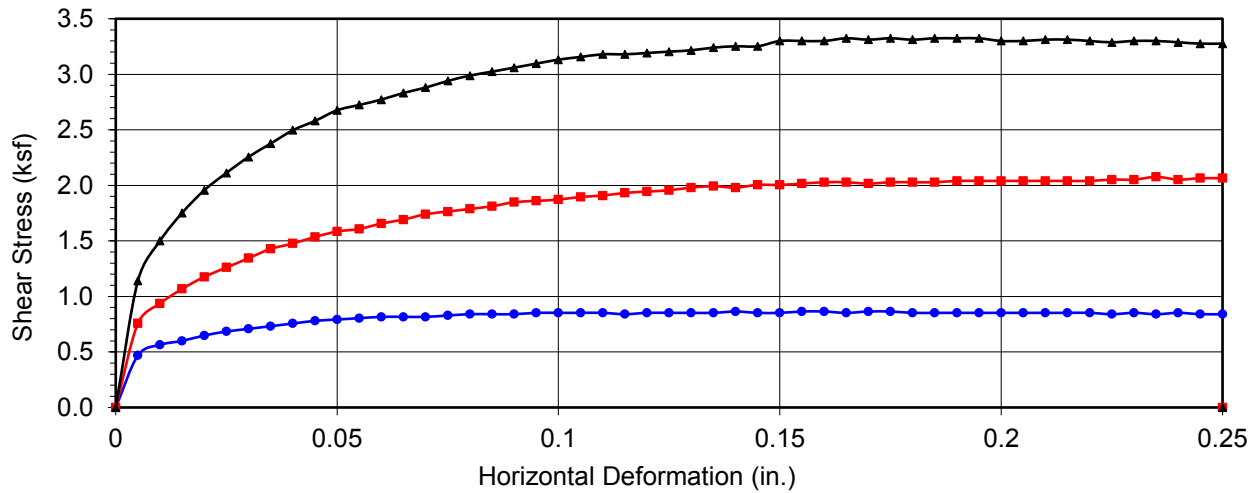
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Project No.: T3010-22-01

PROPOSED CHERRY OUTPOST
NWC OF BUNDY CANYON AND CHERRY STREET
WILDOMAR, CALIFORNIA

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Figure B-8



Boring No.	B3
Sample No.	B3@0-5
Depth (ft)	0-5
<u>Sample Type:</u>	Bulk

<u>Soil Identification:</u>		
Silty SAND (SM), brown		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	243	32
Ultimate	233	31

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 0.86	■ 2.08	▲ 3.32
Shear Stress @ End of Test (ksf)	○ 0.84	□ 2.06	△ 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 (%)	9.1	8.9	9.0
Initial Dry Density (pcf)	115.1	115.0	115.0
Initial Degree of Saturation (%)	52.7	51.3	52.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	14.6	12.6	10.9



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

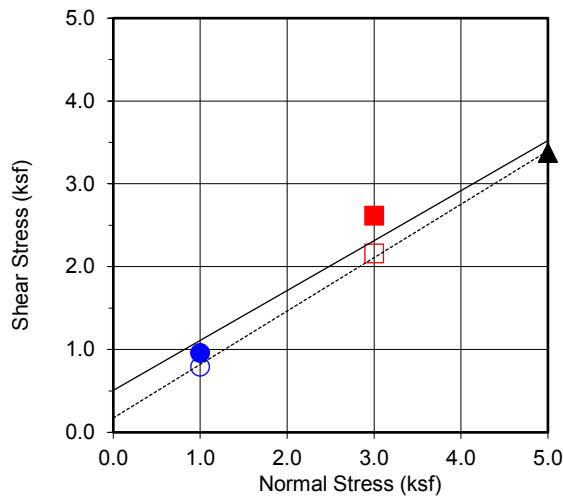
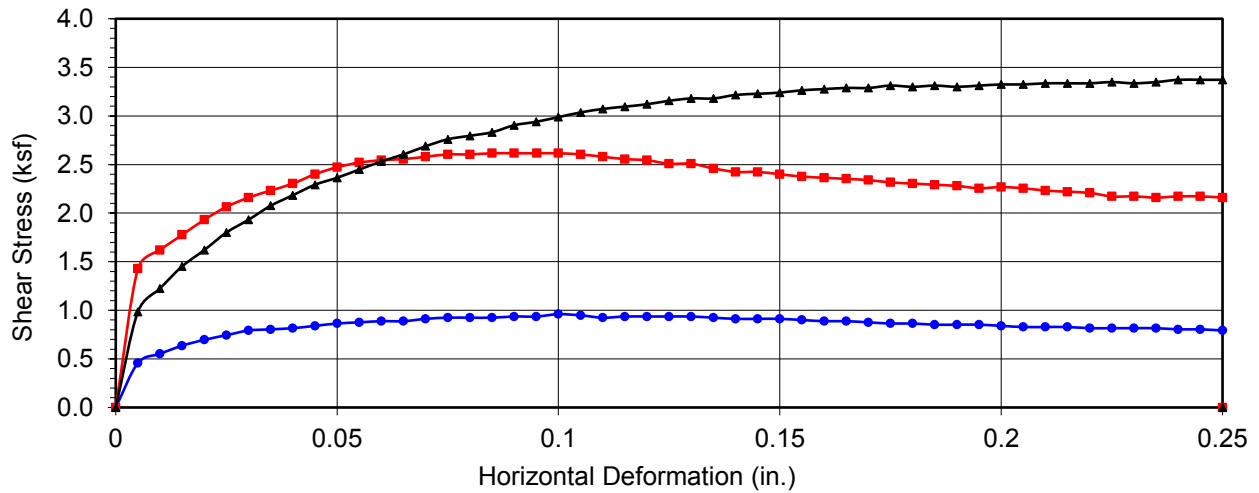
Checked by: ARK

Project No.: T3010-22-01

PROPOSED CHERRY OUTPOST
NWC OF BUNDY CANYON AND CHERRY STREET
WILDOMAR, CALIFORNIA

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Figure B-9



Boring No.	B3
Sample No.	B3@5
Depth (ft)	5
<u>Sample Type:</u>	Ring

<u>Soil Identification:</u>		
Silty SAND (SM), brown		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	507	31
Ultimate	173	33

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 0.96	■ 2.62	▲ 3.37
Shear Stress @ End of Test (ksf)	○ 0.79	□ 2.16	△ 3.37
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 (%)	10.9	10.3	10.3
Initial Dry Density (pcf)	124.3	122.7	119.4
Initial Degree of Saturation (%)	82.5	74.1	67.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.4	15.9	15.5



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

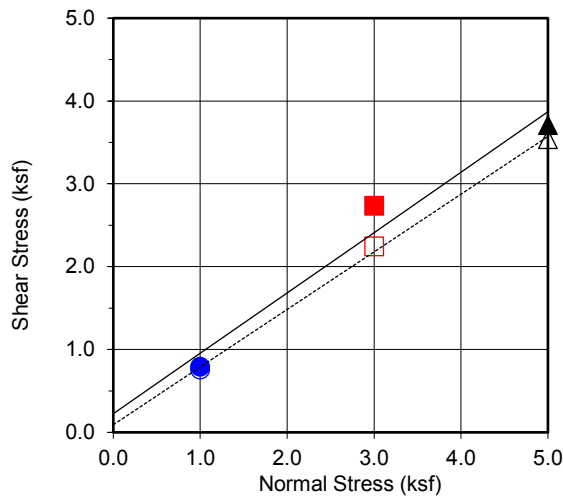
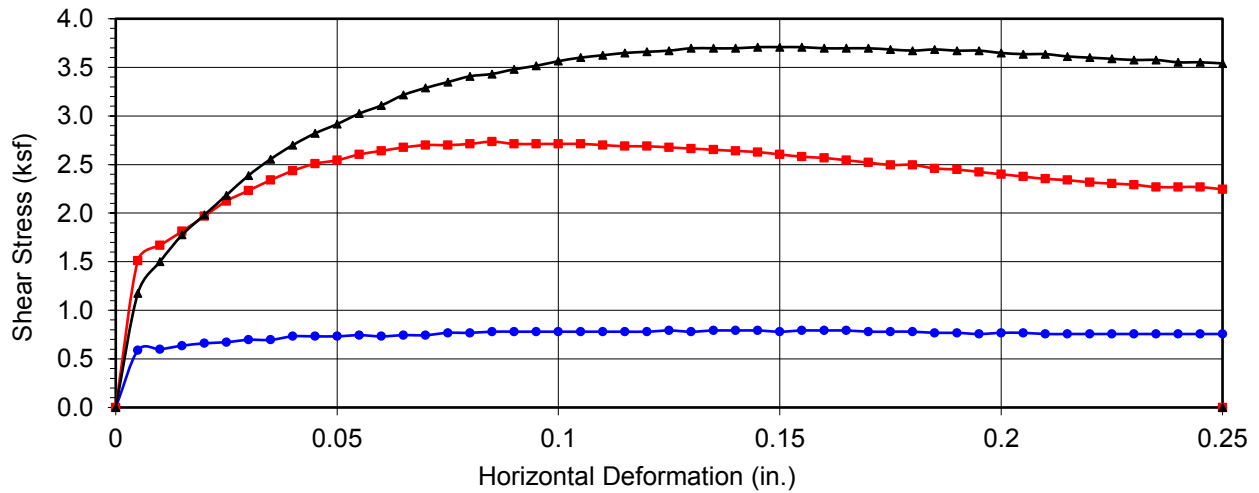
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PROPOSED CHERRY OUTPOST
NWC OF BUNDY CANYON AND CHERRY STREET
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Figure B-10



Boring No.	B3
Sample No.	B3@10
Depth (ft)	10
Sample Type:	ring

<u>Soil Identification:</u>		
Silty SAND (SM), brown		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	225	36
Ultimate	92	35

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 0.79	■ 2.74	▲ 3.71
Shear Stress @ End of Test (ksf)	○ 0.76	□ 2.24	△ 3.54
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 (%)	13.2	11.8	11.0
Initial Dry Density (pcf)	116.7	127.6	115.2
Initial Degree of Saturation (%)	80.2	99.1	64.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.6	15.6	16.2



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

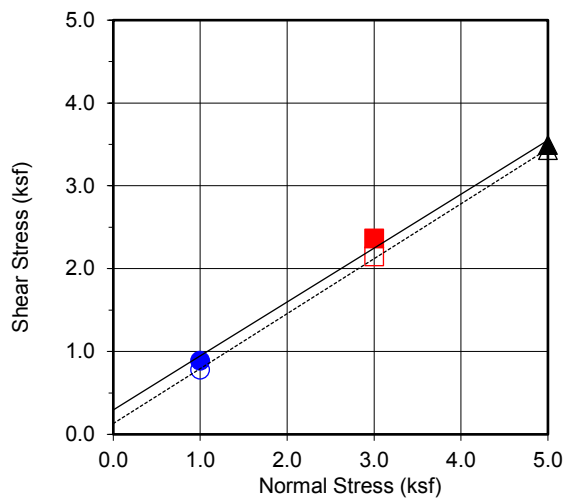
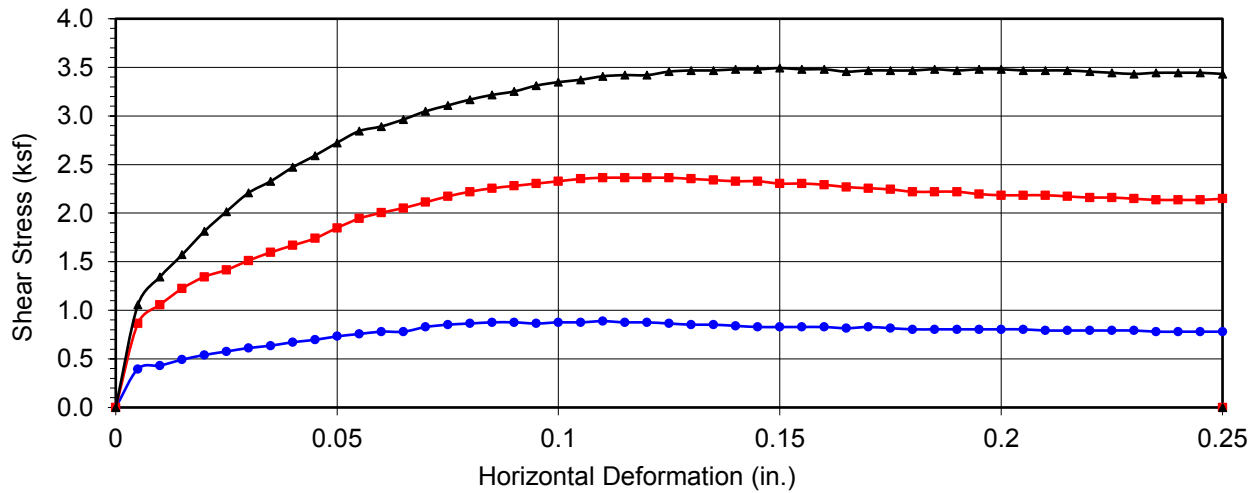
Checked by: ARK

Project No.: T3010-22-01

PROPOSED CHERRY OUTPOST
NWC OF BUNDY CANYON AND CHERRY STREET
WILDOMAR, CALIFORNIA

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Figure B-11



Boring No.	B3
Sample No.	B3@15
Depth (ft)	15
<u>Sample Type:</u>	Ring

<u>Soil Identification:</u>		
Silty SAND (SM), brown		
Strength Parameters		
	C (psf)	ϕ ($^{\circ}$)
Peak	295	33
Ultimate	131	34

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 0.89	■ 2.36	▲ 3.49
Shear Stress @ End of Test (ksf)	○ 0.78	□ 2.15	△ 3.43
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 (%)	10.1	8.7	8.0
Initial Dry Density (pcf)	113.2	113.1	114.2
Initial Degree of Saturation (%)	55.7	48.1	45.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.0	15.0	15.0



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

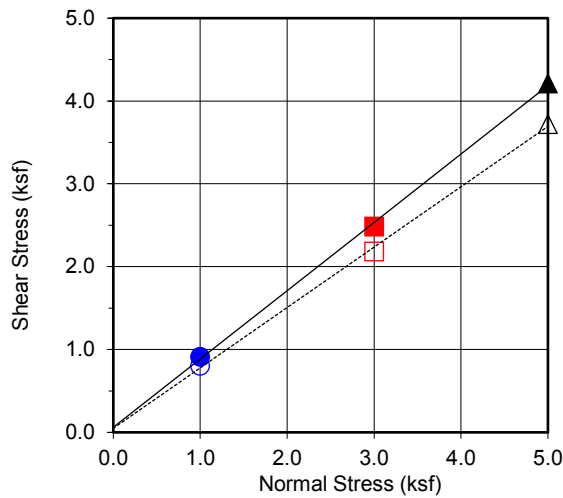
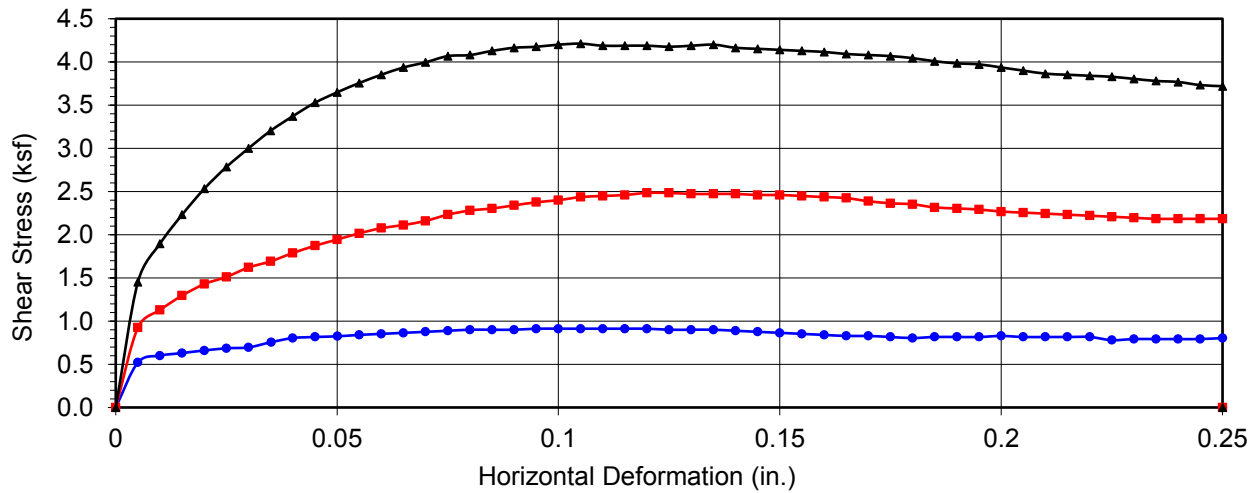
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PROPOSED CHERRY OUTPOST
NWC OF BUNDY CANYON AND CHERRY STREET
WILDOMAR, CALIFORNIA

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Figure B-12



Boring No.	B6
Sample No.	B6@5
Depth (ft)	5
Sample Type:	Ring

<u>Soil Identification:</u>		
SM		
Strength Parameters		
	C (psf)	ϕ ($^{\circ}$)
Peak	61	40
Ultimate	49	36

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 0.91	■ 2.48	▲ 4.21
Shear Stress @ End of Test (ksf)	○ 0.80	□ 2.18	△ 3.72
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 (%)	7.2	7.2	6.6
Initial Dry Density (pcf)	106.7	113.4	124.6
Initial Degree of Saturation (%)	33.5	40.1	50.6
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.1	15.3	14.0



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

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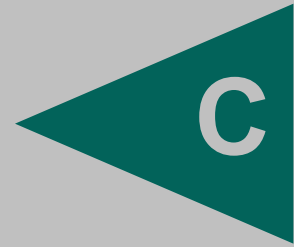
Project No.: T3010-22-01

PROPOSED CHERRY OUTPOST
NWC OF BUNDY CANYON AND CHERRY STREET
WILDOMAR, CALIFORNIA

Jan 23

Figure B-13

APPENDIX



APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

**PROPOSED CHERRY OUTPOST
ASSESSOR'S PARCELS
366-290-007 & -008
NWC OF BUNDY CANYON AND
CHERRY STREET
WILDOMAR, CALIFORNIA**

PROJECT NO. T3010-22-01

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 $\frac{3}{4}$ 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 $\frac{3}{4}$ inch in maximum dimension. The quantity of fines shall be less than approximately 20 percent of the rock fill quantity.
- 3.2 Material of a perishable, spongy, or otherwise unsuitable nature as determined by the Consultant shall not be used in fills.
- 3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9

and 10; 40CFR; and any other applicable local, state or federal laws. The Consultant shall not be responsible for the identification or analysis of the potential presence of hazardous materials. However, if observations, odors or soil discoloration cause Consultant to suspect the presence of hazardous materials, the Consultant may request from the Owner the termination of grading operations within the affected area. Prior to resuming grading operations, the Owner shall provide a written report to the Consultant indicating that the suspected materials are not hazardous as defined by applicable laws and regulations.

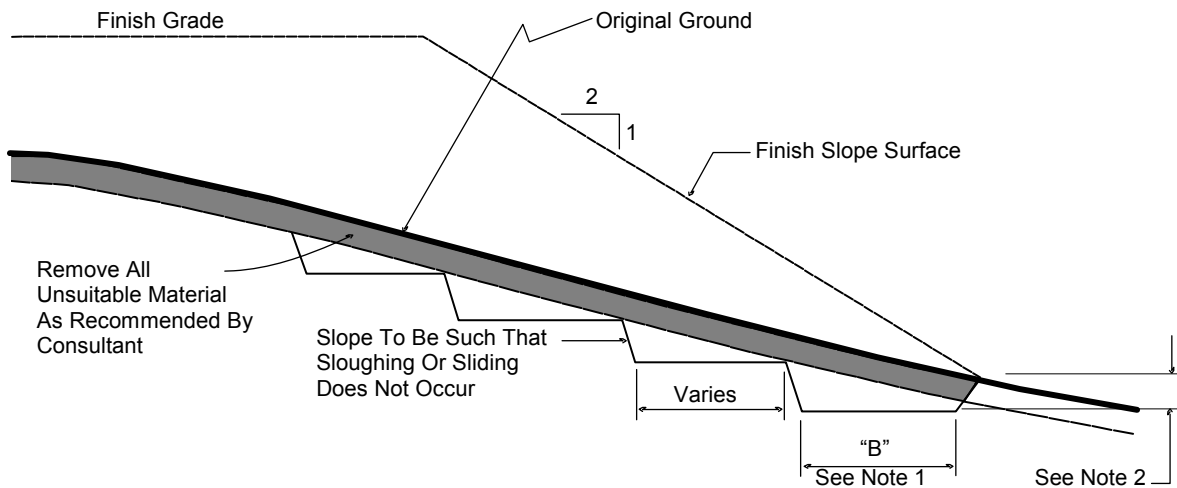
- 3.4 The outer 15 feet of *soil-rock* fill slopes, measured horizontally, should be composed of properly compacted *soil* fill materials approved by the Consultant. *Rock* fill may extend to the slope face, provided that the slope is not steeper than 2:1 (horizontal:vertical) and a soil layer no thicker than 12 inches is track-walked onto the face for landscaping purposes. This procedure may be utilized provided it is acceptable to the governing agency, Owner and Consultant.
- 3.5 Samples of soil materials to be used for fill should be tested in the laboratory by the Consultant to determine the maximum density, optimum moisture content, and, where appropriate, shear strength, expansion, and gradation characteristics of the soil.
- 3.6 During grading, soil or groundwater conditions other than those identified in the Geotechnical Report may be encountered by the Contractor. The Consultant shall be notified immediately to evaluate the significance of the unanticipated condition

4. CLEARING AND PREPARING AREAS TO BE FILLED

- 4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material and shall be performed in areas to be graded. Roots and other projections exceeding 1½ inches in diameter shall be removed to a depth of 3 feet below the surface of the ground. Borrow areas shall be grubbed to the extent necessary to provide suitable fill materials.
- 4.2 Asphalt pavement material removed during clearing operations should be properly disposed at an approved off-site facility or in an acceptable area of the project evaluated by Geocon and the property owner. Concrete fragments that are free of reinforcing steel may be placed in fills, provided they are placed in accordance with Section 6.2 or 6.3 of this document.

- 4.3 After clearing and grubbing of organic matter and other unsuitable material, loose or porous soils shall be removed to the depth recommended in the Geotechnical Report. The depth of removal and compaction should be observed and approved by a representative of the Consultant. The exposed surface shall then be plowed or scarified to a minimum depth of 6 inches and until the surface is free from uneven features that would tend to prevent uniform compaction by the equipment to be used.
- 4.4 Where the slope ratio of the original ground is steeper than 5:1 (horizontal:vertical), or where recommended by the Consultant, the original ground should be benched in accordance with the following illustration.

TYPICAL BENCHING DETAIL



No Scale

- DETAIL NOTES: (1) Key width "B" should be a minimum of 10 feet, or sufficiently wide to permit complete coverage with the compaction equipment used. The base of the key should be graded horizontal, or inclined slightly into the natural slope.
- (2) The outside of the key should be below the topsoil or unsuitable surficial material and at least 2 feet into dense formational material. Where hard rock is exposed in the bottom of the key, the depth and configuration of the key may be modified as approved by the Consultant.

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

5. COMPACTION EQUIPMENT

- 5.1 Compaction of *soil* or *soil-rock* fill shall be accomplished by sheepsfoot or segmented-steel wheeled rollers, vibratory rollers, multiple-wheel pneumatic-tired rollers, or other types of acceptable compaction equipment. Equipment shall be of such a design that it will be capable of compacting the *soil* or *soil-rock* fill to the specified relative compaction at the specified moisture content.
- 5.2 Compaction of *rock* fills shall be performed in accordance with Section 6.3.

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

- 6.1 *Soil* fill, as defined in Paragraph 3.1.1, shall be placed by the Contractor in accordance with the following recommendations:
- 6.1.1 *Soil* fill shall be placed by the Contractor in layers that, when compacted, should generally not exceed 8 inches. Each layer shall be spread evenly and shall be thoroughly mixed during spreading to obtain uniformity of material and moisture in each layer. The entire fill shall be constructed as a unit in nearly level lifts. Rock materials greater than 12 inches in maximum dimension shall be placed in accordance with Section 6.2 or 6.3 of these specifications.
- 6.1.2 In general, the *soil* fill shall be compacted at a moisture content at or above the optimum moisture content as determined by ASTM D 1557.
- 6.1.3 When the moisture content of *soil* fill is below that specified by the Consultant, water shall be added by the Contractor until the moisture content is in the range specified.
- 6.1.4 When the moisture content of the *soil* fill is above the range specified by the Consultant or too wet to achieve proper compaction, the *soil* fill shall be aerated by the Contractor by blading/mixing, or other satisfactory methods until the moisture content is within the range specified.
- 6.1.5 After each layer has been placed, mixed, and spread evenly, it shall be thoroughly compacted by the Contractor to a relative compaction of at least 90 percent. Relative compaction is defined as the ratio (expressed in percent) of the in-place dry density of the compacted fill to the maximum laboratory dry density as determined in accordance with ASTM D 1557. Compaction shall be continuous over the entire area, and compaction equipment shall make sufficient passes so that the specified minimum relative compaction has been achieved throughout the entire fill.

- 6.1.6 Where practical, soils having an Expansion Index greater than 50 should be placed at least 3 feet below finish pad grade and should be compacted at a moisture content generally 2 to 4 percent greater than the optimum moisture content for the material.
 - 6.1.7 Properly compacted *soil* fill shall extend to the design surface of fill slopes. To achieve proper compaction, it is recommended that fill slopes be over-built by at least 3 feet and then cut to the design grade. This procedure is considered preferable to track-walking of slopes, as described in the following paragraph.
 - 6.1.8 As an alternative to over-building of slopes, slope faces may be back-rolled with a heavy-duty loaded sheepsfoot or vibratory roller at maximum 4-foot fill height intervals. Upon completion, slopes should then be track-walked with a D-8 dozer or similar equipment, such that a dozer track covers all slope surfaces at least twice.
- 6.2 *Soil-rock* fill, as defined in Paragraph 3.1.2, shall be placed by the Contractor in accordance with the following recommendations:
- 6.2.1 Rocks larger than 12 inches but less than 4 feet in maximum dimension may be incorporated into the compacted *soil* fill, but shall be limited to the area measured 15 feet minimum horizontally from the slope face and 5 feet below finish grade or 3 feet below the deepest utility, whichever is deeper.
 - 6.2.2 Rocks or rock fragments up to 4 feet in maximum dimension may either be individually placed or placed in windrows. Under certain conditions, rocks or rock fragments up to 10 feet in maximum dimension may be placed using similar methods. The acceptability of placing rock materials greater than 4 feet in maximum dimension shall be evaluated during grading as specific cases arise and shall be approved by the Consultant prior to placement.
 - 6.2.3 For individual placement, sufficient space shall be provided between rocks to allow for passage of compaction equipment.
 - 6.2.4 For windrow placement, the rocks should be placed in trenches excavated in properly compacted *soil* fill. Trenches should be approximately 5 feet wide and 4 feet deep in maximum dimension. The voids around and beneath rocks should be filled with approved granular soil having a Sand Equivalent of 30 or greater and should be compacted by flooding. Windrows may also be placed utilizing an "open-face" method in lieu of the trench procedure, however, this method should first be approved by the Consultant.

- 6.2.5 Windrows should generally be parallel to each other and may be placed either parallel to or perpendicular to the face of the slope depending on the site geometry. The minimum horizontal spacing for windrows shall be 12 feet center-to-center with a 5-foot stagger or offset from lower courses to next overlying course. The minimum vertical spacing between windrow courses shall be 2 feet from the top of a lower windrow to the bottom of the next higher windrow.
- 6.2.6 Rock placement, fill placement and flooding of approved granular soil in the windrows should be continuously observed by the Consultant.
- 6.3 *Rock* fills, as defined in Section 3.1.3, shall be placed by the Contractor in accordance with the following recommendations:
- 6.3.1 The base of the *rock* fill shall be placed on a sloping surface (minimum slope of 2 percent). The surface shall slope toward suitable subdrainage outlet facilities. The *rock* fills shall be provided with subdrains during construction so that a hydrostatic pressure buildup does not develop. The subdrains shall be permanently connected to controlled drainage facilities to control post-construction infiltration of water.
- 6.3.2 *Rock* fills shall be placed in lifts not exceeding 3 feet. Placement shall be by rock trucks traversing previously placed lifts and dumping at the edge of the currently placed lift. Spreading of the *rock* fill shall be by dozer to facilitate *seating* of the rock. The *rock* fill shall be watered heavily during placement. Watering shall consist of water trucks traversing in front of the current rock lift face and spraying water continuously during rock placement. Compaction equipment with compactive energy comparable to or greater than that of a 20-ton steel vibratory roller or other compaction equipment providing suitable energy to achieve the required compaction or deflection as recommended in Paragraph 6.3.3 shall be utilized. The number of passes to be made should be determined as described in Paragraph 6.3.3. Once a *rock* fill lift has been covered with *soil* fill, no additional *rock* fill lifts will be permitted over the *soil* fill.
- 6.3.3 Plate bearing tests, in accordance with ASTM D 1196, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection

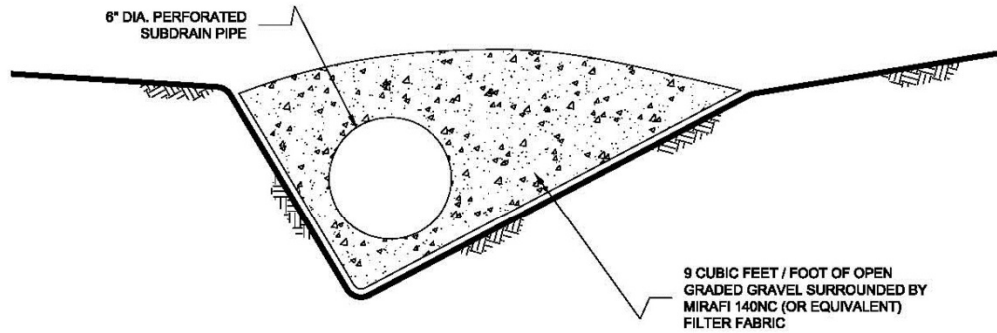
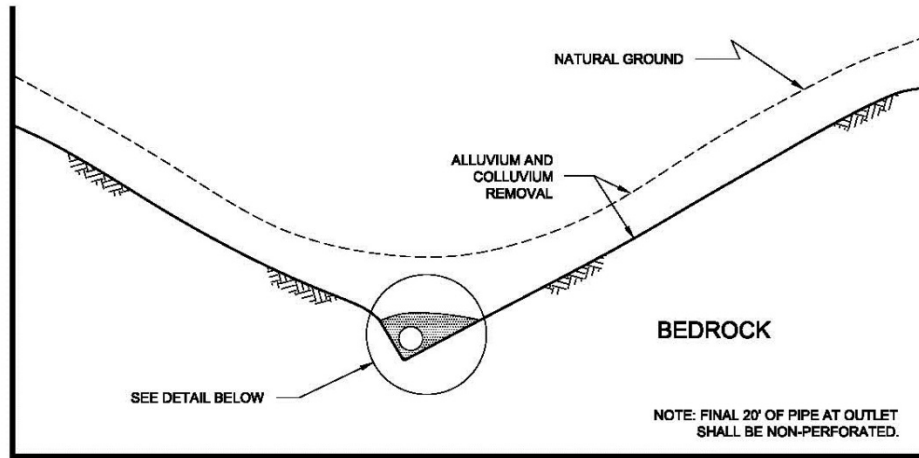
variation with number of passes. The required number of passes of the compaction equipment will be performed as necessary until the plate bearing deflections are equal to or less than that determined for the properly compacted *soil* fill. In no case will the required number of passes be less than two.

- 6.3.4 A representative of the Consultant should be present during *rock* fill operations to observe that the minimum number of “passes” have been obtained, that water is being properly applied and that specified procedures are being followed. The actual number of plate bearing tests will be determined by the Consultant during grading.
- 6.3.5 Test pits shall be excavated by the Contractor so that the Consultant can state that, in their opinion, sufficient water is present and that voids between large rocks are properly filled with smaller rock material. In-place density testing will not be required in the *rock* fills.
- 6.3.6 To reduce the potential for “piping” of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant prior to commencing grading. The gradation of the graded filter material will be determined at the time the *rock* fill is being excavated. Materials typical of the *rock* fill should be submitted to the Consultant in a timely manner, to allow design of the graded filter prior to the commencement of *rock* fill placement.
- 6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

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

TYPICAL CANYON DRAIN DETAIL



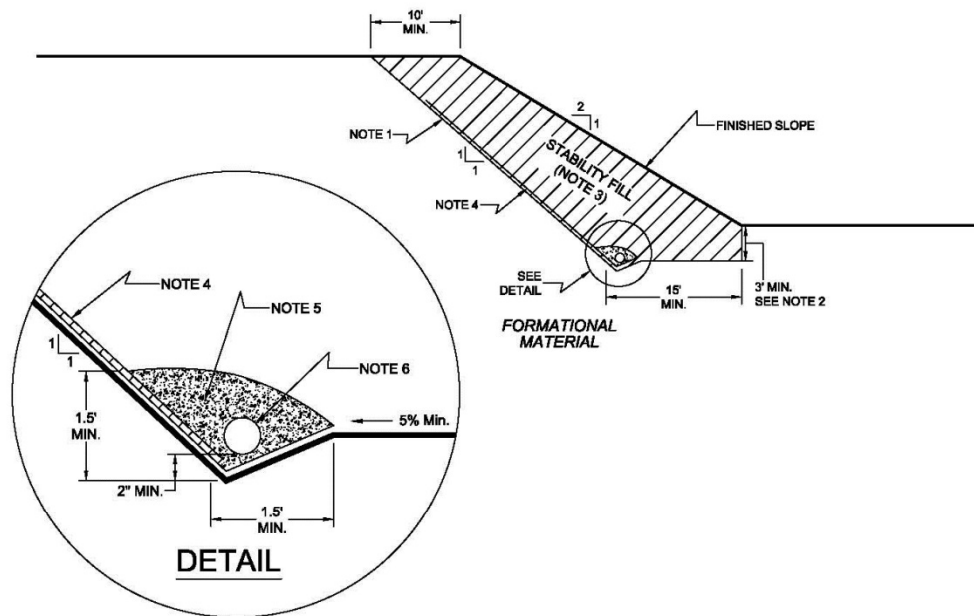
NOTES:

- 1.....8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.
- 2.....6-INCH DIAMETER, SCHEDULE 40 PVC PERFORATED PIPE FOR FILLS LESS THAN 100-FEET IN DEPTH OR A PIPE LENGTH SHORTER THAN 500 FEET.

NO SCALE

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

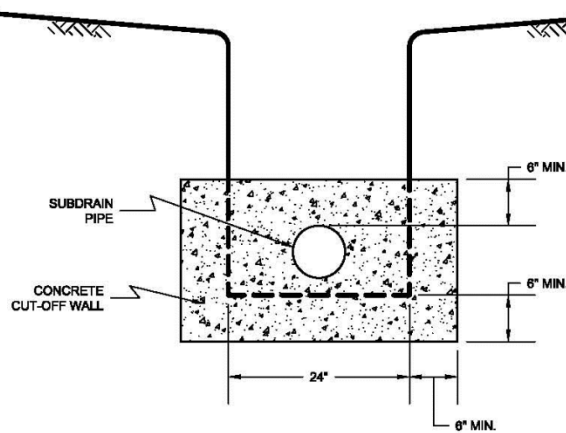
NO SCALE

- 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.

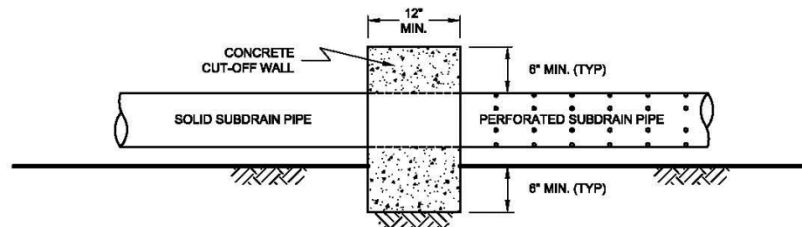
TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW

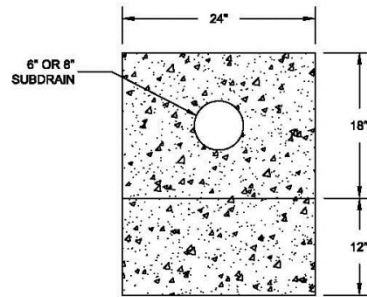


NO SCALE

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

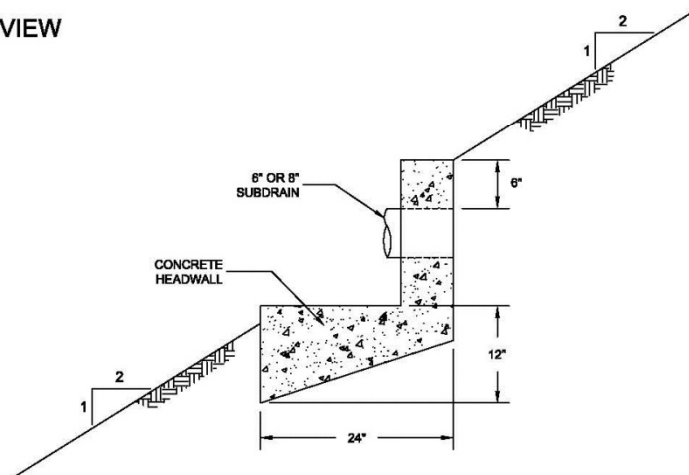
TYPICAL HEADWALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW



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

NO SCALE

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

8. OBSERVATION AND TESTING

- 8.1 The Consultant shall be the Owner's representative to observe and perform tests during clearing, grubbing, filling, and compaction operations. In general, no more than 2 feet in vertical elevation of *soil* or *soil-rock* fill should be placed without at least one field density test being performed within that interval. In addition, a minimum of one field density test should be performed for every 2,000 cubic yards of *soil* or *soil-rock* fill placed and compacted.
- 8.2 The Consultant should perform a sufficient distribution of field density tests of the compacted *soil* or *soil-rock* fill to provide a basis for expressing an opinion whether the fill material is compacted as specified. Density tests shall be performed in the compacted materials below any disturbed surface. When these tests indicate that the density of any layer of fill or portion thereof is below that specified, the particular layer or areas represented by the test shall be reworked until the specified density has been achieved.
- 8.3 During placement of *rock* fill, the Consultant should observe that the minimum number of passes have been obtained per the criteria discussed in Section 6.3.3. The Consultant should request the excavation of observation pits and may perform plate bearing tests on the placed *rock* fills. The observation pits will be excavated to provide a basis for expressing an opinion as to whether the *rock* fill is properly seated and sufficient moisture has been applied to the material. When observations indicate that a layer of *rock* fill or any portion thereof is below that specified, the affected layer or area shall be reworked until the *rock* fill has been adequately seated and sufficient moisture applied.
- 8.4 A settlement monitoring program designed by the Consultant may be conducted in areas of *rock* fill placement. The specific design of the monitoring program shall be as recommended in the Conclusions and Recommendations section of the project Geotechnical Report or in the final report of testing and observation services performed during grading.
- 8.5 We should observe the placement of subdrains, to check that the drainage devices have been placed and constructed in substantial conformance with project specifications.
- 8.6 Testing procedures shall conform to the following Standards as appropriate:

8.6.1 Soil and Soil-Rock Fills:

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

- 8.6.1.2 Field Density Test, Nuclear Method, ASTM D 6938, *Density of Soil and Soil-Aggregate In-Place by Nuclear Methods (Shallow Depth)*.
- 8.6.1.3 Laboratory Compaction Test, ASTM D 1557, *Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 10-Pound Hammer and 18-Inch Drop*.
- 8.6.1.4. Expansion Index Test, ASTM D 4829, *Expansion Index Test*.

9. PROTECTION OF WORK

- 9.1 During construction, the Contractor shall properly grade all excavated surfaces to provide positive drainage and prevent ponding of water. Drainage of surface water shall be controlled to avoid damage to adjoining properties or to finished work on the site. The Contractor shall take remedial measures to prevent erosion of freshly graded areas until such time as permanent drainage and erosion control features have been installed. Areas subjected to erosion or sedimentation shall be properly prepared in accordance with the Specifications prior to placing additional fill or structures.
- 9.2 After completion of grading as observed and tested by the Consultant, no further excavation or filling shall be conducted except in conjunction with the services of the Consultant.

10. CERTIFICATIONS AND FINAL REPORTS

- 10.1 Upon completion of the work, Contractor shall furnish Owner a certification by the Civil Engineer stating that the lots and/or building pads are graded to within 0.1 foot vertically of elevations shown on the grading plan and that all tops and toes of slopes are within 0.5 foot horizontally of the positions shown on the grading plans. After installation of a section of subdrain, the project Civil Engineer should survey its location and prepare an *as-built* plan of the subdrain location. The project Civil Engineer should verify the proper outlet for the subdrains and the Contractor should ensure that the drain system is free of obstructions.
- 10.2 The Owner is responsible for furnishing a final as-graded soil and geologic report satisfactory to the appropriate governing or accepting agencies. The as-graded report should be prepared and signed by a California licensed Civil Engineer experienced in geotechnical engineering and by a California Certified Engineering Geologist, indicating that the geotechnical aspects of the grading were performed in substantial conformance with the Specifications or approved changes to the Specifications.