Temecula Valley Wine Country Archway

Temecula Valley Area

Project No. D2-0111

Preliminary Geotechnical Investigation
Bridge Street Culvert Replacement
Dated July 13, 2023

Notice regarding this Report:

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PRELIMINARY GEOTECHNICAL INVESTIGATION

TEMECULA WINERY ARCHWAY
NORTHEAST OF RANCHO CALIFORNIA ROAD AND AVENIDA BIONA
TEMECULA VALLEY AREA OF RIVERSIDE COUNTY, CALIFORNIA

GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

PREPARED FOR

KOA CORPORATION
SAN DIEGO, CALIFORNIA

PROJECT NO. T2996-22-01
JANUARY 6, 2023
REVISED JULY 13, 2023

Project No. T2996-22-01
January 6, 2023
REVISED July 13, 2023

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Attention: Ali Shahzad, PE

Subject: PRELIMINARY GEOTECHNICAL INVESTIGATION
TEMECULA WINERY ARCHWAY
NORTHEAST OF RANCHO CALIFORNIA ROAD AND AVENIDA BIONA
TEMECULA VALLEY AREA OF RIVERSIDE COUNTY, CALIFORNIA

Mr. Shahzad:

In accordance with the *Agreement for Temecula Gateway Arch – Riverside County*, dated November 16, 2022, Geocon West, Inc. (Geocon) has prepared this preliminary geotechnical investigation report for the planned Temecula Winery Archway (Archway) structure, planned approximately 75 feet northeast of the intersection of Rancho California Road and Avenida Biona, in the Temecula Valley area of Riverside County, California. The accompanying geotechnical report presents our findings, conclusions, and recommendations pertaining to the geotechnical aspects of the proposed archway improvement. Based on the results of this investigation, it is our opinion that the geotechnical aspects of the site are suitable for the proposed Archway improvement, provided the recommendations of this report are followed.

This report is preliminary in nature. Geocon should be afforded the opportunity to review the project design plans throughout their development and provide revised and/or additional recommendations, as needed.

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

Very truly yours,

GEOCON WEST, INC.

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PRELIMINARY GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the findings of our preliminary geotechnical investigation for the planned Archway structure, proposed approximately 75 feet east of the intersection of Rancho California Road and Avenida Biona, in the Temecula Valley area of Riverside County, California, as depicted on the *Vicinity Map* (see Figure 1).

The purpose of this investigation was to evaluate the surface and subsurface soil conditions and general site geology, and to identify geotechnical constraints that may affect design and construction of the planned archway structure. Our scope of services included the following:

- Mark the boring locations and notify Underground Service Alert (USA) to locate and mark utilities within the investigation area.
- Obtain an encroachment permit from the Riverside County.
- Drill two geotechnical borings, one at each proposed archway support location. The borings were drilled to evaluate subsurface geologic conditions and to collect relatively undisturbed in-situ and disturbed bulk soil samples for laboratory testing. Borings were backfilled with soil cuttings and where in pavement, were capped with asphalt concrete and rig tamped. *Appendix A* presents the logs of our borings. The *Geologic Map*, Figure 2, presents the approximate location of our borings.
- Conduct laboratory testing of select soil samples to evaluate the engineering properties of site soils, which includes in-situ dry density and moisture content, maximum dry density and optimum moisture content, expansion, corrosion screening, and in-situ direct shears. *Appendix B* presents our laboratory test results.
- Prepare this preliminary geotechnical report, which presents our findings, conclusions and recommendations as they pertain to the geotechnical aspects of the proposed archway improvement.

2. SITE AND PROJECT DESCRIPTION

The proposed Archway structure will be located approximately 75 feet east of the intersection of Rancho California Road and Avenida Biona, within the Rancho California Road right-of-way, in the Temecula Valley area of Riverside County, California, at an approximate latitude of 33.5205 degrees and longitude of -117.0913 degrees. Currently, site elevations are approximately 1,265 and 1,264 feet above mean sea level (MSL), at the proposed north and south Archway support locations, respectively. References to elevations presented in this report are based on *Google Earth Pro* software (Google, Inc.). Geocon does not practice in the field of land surveying and is not responsible for the accuracy of such topographic information.

Based on the NETR Online *Historic Aerials* database and Google Earth Pro aerial imagery, Rancho California Road and a northern adjacent agricultural development were constructed sometime between 1967 and 1978 at the proposed Archway site. Additional agricultural developments were constructed surrounding the site between 1985 and 1996. The embankment south of Rancho California Road in proximity to the site was widened to accommodate a trail improvement between 2005 and 2006, and was widened again between 2019 and 2020.

Based on the *Temecula Winery Archway* preliminary site plan (undated), prepared by KOA Corporation (KOA), and the conceptual archway renders (undated), prepared by South Coast Lighting & Design, the proposed archway improvement will be aligned perpendicular to and span across Rancho California Road and will be 75 feet in length. Based on communication with KOA, we understand the proposed archway will be supported by cast-in-drilled-hole (CIDH) piles, one at each archway pedestal location. The diameter of the proposed pile foundations are unknown at this time, and as such, we have evaluated pile sizes ranging from 3 to 5 feet in diameter. Associated improvements are proposed to consist of a 36-inch by 36-inch section of concrete flatwork that will abut each pedestal, curb and gutter on the north side of Rancho California Road, and an electrical utility. We anticipate minor earthwork will be necessary for the construction of improvements, resulting in cuts and/or fills on the order of 1 foot or less (exclusive of remedial grading).

As the project progresses towards final design, project design plans should be provided to Geocon for review as they are developed, as changes in the design, location, or elevation of the proposed archway improvement may warrant revised and/or additional geotechnical recommendations.

3. FIELD EXPLORATION AND LABORATORY TESTING

Our field investigation was conducted on December 1, 2022, by drilling two 8-inch diameter geotechnical borings utilizing a truck-mounted hollow-stem auger drill rig in the vicinity of proposed improvements, with one boring advanced in the vicinity of each proposed archway support location. The borings were each drilled to depths of approximately 26 feet below the existing ground surface. We collected disturbed bulk samples from drill cuttings and relatively undisturbed samples by driving a 3-inch O. D. California Modified Sampler into the “undisturbed” soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by $2^{3}/_{8}$-inch inside diameter brass sampler rings to facilitate removal and testing. The in-situ and bulk soil samples collected were transported to our laboratory for testing. Borings were backfilled with soil cuttings and where in pavement, capped with asphalt concrete cold-patch and rig tamped. The approximate locations of the geotechnical borings are depicted on the *Geologic Map,* Figure 2. Detailed logs of the borings can be found in *Appendix A.*

Soil samples collected from our field investigation were transported to our laboratory for evaluation of their geotechnical engineering properties. Laboratory testing included in-situ dry density and moisture content, maximum dry density and optimum moisture content, corrosion screening, expansion, and in-situ direct shears. Appendix B contains the results of our laboratory testing.

4. GEOLOGIC MATERIALS

The geologic units encountered within the geotechnical borings consist of undocumented fill (afu), alluvium (Qal), and Pauba Sandstone (Qpfs). Geologic nomenclature follows that of Morton, D.M., Kennedy, M.P., Bovard, K.R., and Burns, D. (2003).

4.1 Undocumented Fill (afu)

Undocumented fill was encountered to depths of approximately 4 and 5 feet in Borings B-1 and B-2, respectively. This soil generally consists of silty sand, and is characterized as medium dense, slightly moist, and varies in shade of brown and reddish brown. Gravel was encountered at a depth of approximately 2½ feet within B-2.

4.2 Alluvium (Qal)

Alluvium was encountered below the undocumented fill in Boring B-2 to a depth of approximately 11½ feet. This soil generally consists of silty sand, and is characterized as medium dense, slightly moist, and varies in shades of brown.

4.3 Pauba Sandstone (Qpfs)

Pauba Sandstone was encountered underlying the undocumented fill and alluvium to the maximum depth explored of approximately 26 feet. This formational material generally consists of silty sandstone, and is generally characterized as dense to very dense and locally medium dense, slightly moist to moist, and varies in color. Gravel was encountered at a depth of approximately 12 feet within B-1 and at a depth of 19 feet within B-2.

4.4 Groundwater

Static groundwater or saturated soils were not encountered during our field exploration. California Department of Water Resources' *Water Data Library* indicates historic groundwater levels within 1 mile of the site were as shallow as approximately 94 feet below the ground surface (State Well Number 07S02W33E001S). It is not uncommon for seepage conditions to develop where none previously existed. A perched condition could occur as a result of the natural drainage located south of Rancho California Road. Static groundwater and seepage are dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. Proper surface drainage will be important to future performance of improvements.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 General

5.1.1 Neither soil nor geologic conditions were observed which would preclude the construction of the archway structure as presently proposed, provided that the recommendations of this preliminary geotechnical report are followed and implemented during design and construction.

5.1.2 Based on our subsurface exploration and the anticipated construction operations, we expect that undocumented fill, alluvium, and Pauba Sandstone will be exposed along the sidewalls and bottom of excavations. These materials may be subject to caving where materials exhibit low cohesion or cohesionless properties.

5.1.3 Deep foundations consisting of CIDH piles are appropriate for support of the archway structure. Deep foundations will exclusively derive support from side friction within the Pauba Sandstone. End-bearing is not considered in our evaluation. The undocumented fill left in place is not considered in evaluation of side frictional capacity. The use of casing may be necessary to support the integrity of the pile excavation.

5.1.4 Static groundwater and saturated soils were not encountered during the subsurface exploration of this investigation, which we do not expect will impact construction of the CIDH piles. It is not uncommon for seepage conditions to develop where none previously existed. A perched condition could occur as a result of the natural drainage located south of Rancho California Road. Static groundwater and seepage are dependent on seasonal precipitation, irrigation, land use, among other factors, and varies as a result. .

5.1.5 As the project progresses towards final design, project design plans should be provided to Geocon for review as they are developed, as changes in the design, location, or elevation of the proposed archway improvement may warrant revised and/or additional geotechnical recommendations.

5.2 Soil Characteristics

5.2.1 The undocumented fill and alluvium should generally be excavatable with moderate effort using conventional earth moving equipment in proper functioning order. Increased difficulty in excavation effort can occur in the Pauba Sandstone. Caving should be expected in unshored vertical excavations, especially where loose or cohesionless granular soils are encountered, and in larger diameter borings where arching effects are less prevalent.

5.2.2 It is the responsibility of the contractor's competent person to ensure that excavations are properly supported and maintained in accordance with applicable OSHA rules and regulations to maintain safety and the stability of adjacent existing improvements. Excavation recommendations are provided in *Temporary Excavations* section of this report.

5.2.3 Based on laboratory testing of select soil samples collected for evaluation of expansion potential, test results indicate site soils to be "non-expansive" (Expansion Index [EI] of 20 or less) as defined by 2022 California Building Code (CBC) Section 1803.5.3. Table 5.2.3 presents soil classifications based on the expansion index.

TABLE 5.2.3
SOIL CLASSIFICATION BASED ON EXPANSION INDEX

Expansion Index (EI)	Expansion Classification	2022 CBC Expansion Classification
0 – 20	**Very Low**	**Non-Expansive**
21 – 50	Low	Expansive
51 – 90	Medium	
91 – 130	High	
Greater Than 130	Very High	

5.2.4 Laboratory testing of select soil samples collected for evaluation of sulfate indicates that site soils possess a sulfate content of 0.003 percent (30 parts per million [ppm]), equating to a S0 sulfate exposure to concrete structures, as defined by 2022 CBC Section 1904.3 and ACI 318-19. Table 5.2.4 presents a summary of concrete requirements set forth by 2022 CBC Section 1904.3 and ACI 318-19. 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 5.2.4
REQUIREMENTS FOR CONCRETE
EXPOSED TO SULFATE-CONTAINING SOLUTIONS

Exposure Class	Water-Soluble Sulfate (SO4) Percent by Weight	Cement Type (ASTM C150)		Maximum Water to Cement Ratio by Weight[1]	Minimum Compressive Strength (psi)
S0	**SO_4<0.10**	**No Type Restriction**		**n/a**	**2,500**
S1	0.10≤SO_4<0.20	II		0.50	4,000
S2	0.20≤SO_4≤2.00	V		0.45	4,500
S3	SO_4>2.00	Option 1	V+Pozzolan or Slag	0.45	4,500
		Option 2	V	0.40	5,000

5.2.5 Laboratory testing of select soil samples was performed in accordance with Caltrans requirements and indicates site soils to contain a resistivity of 3,100 ohm-cm, a pH of 8.0, a chloride content of 40 ppm, and a sulfate content of 0.003 ppm. The site soils would not be classified as corrosive to metal improvements in accordance with Caltrans *Corrosion Guidelines* (Caltrans, 2021). Table 5.2.5 provides Caltrans' parameters defining a corrosive environment.

TABLE 5.2.5
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

5.2.6 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.

5.3 Grading

5.3.1 Grading should be performed in accordance with the *Recommended Grading Specifications* contained in Appendix C and the Grading Ordinances of Riverside County.

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

5.3.3 We expect that no cuts and fills or removal of existing improvements will be required; however, if needed, site preparation should begin with the removal of surface vegetation and existing improvements. 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. Rock over 6 inches in maximum dimension should be screened and removed, and not used in the engineered fill. Additionally, trash and/or other deleterious material encountered during earthwork should be screened and removed from the engineered fill, where encountered.

5.3.4 Based on our subsurface exploration, we expect undocumented fills at the proposed archway location to vary in thickness between 4 and 5 feet of depth; deeper sections of undocumented fill may exist beyond the locations explored. Due to the proposed CIDH pile foundation system deriving support in the Pauba Sandstone that underlies the undocumented fill and alluvium, it would be cost-prohibitive to do removals.

5.3.5 Fill and backfill soils, if needed, should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned at or slightly above optimum moisture content, and compacted to 90 percent relative compaction, as determined by ASTM D1557.

5.4 Seismic Design Criteria

5.4.1 Table 5.4.1 summarizes the site-specific design criteria obtained from the 2022 California Building Code (CBC; Based on the 2018 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the online application *Seismic Design Maps*, provided by OSHPD. The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2022 CBC and Table 20.3-1 of ASCE 7-16. The values presented below are for the risk-targeted maximum considered earthquake (MCE_R).

TABLE 5.4.1
2022 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2022 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.481g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.549g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.000	Table 1613.2.3(1)
Site Coefficient, F_V	1.751	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.481g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.442g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.987g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.962g*	Section 1613.2.4 (Eqn 16-39)

*Per Supplement 3 of ASCE 7-16, a ground motion hazard analysis (GMHA) shall be performed for projects on Site Class "D" sites with 1-second spectral acceleration (S_1) greater than or equal to 0.2g, which is true for this site. However, Supplement 3 of ASCE 7-16 provides an exception stating that that the GMHA may be waived provided that the parameter S_{M1} is increased by 50% for all applications of S_{M1}. The values for parameters S_{M1} and S_{D1} presented above have been increased in accordance with Supplement 3 of ASCE 7-16.

5.4.2 Table 5.4.2 presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

TABLE 5.4.2
ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.652g	Figure 22-9
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.717g	Section 11.8.3 (Eqn 11.8-1)

5.4.3 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 2022 California Building Code and ASCE 7-16, the MCE is to be utilized for the evaluation of liquefaction, lateral spreading, seismic settlements, and it is our understanding that the intent of the Building Code is to maintain “Life Safety” during a MCE event. The Design Earthquake Ground Motion (DE) is the level of ground motion that has a 10 percent chance of exceedance in 50 years, with a statistical return period of 475 years.

5.4.4 Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 6.86 magnitude event occurring at a hypocentral distance of 10.41 kilometers from the site.

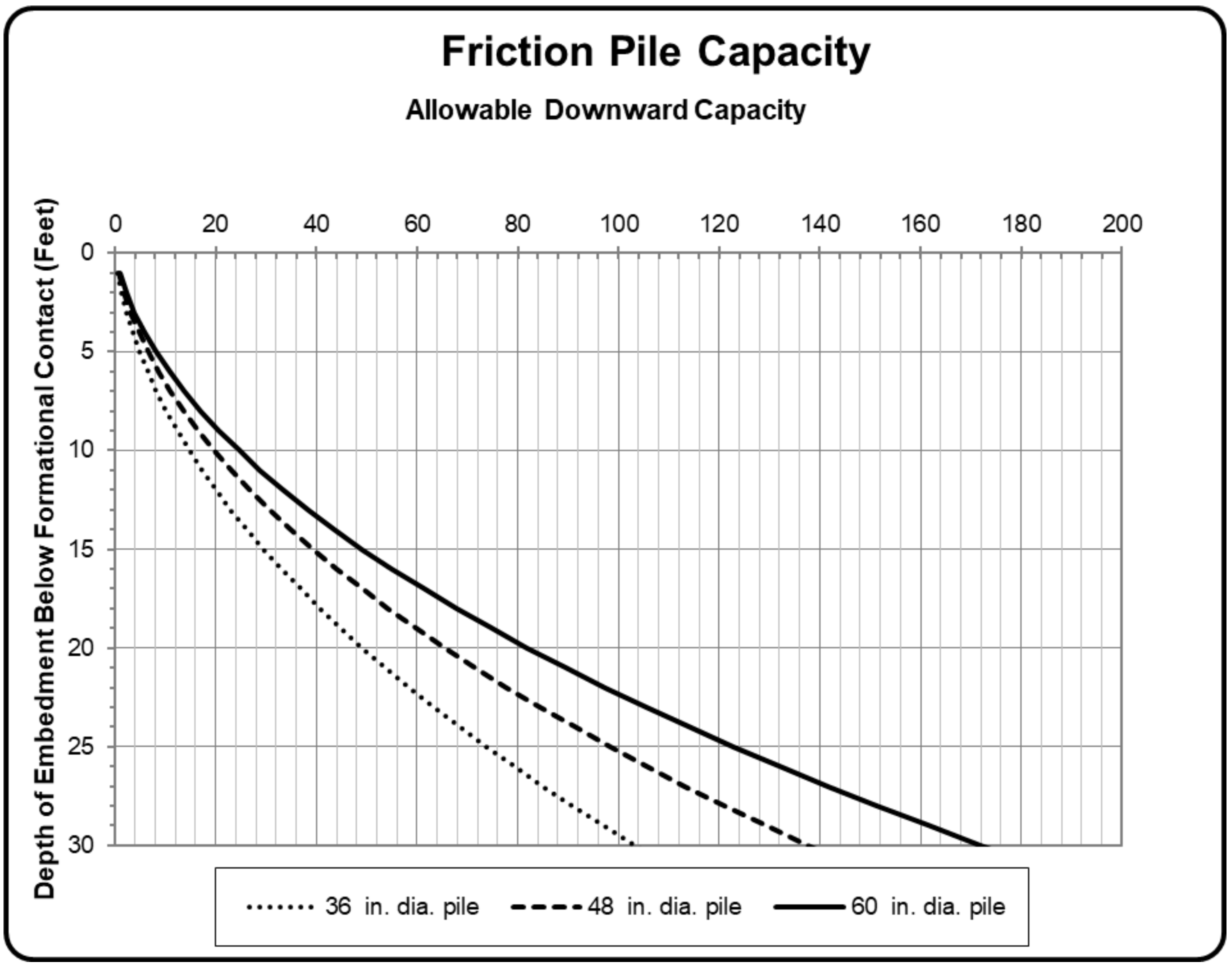
5.4.5 Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.75 magnitude occurring at a hypocentral distance of 16.81 kilometers from the site.

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

5.5 Cast-In-Drilled-Hole (CIDH) Piles

5.5.1 Cast-in-drilled-hole (CIDH) piles may be used for foundation support. The foundation recommendations herein are for CIDH piles and assume that the piles will extend through the undocumented fill and alluvium and will derive support in Pauba Sandstone material.

5.5.2 CIDH piles may be designed to derive support by side friction within the Pauba Sandstone, which the geologic contact was observed within Borings B-1 and B-2 at depths of approximately 4 and 11½ feet, respectively. The frictional capacity can be determined by the Friction Pile Capacity chart below. These allowable values possess a factor of safety of at least 2 for side friction. Based on the lack of liquefiable material at the proposed archway site, downdrag as a result of soil subsidence is not expected to pose a significant risk to the archway improvement and is therefore not considered in our pile capacity analysis.



5.5.3 The CIDH piles can be designed to develop support by side friction within the Pauba Sandstone using the design parameters presented in Table 5.5.3.

TABLE 5.5.3
SUMMARY OF CIDH PILE RECOMMENDATIONS

Parameter	Value
Minimum Pile Diameter	3 Feet
Minimum Pile Spacing	[1]N/A
[2]Minimum Foundation Embedment	5 Feet into Pauba Sandstone Geologic Contact
Allowable End-Bearing Capacity	Per Side Friction Capacity Chart
Estimated Total Settlement	1 Inch
Estimated Differential Settlement	½ Inch Across the Length of the Structure

[1]We expect one CIDH pile will be required to support each end of the archway.
[2]Pile embedment should be deepened by the project structural engineer to achieve required capacity for structural design.

5.5.4 Single pile uplift capacity can be taken as 75 percent of the allowable downward skin friction capacity based on a friction of 175 psf.

5.5.5 The design tip elevation of the CIDH piles should be determined by the project structural engineer based on pile capacity requirements.

5.5.6 During drilling operations, casing may be required to maintain the integrity of the pile excavation near finish grade elevation.

5.5.7 Due to the relatively dense in-situ condition of the Pauba Sandstone, the drilling contractor should anticipate difficult drilling conditions during pile excavations. Concrete should be placed within the excavation as soon as possible after drilling to reduce the potential for discontinuities or caving.

5.5.8 The maximum expected static settlement for the proposed archway structure supported on CIDH piles is estimated to be less than 1 inch. Differential settlement between CIDH pile foundations in Pauba Sandstone is not expected to exceed ½ inch. The majority of the foundation settlement is expected to occur on initial application of loading and during construction.

5.6 Lateral Design

5.6.1 Where a pile cap is used, resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.40 may be used with the dead load forces in engineered fill and undisturbed Pauba Sandstone.

5.6.2 Passive earth pressure for the sides of foundations poured against properly compacted engineered fill may be computed as an equivalent fluid having a density of 295 pounds per cubic foot with a maximum earth pressure of 2,950 pounds per square foot. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

5.6.3 Lateral capacities for ¼ inch deflection of fixed and free-head CIDH piles are presented in the table below. No factors of safety have been applied to the lateral load values calculated to induce ¼-inch lateral deflection. Lateral capacities provided are for 36-, 48-, and 60-inch diameter drilled CIDH piles, penetrating the earth materials encountered during the course of this investigation. Assumed as part of these lateral capacity calculations are a concrete modulus of elasticity of at least 3,000,000 psi. If more detailed lateral pile capacity information is needed, Geocon should be contacted.

LATERAL LOAD CAPACITIES OF DRILLED CAST-IN-PLACE PILES

FIXED HEAD (NO HEAD ROTATION)

PILE NUMBER	PILE DIAMETER (INCHES)	Lateral Load Capacity "P" (KIPS)	Maximum Positive Moment "Mp" (LAT FORCE =P)	Maximum Negative Moment "Mp" (LAT FORCE =P)	Depth to Max Pos. Moment (Feet)	Depth to Zero Moment (Feet)	Depth to Inflection Point (Feet)	MINIMUM PILE LENGTH FOR APPLICABILITY OF LATERAL DESIGN DATA (FEET)
1	36	**76**	**0.1** P	**-7.6** P	**14**	**14**	**13.6**	**14**
2	48	**120**	**0.1** P	**-9.5** P	**17**	**17**	**17.1**	**17**
3	60	**171**	**0.1** P	**-11.4** P	**20**	**20**	**20.5**	**20**

FREE HEAD (HINGED)

PILE NUMBER	PILE DIAMETER (INCHES)	Lateral Load Capacity "P" (KIPS)	Maximum Moment "Mp" (LAT FORCE =P)	Depth to Zero Moment (Feet)	Depth to Maximum Moment (Feet)
1	36	**28**	**3.8** P	**14**	**6**
2	48	**44**	**4.8** P	**18**	**8**
3	60	**63**	**5.7** P	**22**	**9**

Lateral capacities are based on 1/4-inch deflection.
Moment magnitudes are presented as a function of the applied lateral load "P".
"P" is entered in units of kips and the moment magnitude will be in units of kip-feet.
The maximum negative moment is at the rigid, pile to pile cap or grade beam connection at the top of the pile.

5.7 Exterior Concrete Flatwork

5.7.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in Table 5.7.1. The recommended steel reinforcement would help reduce the potential for cracking.

TABLE 5.7.1
MINIMUM CONCRETE FLATWORK RECOMMENDATIONS

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

*In excess of 8 feet square.

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

5.7.3 Even with the incorporation of the recommendations of this report, exterior concrete flatwork has a potential to experience some uplift due to expansive soil beneath grade, if present. The reinforcing 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, where applicable.

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

5.7.5 Where flatwork abuts the archway pedestals, the flatwork should be dowelled into the stemwall. This recommendation is intended to reduce the potential for differential elevations that could result from differential settlement or minor heave of the flatwork. Dowelling details should be designed by the project structural engineer.

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

5.8 Temporary Excavations

5.8.1 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/or trenches are properly constructed and maintained in accordance with applicable OSHA guidelines in order to maintain safety and the stability of the excavations and adjacent improvements. These excavations should not be allowed to become saturated or to dry out. Surcharge loads should not be permitted to a distance equal to the height of the excavation from the top of the excavation. The top of the excavation should be a minimum of 15 feet from the edge of existing improvements. Excavations steeper than those permitted by OSHA guidelines or closer than 15 feet from an existing surface improvement should be shored in accordance with applicable OSHA codes and regulations.

5.9 Site Drainage and Moisture Protection

5.9.1 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 foundations. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2022 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. Drainage should be directed into conduits that carry runoff away from the proposed structure.

5.9.2 Irrigation lines should be checked periodically for leaks and detected leaks should be repaired promptly. Detrimental soil movement could occur if water is allowed to infiltrate the soil for prolonged periods of time.

5.9.3 Landscaping planters adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate pavement subgrade and base course in vicinity of the proposed improvement. We recommend that area drains 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 pavement, we recommend construction of a cutoff wall or use of an impermeable geosynthetic material along the edge of pavements that extends at least 6 inches below the bottom of base material sections.

5.10 Plan Review

5.10.1 Geocon should be provided the opportunity to review the design plans prior to final submittal to verify substantial conformance with the recommendations of this report.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon 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.

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

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

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, 2011, *Building Code Requirements for Structural Concrete*, Report by ACI Committee 318.

2. California Building Standards Commission, 2022, *California Building Code (CBC)*, California Code of Regulations Title 24, Part 2.

3. California Department of Transportation (Caltrans), Division of Engineering Services, Materials Engineering and Testing Services, *Corrosion Guidelines*, *Version 3.2*, dated May 2021.

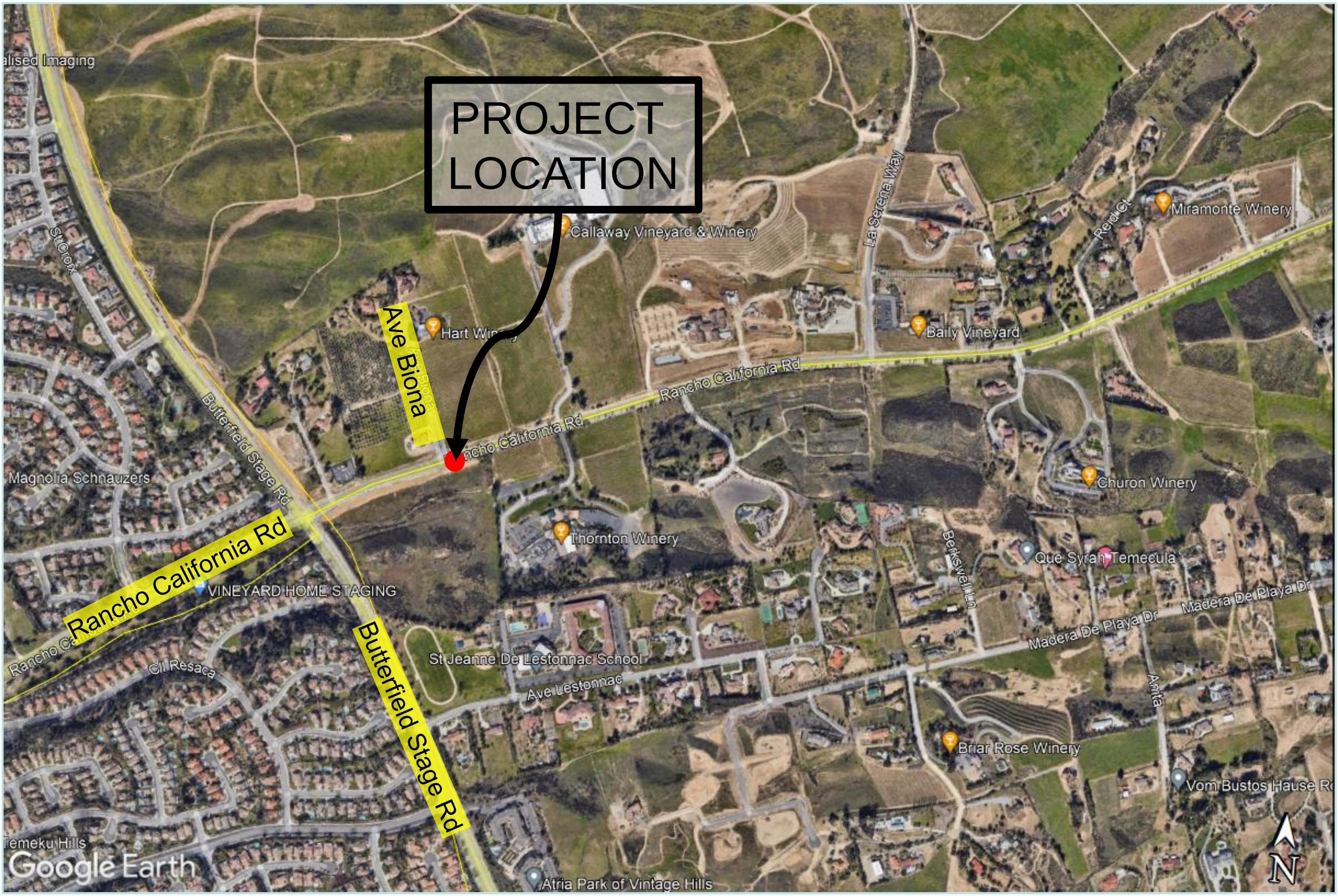
4. Morton, D.M., Kennedy, M.P., Bovard, K.R., and Burns, Diane, 2003, *Geologic Map of the Bachelor Mountain 7.5' Quadrangle, Riverside County, California*, U.S. Geological Survey, Open-File Report OF-2003-103, 1:24,000.

5. OSHPD, *Seismic Design Maps*, seismicmaps.org, accessed December 2022.

6. Public Works Standards, Inc., 2021, *Standard Specifications for Public Works Construction "Greenbook,"* Published by BNi Building News.

7. U.S. Geological Survey (USGS), *Deaggregation of Seismic Hazard for PGA and 2 Periods of Spectral Acceleration*, 2002, USGS Website: www.earthquake.usgs.gov/research/hazmaps.3

8. USGS, Unified Hazard Tool, https://earthquake.usgs.gov/hazards/interactive/, accessed December 2022



0' 1000' 2000'

SCALE: 1" = 1000'

N

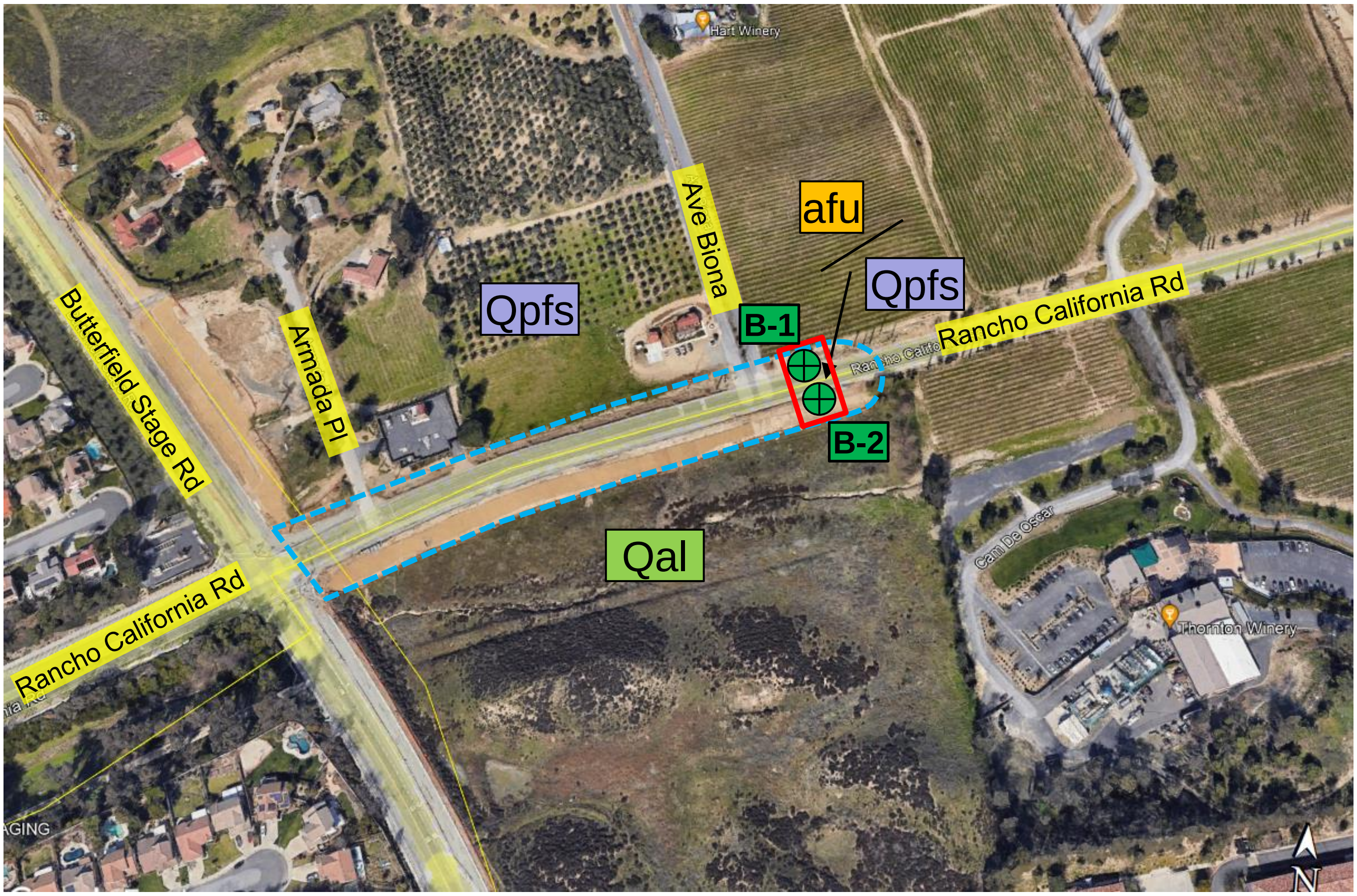
SOURCE: Google Earth, 2022

VICINITY MAP

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

TEMECULA WINERY ARCHWAY
NORTHEAST OF RANCHO CALIFORNIA ROAD
AND AVENIDA BIONA
TEMECULA VALLEY AREA OF
RIVERSIDE COUNTY, CALIFORNIA

ARK/ATS			JULY 2023	PROJECT NO. T2996-22-01	FIG. 1



GEOCON LEGEND

Locations are approximate

B-1 .... APPROXIMATE BORING LOCATION

.... PROJECT EXTENTS

.... PROJECT EXTENTS

afu .... UNDOCUMENTED FILL

Qal .... ALLUVIUM
(.... Line Above Symbol
Indicates Contact is Buried)

Qpfs .... PAUBA SANDSTONE
(.... Line Above Symbol
Indicates Contact is Buried)

0' 500' 1000'

SCALE: 1" = 500'

N

SOURCE: Google Earth, 2022

GEOLOGIC MAP

GEOCON
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RIVERSIDE COUNTY, CALIFORNIA

ARK/ATS			JULY 2023	PROJECT NO. T2996-22-01	FIG 2

APPENDIX A

Geotechnical Report - Temecula Valley Wine Country Archway, Page 22 of 49

APPENDIX A

FIELD INVESTIGATION

Our field investigation was conducted on December 1, 2022 by drilling two 8-inch diameter geotechnical borings utilizing a truck-mounted hollow-stem auger drill rig in the vicinity of proposed improvements, with one boring advanced in the vicinity of each proposed archway support location. The borings were each drilled to depths of approximately 26 feet below the existing ground surface. We collected disturbed bulk samples from drill cuttings and relatively undisturbed samples by driving a 3-inch O. D. California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch high by 2³/₈-inch inside diameter brass sampler rings to facilitate removal and testing. The in-situ and bulk soil samples collected were transported to our laboratory for testing. Borings were backfilled with soil cuttings and where in pavement, capped with asphalt concrete cold-patch and rig tamped.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). Figures A-1 and A-2 present detailed logs of our geotechnical borings. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. *Geologic Map*, Figure 2 indicates the approximate location of our borings.

PROJECT NO. T2996-22-01

BORING B-1

ELEV. (MSL.) **1265** DATE COMPLETED **12/1/2022**

EQUIPMENT **CME 75** BY: **L. WEIDMAN**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					**PAVEMENT SECTION** 6" Asphalt			
				SM	**UNDOCUMENTED FILL (afu)** Silty SAND, medium dense, slightly moist, dark brown; fine to coarse sand			
2	B-1@2.5'				-Becomes reddish brown	48	131.7	5.6
4					**PAUBA FORMATION (Qpfs)** Silty SAND, dense, moist, yellowish brown; medium to coarse sand; little fine sand; slightly oxidized; micaceous			
6	B-1@5'				-Becomes brown; rootlets	73	116.9	5.8
	B-1@7-12'				-Becomes medium dense, moist; few clay lenses			
8	B-1@7.5'					44	114.2	17.6
10	B-1@10'			SM		72	109.5	7.4
12								
14								
16	B-1@15'				- Becomes brown; fine to coarse sand	55	118.6	13.7
18								
20	B-1@20'				- Becomes very dense, reddish brown; coarse sand dominant	90/9"	125.6	12.6
22					- Becomes dark, reddish brown and olive brown; medium sand dominant; little coarse and fine sand			
24	B-1@25'					88/11"	120.9	9.1

Total Depth = 25'11"
Groundwater not encountered
Penetration resistance for 140-lb hammer falling 30 inches by auto hammer
Backfilled with cuttings and capped with AC patch 12/1/2022

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

T2996-22-01 BORING LOGS.GPJ

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

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

GEOCON

PROJECT NO. T2996-22-01

BORING B-2

ELEV. (MSL.) **1262** DATE COMPLETED **12/1/2022**

EQUIPMENT **CME 75** BY: **L. WEIDMAN**

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT.)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	B-2@0-5'			SM	**UNDOCUMENTED FILL (afu)** Silty SAND, medium dense, slightly moist, brown; fine to medium sand; some coarse sand			
2	B-2@2.5'				-Few fine gravel	25	118.1	5.6
4								
	B-2@5'			SM	**ALLUVIUM (Qal)** Silty SAND, medium dense, slightly moist, light brown to brown; little oxidized grains	25	119.8	3.8
6					-Few mica; few pores			
8	B-2@7.5'					32	125.1	6.1
10	B-2@10'					30	116.9	4.8
12				SM	**PAUBA FORMATION (Qpfs)** Silty SAND, medium dense, slightly moist, reddish to yellowish brown; fine to medium sand; little coarse sand; few fine gravel			
14								
16	B-2@15'				- Becomes dense, moist, strong brown; no gravel; trace clay development; laminations	60	115.7	14.1
18								
20	B-2@20'				- Becomes reddish brown and olive brown; some coarse sand; moderately oxidized; large biotite; trace manganese staining; massive	73	134.0	6.9
22								
24								
26	B-2@25'				- Becomes very dense, strong brown; micaceous; lenses of fine sand	71/12"	118.8	15.3

Total Depth = 26'
Groundwater not encountered
Penetration resistance for 140-lb hammer falling 30 inches by auto hammer
Backfilled with cuttings on 12/1/2022

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

T2996-22-01 BORING LOGS.GPJ

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

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

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APPENDIX B

Geotechnical Report - Temecula Valley Wine Country Archway, Page 26 of 49

APPENDIX B

LABORATORY TESTING

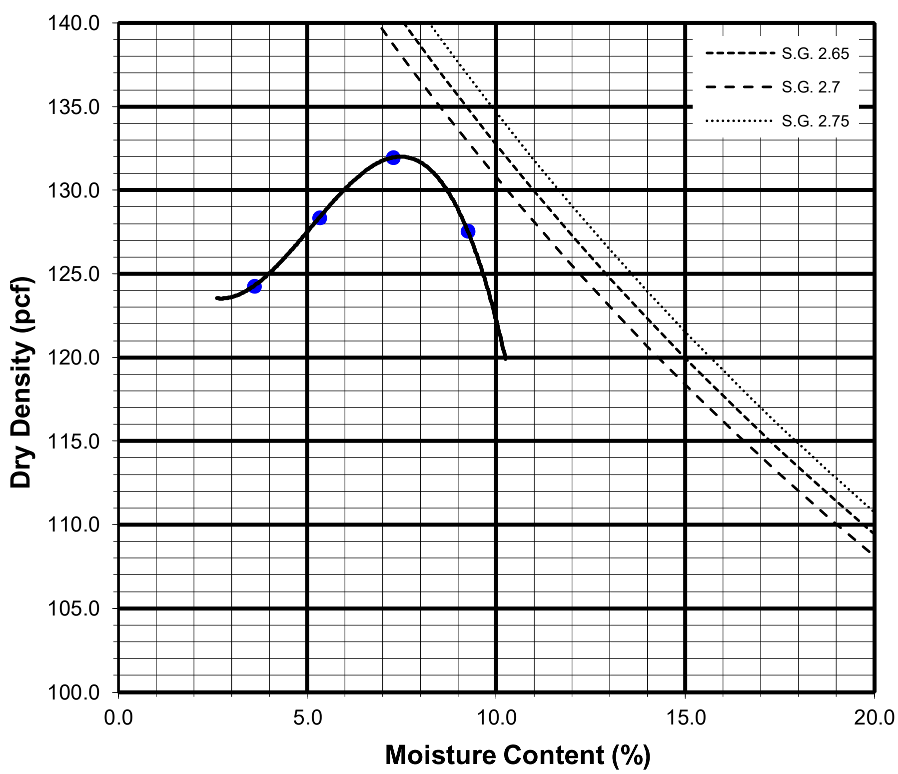
Laboratory tests were performed in general accordance with test methods of ASTM International (ASTM), Caltrans test methods, or other suggested procedures. Selected samples were tested to evaluate in-situ dry density and moisture content, maximum dry density and optimum moisture content, corrosion, expansion, and in-situ direct shear. The results of our laboratory testing program are presented on Figures B-1 through B-6. The in-situ dry density and moisture content of the samples tested are presented on the boring logs in *Appendix A*.

Sample No:

B2@0-5'	Brown Silty Sand (SM)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6167	6264	6360	6327		
Weight of Mold	(g)	4222	4222	4222	4222		
Net Weight of Soil	(g)	1945	2042	2138	2105		
Wet Weight of Soil + Cont.	(g)	687.6	754.8	783.8	695.8		
Dry Weight of Soil + Cont.	(g)	670.1	726.0	743.2	652.4		
Weight of Container	(g)	184.2	184.6	184.4	183.0		
Moisture Content	(%)	3.6	5.3	7.3	9.2		
Wet Density	(pcf)	128.8	135.2	141.5	139.4		
Dry Density	(pcf)	124.3	128.4	132.0	127.6		

Maximum Dry Density (pcf)	**132.0**	**Optimum Moisture Content (%)**	**7.0**



Preparation Method: A

GEOCON	**COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS** ASTM D-1557	Project No.: T2996-22-01
		Temecula Winery Archway Northeast of Rancho California Rd and Ave Biona Temecula Valley Area of Riverside County, California
	Checked by: JTA	July 2023 Figure B-1

B2@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)	782.4	798.2
Wt. of Mold	(gm)	376.2	376.2
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	515.8	798.2
Dry Wt. of Soil + Cont.	(gm)	491.0	372.7
Wt. of Container	(gm)	215.8	376.2
Moisture Content	(%)	9.0	13.2
Wet Density	(pcf)	122.5	127.1
Dry Density	(pcf)	112.4	112.3
Void Ratio		0.5	0.5
Total Porosity		0.3	0.3
Pore Volume	(cc)	69.0	68.0
Degree of Saturation	(%) [S_{meas}]	49.0	72.5

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
12/27/2022	10:00	1.0	0	0.2622
12/27/2022	10:10	1.0	10	0.2617
Add Distilled Water to the Specimen				
12/28/2022	10:00	1.0	1430	0.2571
12/28/2022	11:00	1.0	1490	0.2571

Expansion Index (EI meas) =	-4.6
Expansion Index (Report) =	**0**

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

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

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EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: ATS

Project No.: T2996-22-01

Temecula Winery Archway
Northeast of Rancho California Rd and Ave Biona
Temecula Valley Area of Riverside County, California

July 2023 Figure B-2

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)
B1@7-12'	8.0	3100

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
AASHTO T291 ASTM C1218

Sample No.	Chloride Ion Content (%)
B1@7-12'	0.004

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure
B1@7-12'	0.003	S0

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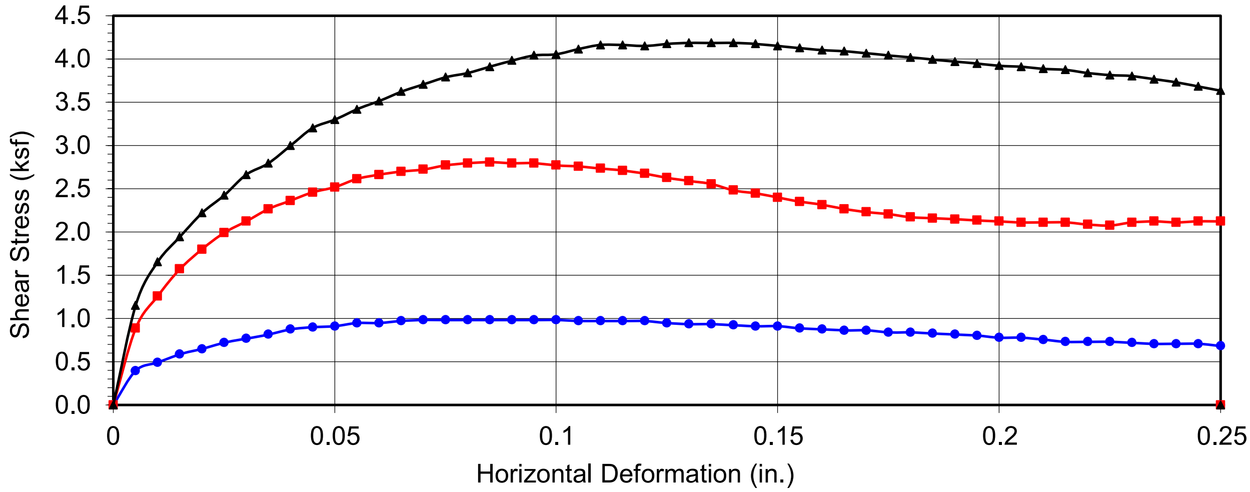
CORROSIVITY TEST RESULTS

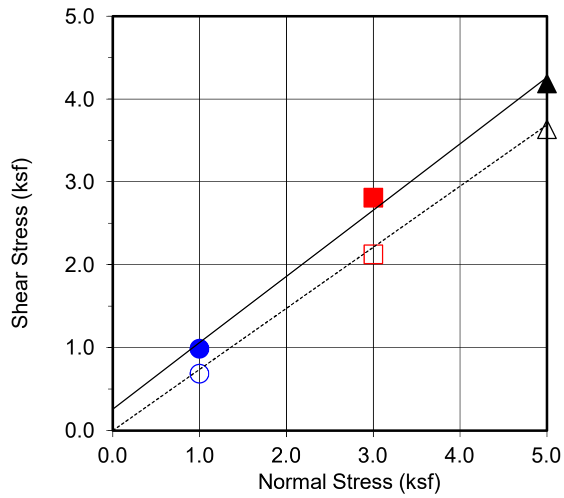
Checked by:

Project No.: T2996-22-01

Temecula Winery Archway
Northeast of Rancho California Rd and Ave Biona
Temecula Valley Area of Riverside County, California

July 2023 Figure B-3





Boring No.	**B1**
Sample No.	**B1@10'**
Depth (ft)	**10'**
Sample Type:	Ring

Soil Identification:		
Light Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	257	39
Ultimate	0	36

Normal Strest (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.98	■ 2.81	▲ 4.19
Shear Stress @ End of Test (ksf)	○ 0.68	□ 2.12	△ 3.64
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 (%)	7.4	7.1	7.1
Initial Dry Density (pcf)	108.6	110.3	105.9
Initial Degree of Saturation (%)	36.2	36.1	32.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.4	17.1	18.1

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DIRECT SHEAR TEST RESULTS

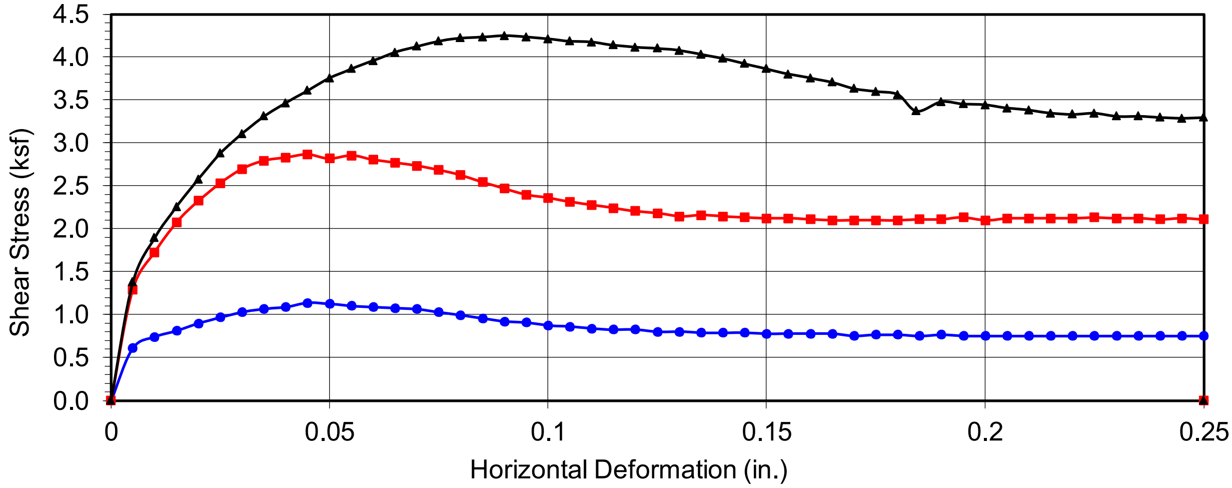
Consolidated Drained ASTM D-3080

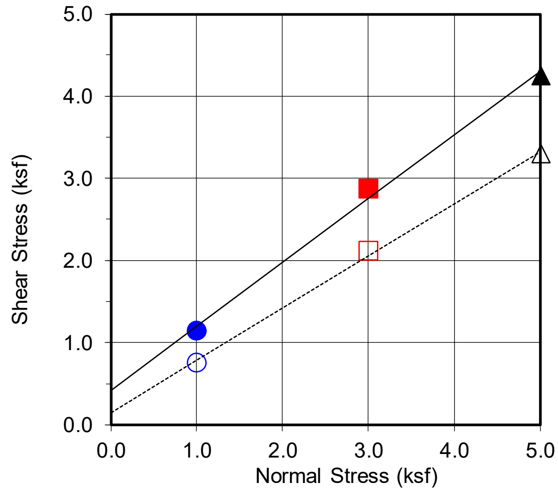
Checked by: ATS

Project No.: T2996-22-01

Temecula Winery Archway
Northeast of Rancho California Rd and Ave Biona
Temecula Valley Area of Riverside County, California

July 2023 Figure B-4





Boring No.	**B1**
Sample No.	**B1@15'**
Depth (ft)	**15**
Sample Type:	Ring

Soil Identification:		
Brown Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	421	38
Ultimate	148	32

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.14	■ 2.87	▲ 4.25
Shear Stress @ End of Test (ksf)	○ 0.76	□ 2.11	△ 3.30
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	13.5	13.7
Initial Dry Density (pcf)	114.3	119.0	120.7
Initial Degree of Saturation (%)	75.4	87.6	93.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.4	17.5	17.9

GEOCON

DIRECT SHEAR TEST RESULTS

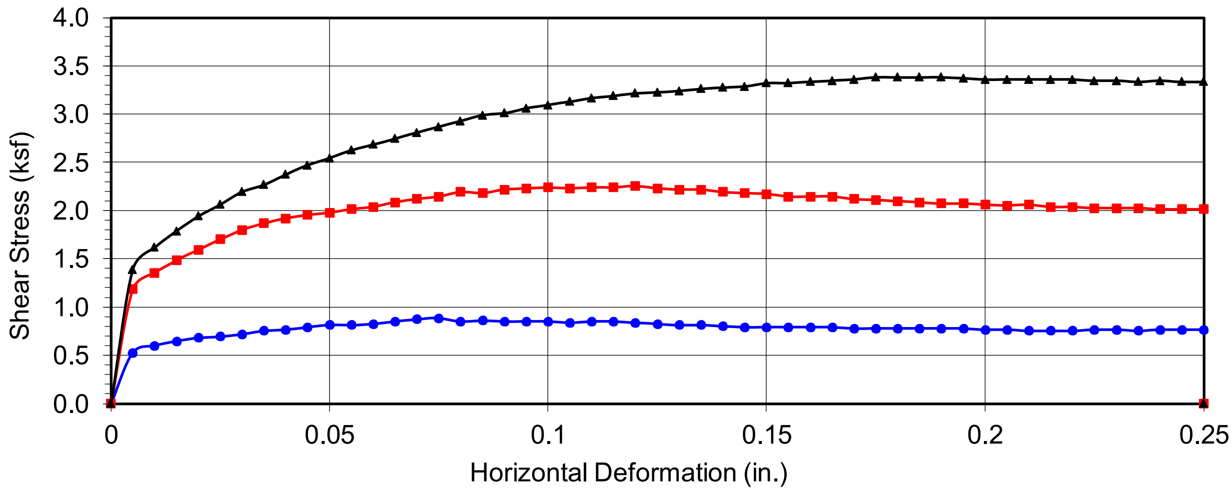
Consolidated Drained ASTM D-3080

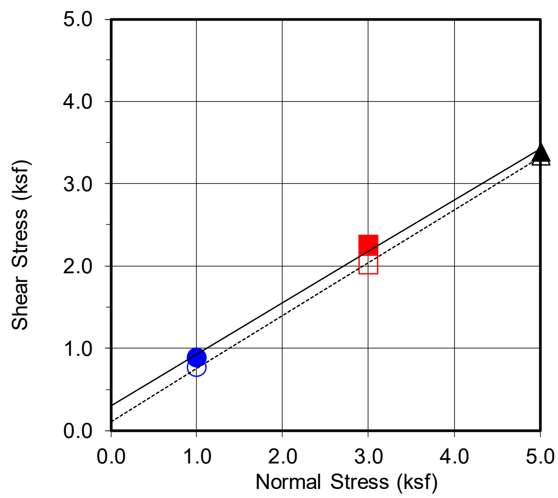
Checked by:

Project No.: T2996-22-01

Temecula Winery Archway
Northeast of Rancho California Rd and Ave Biona
Temecula Valley Area of Riverside County, California

July 2023 Figure B-5





Boring No.	**B2**
Sample No.	**B2@15'**
Depth (ft)	**15**
Sample Type:	Ring

Soil Identification:
Strong Brown Silty Sand (SM)

Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	304	32
Ultimate	114	33

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.89	■ 2.26	▲ 3.38
Shear Stress @ End of Test (ksf)	○ 0.77	□ 2.02	△ 3.34
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 (%)	12.8	16.3	14.1
Initial Dry Density (pcf)	116.3	109.4	120.9
Initial Degree of Saturation (%)	77.1	81.2	96.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.7	18.3	17.3

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by:

Project No.: T2996-22-01

Temecula Winery Archway
Northeast of Rancho California Rd and Ave Biona
Temecula Valley Area of Riverside County, California

July 2023 Figure B-6

APPENDIX C

Geotechnical Report - Temecula Valley Wine Country Archway, Page 34 of 49

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

TEMECULA WINERY ARCHWAY
NORTHEAST OF RANCHO CALIFORNIA ROAD
AND AVENIDA BIONA
TEMECULA VALLEY AREA OF
RIVERSIDE COUNTY, CALIFORNIA

PROJECT NO. T2996-22-01

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

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

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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

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

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

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

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

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

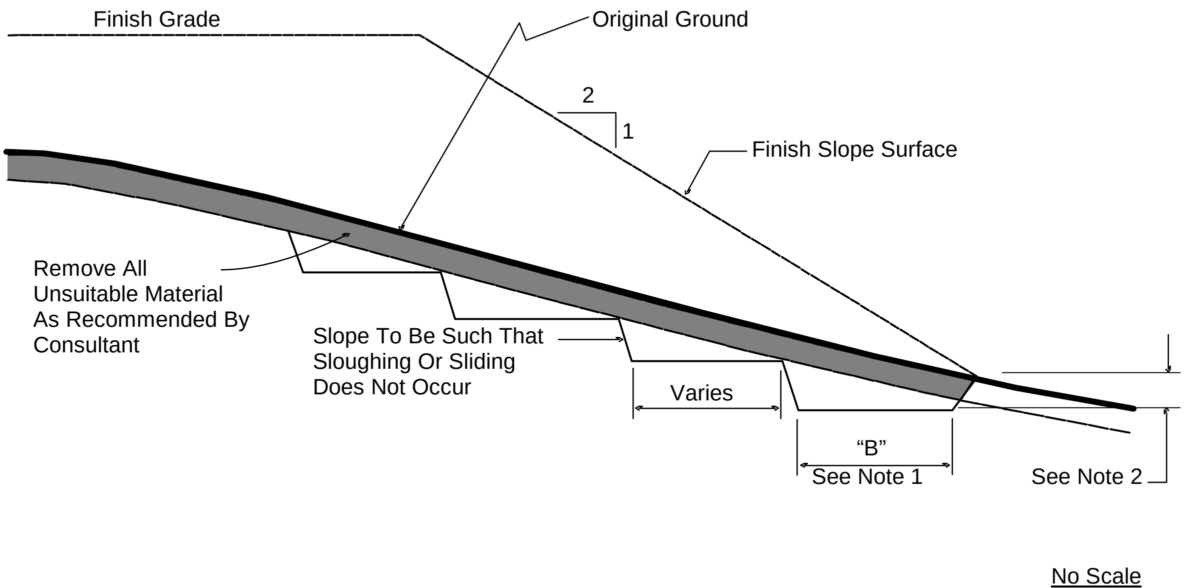
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

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

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

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

TYPICAL BENCHING DETAIL



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

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

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

5. COMPACTION EQUIPMENT

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

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

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

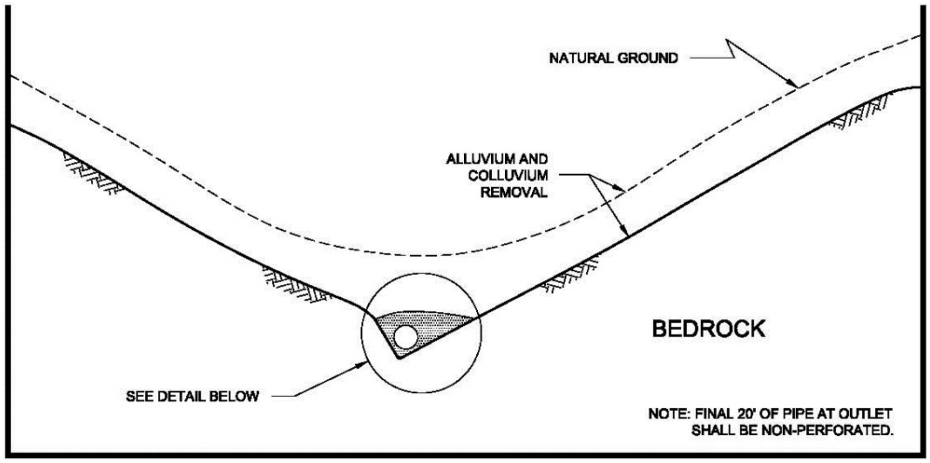
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

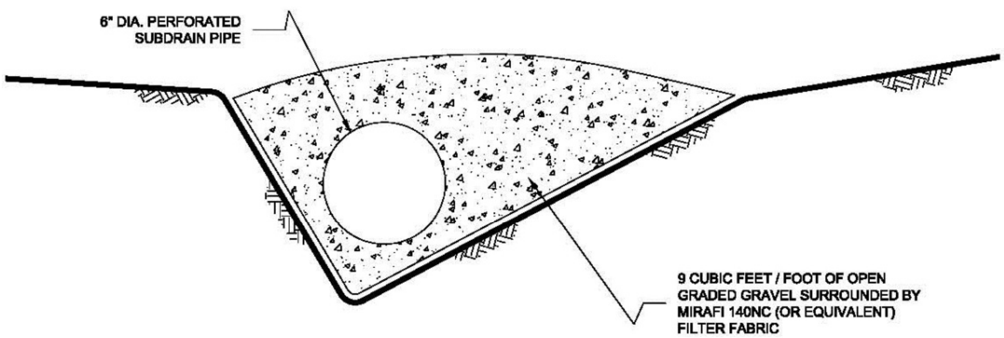
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

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

TYPICAL CANYON DRAIN DETAIL





NOTES:

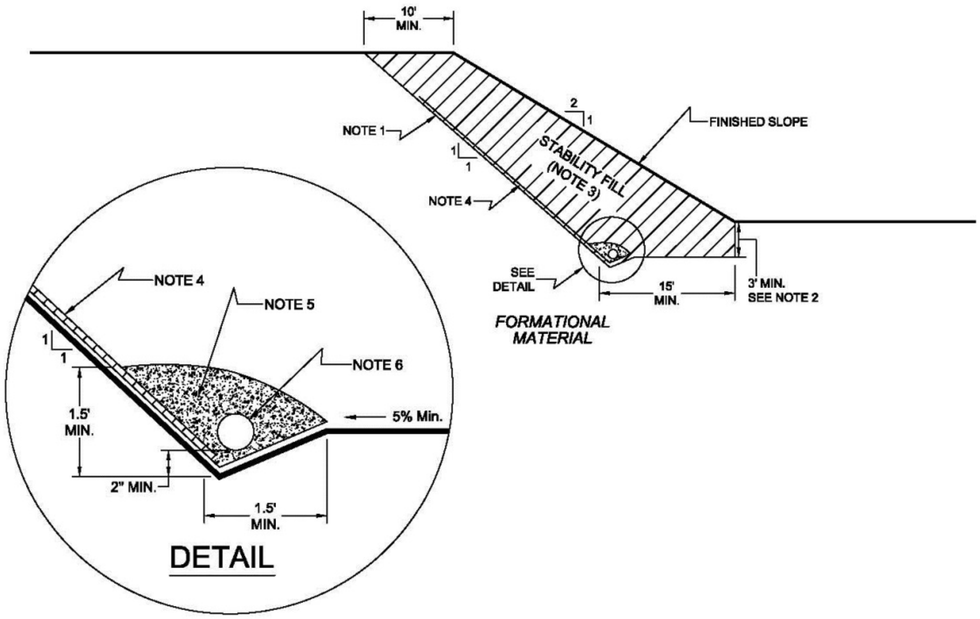
1......8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

NO SCALE

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

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

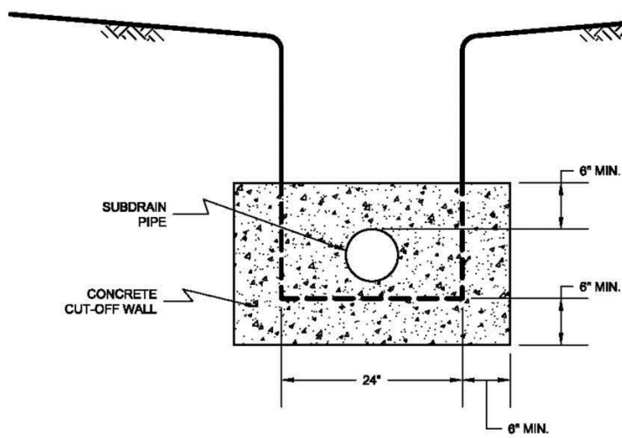
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

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

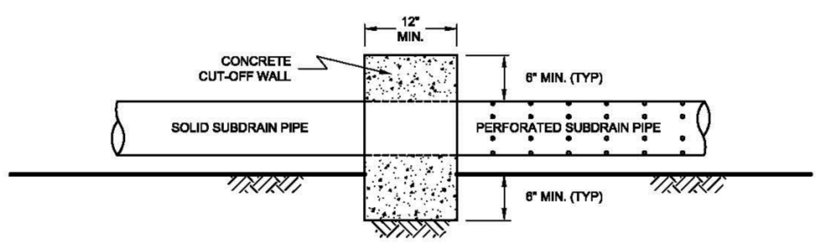
TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW

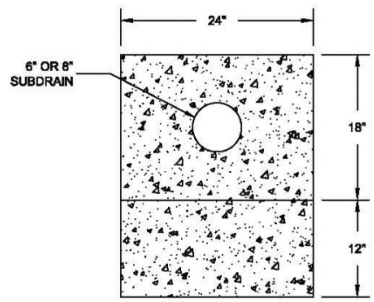


NO SCALE

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

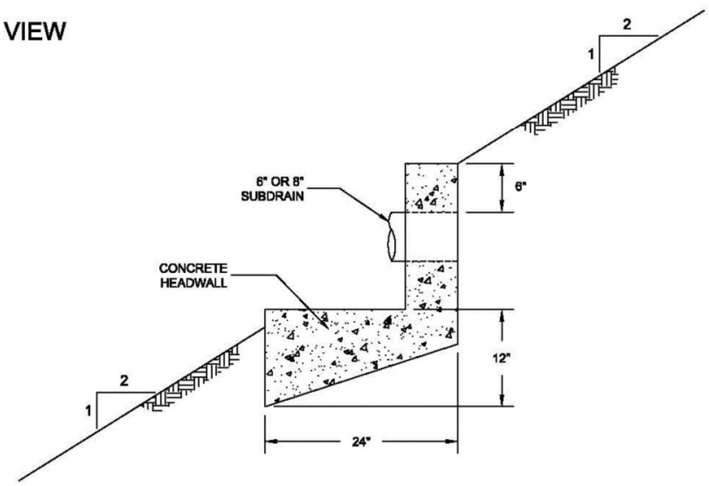
TYPICAL HEADWALL DETAIL

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.