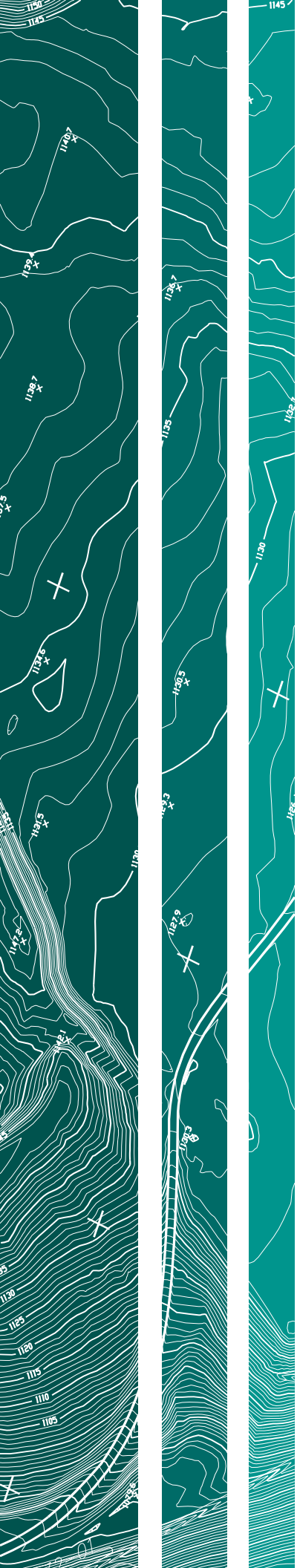
Appendix E

Geotechnical Update



GEOCON

GEOTECHNICAL UPDATE

ROCAS GRANDES II
MULTI-FAMILY RESIDENTIAL DEVELOPMENT
SOUTHEAST OF ALESSANDRO BOULEVARD AND DARWIN DRIVE
MORENO VALLEY, CALIFORNIA

MAY 28, 2024
PROJECT NO. T2927-22-02A

PREPARED FOR:
REALM SEC ALESSANDRO DARWIN, LLC
DEL MAR, CALIFORNIA

Project No. T2927-22-02A
May 28, 2024

Realm SEC Alessandro Darwin, LLC
122 15th Street, #2015
Del Mar, CA 92014

Attention: Mr. Robert Emri

Subject: GEOTECHNICAL UPDATE
ROCAS GRANDES II MULTI-FAMILY RESIDENTIAL DEVELOPMENT
SOUTHEAST OF ALESSANDRO BOULEVARD AND DARWIN DRIVE
MORENO VALLEY, CALIFORNIA

Mr. Emri:

In accordance with the *Terms and Conditions for Performance of Services*, dated May 10, 2024, Geocon West, Inc. (Geocon) herein submits our geotechnical update for the proposed Rocas Grandes II multi-family residential development, planned at the southeast corner of the intersection of Alessandro Boulevard and Darwin Drive, in the City of Moreno Valley, California.

This update report is based on a geotechnical investigation performed at the site by GeoTek, Inc. in 2021 and Geocon's experience during grading and construction of the adjacent Rocas Grandes I development to the west. The accompanying report presents our findings, conclusions, and updated geotechnical recommendations pertaining to the geotechnical aspects of the planned Rocas Grandes II multi-family development in accordance with the 2022 California Building Code. Based on the results of this study, it is our opinion that the site is considered suitable for the proposed improvements provided the recommendations in this report are followed.

The recommendations presented are preliminary at this time and should be verified when design plans are at or near completion, which Geocon should be afforded the opportunity to review and provide revised 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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GEOTECHNICAL UPDATE

1. PURPOSE AND SCOPE

This geotechnical update report presents our findings, conclusions and updated geotechnical recommendations for the Rocas Grandes II multi-family residential development, located southeast of the intersection of Alessandro Boulevard and Darwin Drive, in the City of Moreno Valley, California, as depicted on the *Vicinity Map*, Figure 1. This geotechnical update report is based on the *Geotechnical and Infiltration Evaluation* report, prepared by GeoTek, Inc. (GeoTek) dated December 17, 2021, and Geocon's experience with the Rocas Grandes I multi-family development, which is located immediately west of the subject site. The recommendations of this update report will specifically supersede those of the GeoTek report and are based on the 2022 California Building Code. The GeoTek report should be referenced for evaluations or recommendations not specifically addressed herein. A summary of the information and documentation reviewed for this study is presented in the *List of References*. The *Geologic Map*, Figure 2, presents the approximate locations of the previous geotechnical borings and approximate geologic contacts. The boring logs are presented in Appendix A and the laboratory test data is presented in Appendix B.

2. PROJECT DESCRIPTION

Based on the *Option 2 Site Plan*, prepared by Architects Orange on January 28, 2022, the proposed residential development will consist of twenty, three-story, multi-family residential buildings, a leasing office, a club/fitness cabana, and a pool/spa. We understand that the buildings will be constructed with post-tensioned mat slab type foundation systems. Ancillary improvements are expected to include retaining walls, paved access roadways and parking areas, utilities, and landscape and hardscape areas. Based on the site elevations and surrounding roadway grades, we expect grading will entail cuts and fills on the order of 5 feet or less, exclusive of remedial grading. We expect slopes will be 5 feet or less in height at slope inclinations of 2:1 (horizontal:vertical). The site coordinates are at latitude 33.9153° and longitude -117.2036°.

Structural plans and loading information were not provided to us at this time; however, based on the information provided to us and our experience, we expect the proposed buildings to be constructed of wood or light gauge steel framing, supported on either conventional shallow foundation systems or post-tensioned foundation systems, founded on engineered fill. For our evaluation, we considered that column loads for the proposed structures will be up to 250 kips and wall loads will be up to 5 kips per linear foot.

3. CONCLUSIONS AND RECOMMENDATIONS

3.1 General

3.1.1 Based on our review of the *Geotechnical and Infiltration Evaluation* report and Geocon's findings and analyses for the Rocas Grandes I site, the existing soil or geologic conditions would not preclude the construction of the proposed multi-family residential development, provided the updated geotechnical recommendations presented herein are followed and implemented during design and construction.

3.1.2 The potential geologic hazards at the site include seismic shaking, hydrocompression, seismically induced settlement, and compressible near surface soils.

3.1.3 Mapped and encountered geologic units present at the site consist of older alluvium consisting of silty and clayey sands and sandy silts. Tonalite bedrock is present at depths of more than 50 feet.

3.1.4 Our evaluation of the hydrocompressive characteristics of site soils included a review of the 2021 GeoTek laboratory test results and those of our geotechnical update for the planned Rocas Grandes I development. Based on our evaluation of the test results, we expect hydrocompression of site soils to be less than 1 percent, post-remedial grading. We opine the highest test result of 4.4 percent is associated with surficial materials that will be mitigated via remedial grading.

3.1.5 GeoTek performed an evaluation of seismically induced settlement, as documented in their 2021 geotechnical report. The results of their evaluation indicate that seismic dry sand settlement is on the order of 0.83 inches, with differential settlement of ½ inch occurring over the span of 40 feet. These seismic settlement values should be considered in addition to the static settlement values provided herein.

3.1.6 When design 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 update report.

3.2 Soil Characteristics

3.2.1 The alluvium present at the site should generally be excavatable with moderate effort using conventional earth moving equipment in proper functioning order.

3.2.2 Excavations up to 15 feet are expected during grading operations (exclusive of remedial grading) and utility installation. Excavations should be performed in conformance with OSHA requirements. Some site soils may have little to no cohesion and may be subject to caving in un-shored excavations. The contractor's competent person should evaluate the necessity for the lay back of vertical cut areas or the use of shoring.

3.2.3 Based on the material classifications and laboratory testing by GeoTek, the site soils generally possess a "very low" expansion potential (expansion index (EI) of 0 to 20) and are considered "non-expansive" as defined by 2022 CBC Section 1803.5.3. Table 3.2.3 presents soil classifications based on the EI.

TABLE 3.2.3
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	

3.2.4 Laboratory testing was performed by GeoTek on samples of the site materials to evaluate the percentage of water-soluble sulfate content. *Appendix B* presents results of the laboratory water-soluble sulfate content tests. The test results indicate the on-site materials at the locations tested possess a high sulfate content of 0.0003 percent (less than 10 parts per million [ppm]) equating to an exposure class of "S0" as defined by 2022 CBC Section 1904.3 and ACI 318-19 (Chapter 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 3.2.4
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

[1] Maximum water to cement ratio limits do not apply to lightweight concrete.

3.2.5 The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities along the access roads or from nearby developments (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

3.2.6 According to Caltrans *Corrosion Guidelines* (Version 3.2; May 2021), soils are considered corrosive to buried concrete and steel structures if one or more of the following conditions exist: chloride concentration is 500 parts per million (ppm) or greater, or sulfate concentration is 1,500 ppm or greater, or the potential of hydrogen (pH) is 5.5 or less. Resistivity serves as an indicator parameter for the possible presence of soluble salts and is not included as a parameter to define a corrosive area for structures. A minimum resistivity value for soil and/or water less than 1,500 ohm-centimeters may indicate the presence of high quantities of soluble salts and a higher propensity for corrosion.

3.2.7 Laboratory testing indicates the site soils have a minimum electrical resistivity of 2,750 ohm-cm, possess 35 ppm chloride, less than 10 ppm sulfate, and a pH of 7.9. As shown in Table 3.2.7 below, the site would not be classified as "corrosive" to buried metallic improvements, in accordance with the Caltrans *Corrosion Guidelines* (Version 3.2; May 2021).

TABLE 3.2.7
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

3.2.8 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.

3.3 Grading

3.3.1 Grading should be performed in accordance with the *Recommended Grading Specifications* of Appendix C and the grading ordinances of the City of Moreno Valley.

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

3.3.3 Site preparation should begin with the removal of any previous structures and infrastructure, deleterious material, debris, buried trash, 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 and debris. Material generated during stripping and/or site demolition should be exported from the site.

3.3.4 The upper portion of older alluvium within a 1:1 (h:v) projection of the limits of grading should be removed to expose older alluvium with a relative compaction of at least 85 percent (per ASTM D1557). Areas of loose, dry, or compressible soils will require deeper excavation and processing prior to fill placement. Within structural areas, removals should extend approximately 5 feet below the existing ground surface, or for a depth that will provide a minimum of 3 feet of engineered fill below proposed foundation bottoms. Lateral over-excavations should be performed that are at least 5 feet or equal to the depth of removal, whichever is greater. Within pavement and walkway areas, removals should extend at least 3 feet below subgrade elevation. The Engineering Geologist should evaluate the actual depth of removals during grading operations. The bottom of the excavations should consist of competent older alluvium, and should be scarified to a depth of at least 1 foot, moisture conditioned at 0 to 2 percent above optimum moisture content, and compacted to 90 percent of the maximum dry density, as determined by ASTM D1557.

3.3.5 Geocon should observe the removal bottoms to check the competence of the exposed soil. Deeper excavations may be required if dry, loose, or soft materials are present at the base of the removals.

3.3.6 The site should be brought to finish grade elevations with fill compacted in layers. Layers of fill should be no thicker than will allow for adequate bonding and compaction. Fill, including backfill and scarified ground surfaces, should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density, at 0 to 2 percent 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. Earthwork should be observed, and compacted fill tested by representatives of Geocon.

3.3.7 The engineered fill placed within 4 feet of proposed finish grade elevation should have a “very low” to “low” expansion potential (EI less than 50).

3.3.8 Should oversized rock (i.e. rock greater than 12-inches in maximum dimension) be encountered during earthwork operations. The oversize rock will require special handling and placement. Rocks greater than 3 inches in maximum dimension should not be placed within utility trench backfill. Rocks greater than 6 inches in maximum dimension should not be placed in engineered fill within the upper 3 feet of finish grade. Rocks 6 to 12 inches in maximum dimension should be placed deeper than 3 feet below finished grade elevations. Rocks 12 inches or larger in maximum dimension should be exported from the site or placed at least 5 feet below finished grade elevations, in accordance with the *Recommended Grading Specifications* in Appendix C.

3.4 Utility Trench Backfill

3.4.1 Utility trenches should be properly backfilled in accordance with the requirements of the City of Moreno Valley and the following recommendations. Pipes should be bedded with well-graded crushed rock or clean sands (Sand Equivalent greater than 30) to a depth of at least one foot over the pipe. The bedding material must be inspected and approved in writing by a qualified representative of Geocon. The use of well-graded crushed rock is only acceptable if used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil. 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.

3.4.2 Trench excavation bottoms must be observed and approved in writing by a representative of Geocon, prior to placing bedding materials, fill, gravel, or concrete.

3.4.3 Utility trench backfill should be placed in layers no thicker than will allow for adequate bonding and compaction. Utility backfill should be compacted to a dry density of at least 90 percent of the laboratory maximum dry density and moisture conditioned at 0 to 2 percent above optimum moisture content as determined by ASTM D1557. Backfill at the finish subgrade elevation of new pavements should be compacted to at least 95 percent of the maximum dry density. Backfill materials placed below the recommended moisture content may require additional moisture conditioning prior to placing additional fill.

3.5 Seismic Design Criteria

3.5.1 The following table summarizes site-specific design criteria obtained from the 2022 California Building Code (CBC; Based on the 2021 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 2022 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 3.5.1
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	1.749g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.684g	Figure 1613.2.1(3)
Site Coefficient, F_A	1.0	Table 1613.2.3(1)
Site Coefficient, F_V	1.7	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.749g	Section 1613.2.3 (Eqn 16-20)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	1.164g*	Section 1613.2.3 (Eqn 16-21)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.166g	Section 1613.2.4 (Eqn 16-22)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.776g*	Section 1613.2.4 (Eqn 16-23)

*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 **not** been increased in accordance with Supplement 3 of ASCE 7-16.

3.5.2 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 3.5.2
ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.74g	Figure 22-9
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.814g	Section 11.8.3 (Eqn 11.8-1)

3.5.3 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 7.21 magnitude event occurring at a hypo central distance of 9.27 kilometers from the site.

3.5.4 Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, corresponding to two-thirds of the MCE peak ground acceleration. The result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 6.96 magnitude occurring at a hypocentral distance of 12.79 kilometers from the site.

3.5.5 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.

3.6 Conventional Foundation Design

3.6.1 The foundation recommendations presented herein are for proposed multi-family residential structures founded upon properly compacted engineered fill.

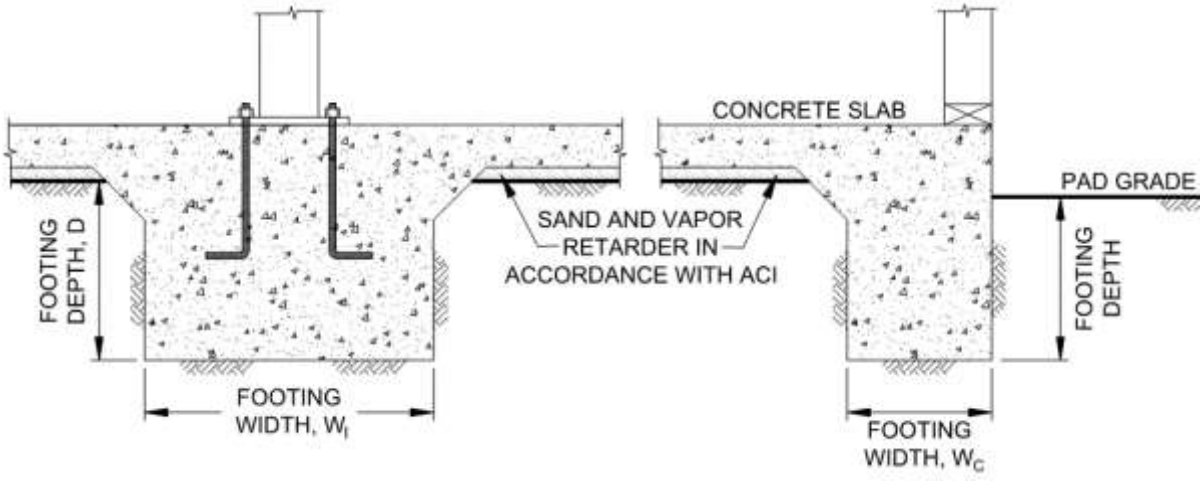
3.6.2 Foundations deriving support in newly placed engineered fill should be underlain by a minimum of 3 feet of engineered fill. Foundations for the structure should consist of continuous strip footings and/or isolated spread footings. Table 3.6.2 provides a summary of the foundation design recommendations.

TABLE 3.6.2
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	Four No. 4 Bars, Two at the Top and Two at the Bottom
Allowable Bearing Pressure	2,500 psf
Bearing Pressure Increase	500 psf per Foot of Depth
	250 psf per Foot of Width
Maximum Allowable Bearing Pressure	3,500 psf
* Estimated Total Settlement	1¼ Inch
*Estimated Differential Settlement	⅝ Inch over 20 Feet
Design Expansion Index	50 or less

*Total settlement should consider expected seismic settlement presented herein.

3.6.3 The foundations should be embedded in accordance with the recommendations herein and the *Wall/Column Footing Dimension Detail* below. The embedment depths should be measured from the lowest adjacent pad grade for both interior and exterior footings. Footings should be deepened such that the bottom outside edge of the footing is at least 7 feet horizontally from the face of the slope.



Wall/Column Footing Dimension Detail

3.6.4 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.

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

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

3.6.7 Once the design and foundation loading configurations for the proposed improvements proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.

3.7 Concrete Slabs-On-Grade

3.7.1 Concrete slabs-on-grade for the structures should be constructed in accordance with Table 3.7.1.

TABLE 3.7.1
MINIMUM CONCRETE SLAB-ON-GRADE RECOMMENDATIONS

Parameter	Value
Minimum Concrete Slab Thickness	4 inches
Minimum Steel Reinforcement	No. 3 Bars 18 Inches on Center, Both Directions
Typical Slab Underlayment	3 to 4 Inches of Sand/Gravel/Base
Design Expansion Index	50 or less

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

3.7.3 The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. It is common to have 3 to 4 inches of sand for 5-inch and 4-inch thick slabs, respectively, in the southern California region. However, we should be contacted to provide recommendations if the bedding sand is thicker than 6 inches. The foundation design engineer should provide appropriate concrete mix design criteria and curing measures to assure proper curing of the slab by reducing the potential for rapid moisture loss and subsequent cracking and/or slab curl. We suggest that the foundation design engineer present the concrete mix design and proper curing methods on the foundation plans. It is critical that the foundation contractor understands and follows the recommendations presented on the foundation plans.

3.7.4 Some projects remove the sand layer below the slab in parking structure areas. This is acceptable from a geotechnical engineering standpoint; however, relatively minor cracks could form due to differential curing. Therefore, the structural engineer and/or the concrete contractor should provide recommendations for proper curing techniques to help prevent cracking.

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

3.7.6 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisturized to maintain a moist condition as would be expected in any such concrete placement.

3.7.7 The concrete slab-on-grade recommendations are based on soil support characteristics only. The project structural engineer should evaluate the structural requirements of the concrete slabs for supporting expected loads.

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

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

3.8 Post-Tensioned Foundation Recommendations

3.8.1 As an alternative to conventional foundations, the proposed multi-family residential structures may be supported by a post-tensioned foundation system founded upon properly compacted engineered fill. Foundations deriving support in newly placed engineered fill should be underlain by a minimum of 3 feet of engineered fill.

3.8.2 The post-tensioned foundation 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 2022 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 3.8.2 for the Foundation Categories listed. The parameters presented in Table 3.8.2 are based on the guidelines presented in the PTI DC 10.5 design manual.

TABLE 3.8.2
POST-TENSIONED FOUNDATION SYSTEM DESIGN PARAMETERS

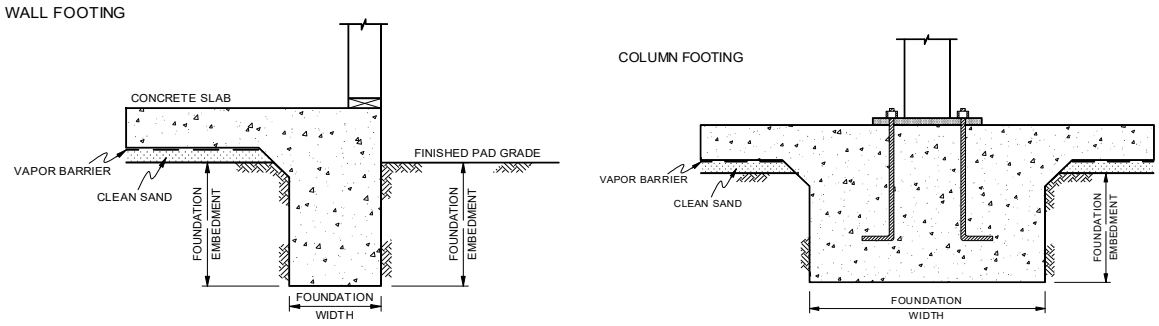
Post-Tensioning Institute (PTI) Third Edition Design Parameters	Foundation Parameters
Thornthwaite Index	-20
Equilibrium Suction	3.9
Edge Lift Moisture Variation Distance, e_M (feet)	5.1
Edge Lift, y_M (inches)	1.10
Center Lift Moisture Variation Distance, e_M (feet)	9.0
Center Lift, y_M (inches)	0.47

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

3.8.4 If the Structural Engineer proposes a post-tensioned foundation design method other than the PTI DC 10.5:

- The deflection criteria presented in Table 3.8.2 are still applicable.
- Interior stiffener beams should be used.
- The width of the perimeter foundations should be at least 12 inches.
- The perimeter footing embedment depths should be at least 18 inches. The embedment depths should be measured from the lowest adjacent pad grade.

3.8.5 A wall/column footing dimension detail depicting footing embedment is provided below.



Wall/Column Footing Detail

3.8.6 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.

3.8.7 During the construction of the 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 specifically designed by the Structural Engineer.

3.8.8 Foundations may be designed for an allowable soil bearing pressure of 3,500 pounds per square foot (psf) (dead plus live load). This bearing pressure may be increased by one-third for transient loads due to wind or seismic forces. We estimate the total settlements under the imposed allowable loads to be up to 1¼ inch with differential settlements on the order of ⅝ inch over a horizontal distance of 20 feet. Static settlements should be considered in addition to expected seismic settlements presented herein.

3.8.9 Slabs that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder. The vapor retarder design should be consistent with the guidelines presented in the American Concrete Institute's (ACI) *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials* (ACI 302.2R-06). The vapor retarder used should be specified by the project architect or developer based on the type of floor covering that will be installed and if the structure will possess a humidity-controlled environment. The bedding sand thickness should be determined by the project foundation engineer, architect, and/or developer. However, we should be contacted to provide recommendations if the bedding sand is thicker than 4 inches. Placement of 3 inches and 4 inches of sand is common practice in southern California for 5-inch and 4-inch thick slabs, respectively. The foundation engineer should provide appropriate concrete mix design criteria and curing measures that may be utilized to assure proper curing of the slab to reduce the potential for rapid moisture loss and subsequent cracking and/or slab curl.

3.8.10 Special subgrade presaturation is not deemed necessary prior to placing concrete; however, the exposed foundation and slab subgrade soil should be moisture conditioned, as necessary, to maintain a moist condition as would be expected in such concrete placement.

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

3.8.12 Geocon should be consulted to provide additional design parameters as required by the Structural Engineer.

3.8.13 Foundation excavations should be observed and approved in writing by a representative of Geocon, prior to the placement of reinforcing steel and concrete to check that the excavations and exposed soil conditions are consistent with those expected. If unexpected soil conditions are encountered, foundation modifications may be required. This office should be provided a copy of the foundation plans so that the recommendations presented herein can be reviewed and revised if necessary.

3.9 Miscellaneous Foundations

3.9.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed structure, can be supported on shallow foundation systems supported by engineered fill, following the completion of grading, per the recommendations provided in the *Grading* section of this report. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.

3.9.2 Miscellaneous foundations may be designed for a bearing value of 1,500 psf, and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

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

3.10 Exterior Concrete Flatwork

3.10.1 Exterior concrete flatwork not subject to vehicular traffic should be constructed in accordance with the recommendations presented in Table 3.10.1. The recommended steel reinforcement would help reduce the potential for cracking.

TABLE 3.10.1
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.

3.10.2 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 0 to 2 percent above optimum moisture content in accordance with ASTM D1557.

3.10.3 Even with the incorporation of the recommendations of this report. 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.

3.10.4 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.

3.10.5 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.

3.10.6 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.

3.11 Conventional Retaining Walls

3.11.1 The recommendations presented herein are generally applicable to the design of rigid concrete or masonry retaining walls having a maximum height of 5 feet. In the event that walls higher than 5 feet or other types of walls are planned, Geocon should be consulted for additional recommendations.

3.11.2 Retaining walls not restrained at the top and having a level backfill surface should be designed for an active soil pressure equivalent to the pressure exerted by a fluid density of 30 pounds per cubic foot (pcf). These soil pressures assume that the backfill materials within an area bounded by the wall and a 1:1 plane extending upward from the base of the wall possess an EI of 50 or less. For walls where backfill materials do not conform to the criteria herein, Geocon should be consulted for additional recommendations.

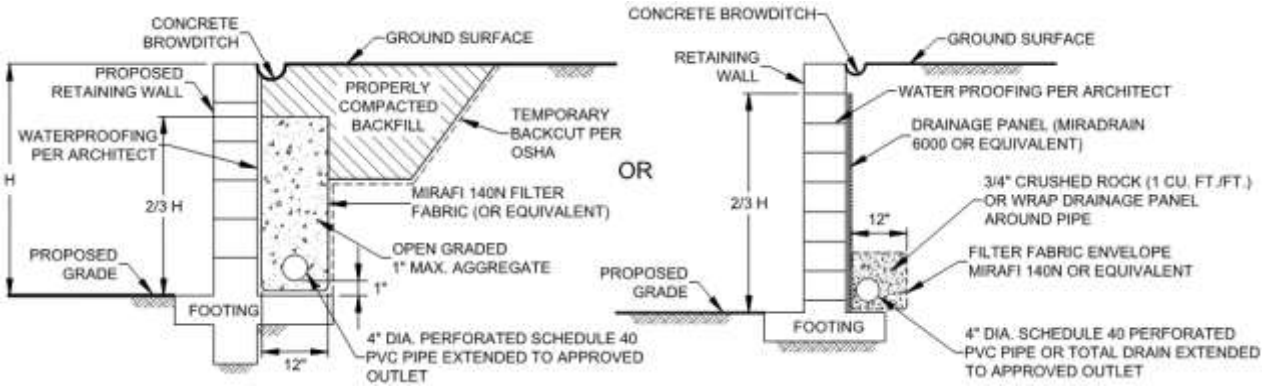
3.11.3 Unrestrained walls are those that are allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where level walls are restrained from movement at the top, the walls should be designed for a soil pressure equivalent to the pressure exerted by a fluid density of 64 pcf.

3.11.4 The wall pressures provided above assume that the proposed retaining walls will support relatively undisturbed alluvial soils or engineered fill derived from onsite soil. If import soil is used to backfill proposed walls, revised earth pressures may be required to account for the geotechnical properties of the soil placed as engineered fill. This should be evaluated once the use of import soil is established. All imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site.

3.11.5 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 at-rest pressure with an equivalent fluid density of 95 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.

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

3.11.7 Retaining walls should be provided with a drainage system adequate to prevent the buildup of hydrostatic forces and waterproofed as required by the project architect. The soil immediately adjacent to the backfilled retaining wall should be composed of free draining material completely wrapped in Mirafi 140N (or equivalent) filter fabric for a lateral distance of 1 foot for the bottom two-thirds of the height of the retaining wall. The upper one-third should be backfilled with less permeable compacted fill to reduce water infiltration. Alternatively, a drainage panel, such as a Miradrain 6000 or equivalent, can be placed along the back of the wall. The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the property adjacent to the base of the wall. The recommendations herein assume a properly compacted backfill (EI of 50 or less) with no hydrostatic forces or imposed surcharge load. If conditions different than those described are expected or if specific drainage details are desired, Geocon should be contacted for additional recommendations. A graphic depicting typical retaining wall drainage is provided below.



Typical Retaining Wall Drainage Detail

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

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

3.11.10 It is recommended that line-load surcharges from adjacent wall footings, use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For\ {}^{x}/_{H} \leq 0.4$$

$$\sigma_H(z) = \frac{0.20 \times \left(\frac{z}{H}\right)}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

and

$$For\ {}^{x}/_{H} > 0.4$$

$$\sigma_H(z) = \frac{1.28 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^2} \times \frac{Q_L}{H}$$

where *x* is the distance from the face of the excavation or wall to the vertical line-load, *H* is the distance from the bottom of the footing to the bottom of excavation or wall, *z* is the depth at which the horizontal pressure is desired, Q_L is the vertical line-load and $\sigma_H(z)$ is the horizontal pressure at depth *z*.

3.11.11 It is recommended that vertical point-loads, from construction equipment outriggers or adjacent building columns use horizontal pressures generated from NAV-FAC DM 7.2. The governing equations are:

$$For\ {}^{x}/_{H} \leq 0.4$$

$$\sigma_H(z) = \frac{0.28 \times \left(\frac{z}{H}\right)^2}{\left[0.16 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

and

$$For\ {}^{x}/_{H} > 0.4$$

$$\sigma_H(z) = \frac{1.77 \times \left(\frac{x}{H}\right)^2 \times \left(\frac{z}{H}\right)^2}{\left[\left(\frac{x}{H}\right)^2 + \left(\frac{z}{H}\right)^2\right]^3} \times \frac{Q_P}{H^2}$$

then

$$\sigma'_H(z) = \sigma_H(z)cos^2(1.1\theta)$$

where *x* is the distance from the face of the excavation/wall to the vertical point-load, *H* is distance from the outrigger/bottom of column footing to the bottom of excavation, *z* is the depth at which the horizontal pressure is desired, Q_p is the vertical point-load, $\sigma_H(z)$ is the horizontal pressure at depth *z*, Θ is the angle between a line perpendicular to the excavation/wall and a line from the point-load to location on the excavation/wall where the surcharge is being evaluated, and $\sigma_H(z)$ is the horizontal pressure at depth *z*.

3.11.12 In addition to the recommended earth pressure, the upper 10 feet of the retaining wall adjacent to the street or driveway areas should be designed to resist a uniform lateral pressure of 100 psf, acting as a result of an assumed 300 psf surcharge behind the shoring due to normal street traffic. If the traffic is kept back at least 10 feet from the wall, the traffic surcharge may be neglected.

3.11.13 Retaining wall foundations may be designed in accordance with the recommendations in the *Conventional Foundation Design* section of this report.

3.12 Elevator Pit Design

3.12.1 If used, the elevator pit slab and retaining wall should be designed by the project structural engineer. Elevator pit foundation and walls may be designed in accordance with the recommendations in the *Conventional Foundation Design* and *Conventional Retaining Walls* sections of this report.

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

3.12.3 If retaining wall drainage is to be provided, the drainage system should be designed in accordance with the *Conventional Retaining Walls* section of this report.

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

3.13 Elevator Piston

3.13.1 If a plunger-type elevator piston is installed for this project, a deep drilled excavation will be required. It is important to verify that the drilled excavation is not situated immediately adjacent to a foundation, or the drilled excavation could compromise the existing foundation support, especially if the drilling is performed subsequent to the foundation construction. In addition, boulders and cobbles may be encountered in the existing fill or alluvial soils, and some of the site soils have little to no cohesion and are prone to excessive caving. The contractor should be prepared for difficult drilling conditions.

3.13.2 Caving is expected, and the contractor should be prepared to use casing and should have it readily available at the commencement of drilling activities. Continuous observation of the drilling and installation of the elevator piston by the Geotechnical Engineer should be performed.

3.13.3 The annular space between the piston casing and drilled excavation wall should be filled with a minimum of 1½-sack slurry pumped from the bottom up. As an alternative, pea gravel may be utilized. The use of soil to backfill the annular space is not acceptable.

3.14 Swimming Pools

3.14.1 For the proposed pools, the shell bottoms should be designed as a free-standing structure and may derive support on a minimum of 2 feet of engineered fill 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.

3.14.2 Swimming pool foundations and walls may be designed in accordance with the recommendations in the *Conventional Foundation Design* and *Conventional Retaining Walls* sections of this report. A hydrostatic relief valve should be considered as part of the swimming pool design unless a gravity drain system can be placed beneath the pool shell.

3.14.3 Based on the soil overburden load that will be removed during excavation of the swimming pool, anticipated settlements are expected to be small. Static differential settlement of the pool is not expected to exceed ¼ inch over a horizontal distance of 40 feet.

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

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

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

3.14.7 To mitigate the potential for moisture infiltration into the subgrade soils beneath the pool deck, we recommend the construction of a deepened footing along the outside edge of the pool deck flatwork. A subdrain consisting of 4-inch diameter perforated PVC pipe should be installed inside the deepened footing and sloped to drain into an approved outlet. The pipe should be surrounded by ¾ inch open-graded gravel and wrapped with filter fabric.

3.14.8 If the proposed pools are in proximity to a proposed or existing structure, consideration should be given to the construction sequence. If the proposed pool is to be constructed near an existing structure, or a proposed structure that is constructed before the pool construction, the excavation required for the pool could remove a critical component of lateral support from the foundations of the structure and would therefore require shoring to safeguard the foundations. Once information regarding the pool locations and depth becomes available, this information should be provided to Geocon for review and possible revision of these recommendations.

3.15 Lateral Design

3.15.1 To resist lateral loads, a passive pressure exerted by an equivalent fluid weight of 350 pounds per square foot per foot (psf/ft) up to a maximum of 3,500 psf, should be used for the design of footings or shear keys poured neat against engineered fill. The allowable passive pressure assumes a horizontal surface extending at least 5 feet, or three times the surface generating the passive pressure, whichever is greater. The upper 12 inches of material in areas not protected by slabs or pavement should not be included in design for passive resistance.

3.15.2 If friction is to be used to resist lateral loads, an allowable coefficient of friction between soil and concrete of 0.4 should be used for design.

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

3.16 Pavement Design

3.16.1 The final pavement sections for roadways should be based on the R-value of the subgrade soils encountered at final subgrade elevation. The civil engineer should evaluate the final traffic index for pavements. Pavements should be designed in accordance with the City of Moreno Valley's 2022 *Standard Plans, Street Improvements (Section 1)*, when final Traffic Indices and R-value test results of subgrade soil are completed. GeoTek performed R-value testing of site soils and obtained a test result of 50 or greater, as documented in their 2021 geotechnical report. An R-value of 50 was used in their calculations of preliminary pavement sections. The GeoTek calculated preliminary flexible pavement sections are presented in Table 3.16.1; these sections are based on City of Moreno minimum requirements.

TABLE 3.16.1
PRELIMINARY FLEXIBLE PAVEMENT SECTIONS

Location	Traffic Index	Assumed Subgrade R-Value	Asphalt Concrete (inches)	Aggregate Base (inches)
Local Street / Modified Local Street	6	50	3.6[1]	6.0[1]
Collector	7		3.6[1]	6.0[1]
Divided Major Arterial (Truck Route[2])	12[2]		6.0[1]	12.0[1]

[1]Minimum requirements per City of Moreno Valley *Standard Plan MVSI-100A-3*.

[2]Truck Route designation and corresponding Traffic Index of 12 for a Divided Major Arterial per City of Moreno Vallet *Standard Plan MVSI-100B-1*.

3.16.2 The upper 3 feet of the subgrade soil should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density, at or slightly above optimum moisture content, per the City of Moreno Valley's *Standard Plan MVSI-166D-2*.

3.16.3 The crushed aggregate base and asphalt concrete materials should conform to Section 200-2.2 and Section 203-6, respectively, of the latest edition of the *Standard Specifications for Public Works Construction* (Greenbook). Aggregate base materials should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density, at or slightly above optimum moisture content. Asphalt concrete should be compacted to a density of 95 percent of the laboratory Hveem density in accordance with ASTM D1561.

3.16.4 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 3.16.4 provides the traffic categories and design parameters used for the calculations for 20-year design life.

TABE 3.16.4
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

3.16.5 We used the parameters presented in Table 3.16.5 to calculate the pavement design sections. We should be contacted to provide updated design sections, if necessary.

TABLE 3.16.5
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

3.16.6 Based on the criteria presented herein, the PCC pavement sections should have a minimum thickness as presented in Table 3.16.6.

TABLE 3.16.6
RIGID VEHICULAR PAVEMENT RECOMMENDATIONS

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

3.16.7 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, at 0 to 2 percent above optimum moisture content.

3.16.8 Adequate joint spacing should be incorporated into the design and construction of the rigid pavement in accordance with Table 3.16.8.

TABLE 3.16.8
MAXIMUM JOINT SPACING

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

3.16.9 The rigid pavement should also be designed and constructed incorporating the parameters presented in Table 3.16.9.

TABLE 3.16.9
ADDITIONAL RIGID PAVEMENT RECOMMENDATIONS

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

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

3.16.11 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.

3.16.12 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.

3.16.13 Concrete curb/gutter should be placed on soil subgrade compacted to a dry density of at least 95 percent of the laboratory maximum dry density, at 0 to 2 percent above optimum moisture content. Cross-gutters that receives vehicular traffic 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.

3.17 Site Drainage and Moisture Protection

3.17.1 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 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. Roof and pavement drainage should be directed into conduits that carry runoff away from the proposed structure.

3.17.2 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.

3.17.3 Storm water mitigation systems should be offset a minimum of 20 feet from the outside edge of structural footings, and 15 feet away from improvements, so as to reduce to potential for distress to occur due to migrating water.

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

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

3.18 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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 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 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 letter. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

LIST OF REFERENCES

1. American Concrete Institute (ACI), 2019, *Building Code Requirements for Structural Concrete*, ACI 318-19, Report by ACI Committee 318.

2. American Concrete Institute (ACI), 2021, *Commercial Concrete Parking Lots and Site Paving Design and Construction*, ACI PRC-330-21, Report by ACI Committee 330.

3. AO, 2022, *Rocas Grandes II Option 2 Site Plan*, Job No. 0021-742, dated January 28.

4. ASCE 7-16, 2016, *Minimum Design Loads for Buildings and Other Structures*.

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

6. California Department of Transportation (Caltrans), 2018, *Highway Design Manual*, Seventh Edition, dated July 1.

7. Geocon West, Inc., 2021a, *Addendum Foundation Recommendations, Rocas Grandes Multi-Family Residential Development, Allessandro Boulevard and Darwin Drive, Moreno Valley, California*, Project T2927-22-01, dated December 8.

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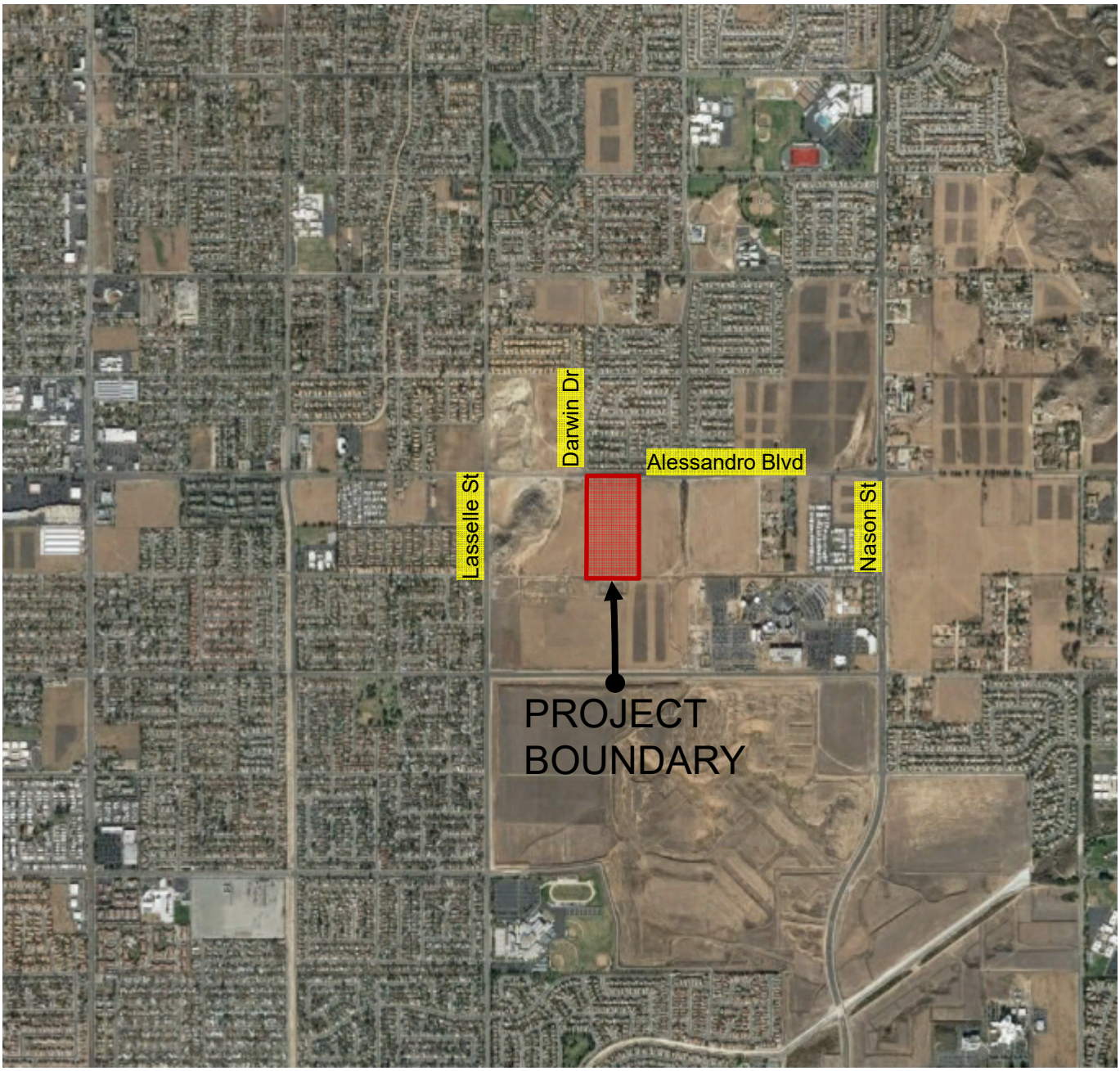
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11. OSHPD, Seismic Design Maps, accessed May 2024.

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

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0’ 2000’ 4000’

SCALE: 1” = 2000’

N

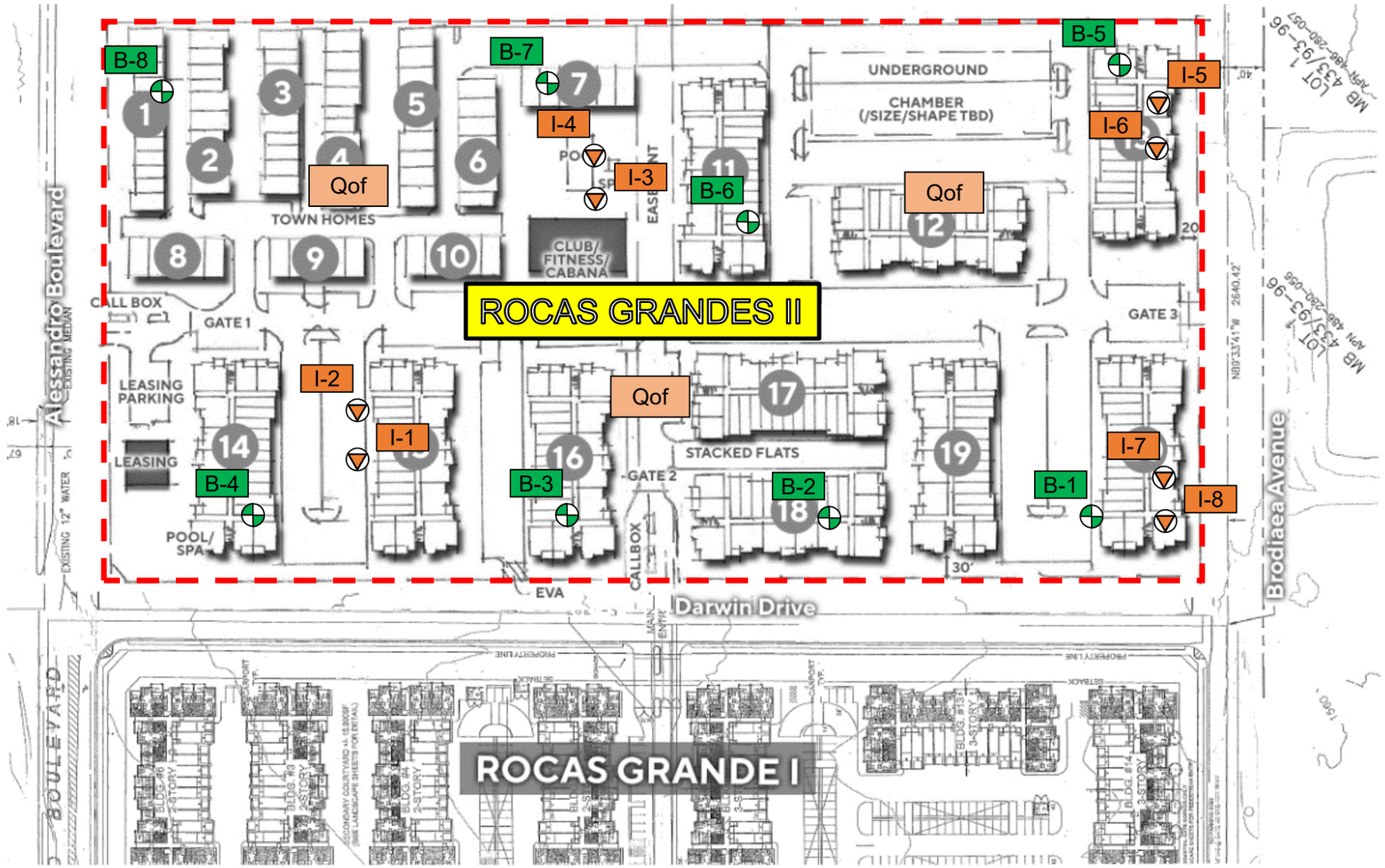
SOURCE: Google Earth, 2021

VICINITY MAP

GEOCON
WEST, INC.
GEOTECHNICAL, ENVIRONMENTAL, MATERIALS
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PHONE 951-304-2300 www.geoconinc.com

ROCAS GRANDES II
MULTI-FAMILY RESIDENTIAL DEVELOPMENT
SOUTHEAST OF ALESSANDRO BOULEVARD
AND DARWIN DRIVE
MORENO VALLEY, CALIFORNIA

ATS			MAY 2024	PROJECT NO. T2927-22-02	FIG. 1



GEOCON LEGEND

Locations are approximate

B-8 .....GEOTEK GEOTECHNICAL BORING LOCATION (2021)

I-8 .....GEOTEK PERCOLATION TEST BORING LOCATION (2021)

.....PROJECT LIMITS

Qof ..... OLD ALLUVIAL FAN DEPOSITS

0’ 150’ 300’

SCALE: 1” = 150’

N

Source: AO, L.P., 2022, *Option 2 Site Plan*, Rocas Grandes II, Job No. 0021-742, dated January 28.

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ATS

GEOLOGIC MAP

ROCAS GRANDES II
MULTI-FAMILY RESIDENTIAL DEVELOPMENT
SOUTHEAST OF ALESSANDRO BOULEVARD
AND DARWIN DRIVE
MORENO VALLEY, CALIFORNIA

MAY 2024 | PROJECT NO. T2927-22-02 | FIG. 2

APPENDIX A

APPENDIX A
EXPLORATORY EXCAVATIONS

In 2021, GeoTek performed a subsurface investigation by drilling eight small-diameter geotechnical borings and eight percolation test borings within the site. The borings were drilled to depths ranging between approximately 21 and 51½ feet below the existing ground surface, and the percolation borings were drilled to depths of approximately 5 feet below the existing ground surface. Logs of the geotechnical and percolation test borings are presented herein. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The approximate locations of the borings and percolation tests are presented on the *Geologic Map* (see Figure 2).

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/19/2021

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/6 in.	SAMPLES: Sample Number	USCS Symbol	BORING NO.: B-1 / MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium: Disced in upper one foot			
		11 18 20	R1	ML	Sandy SILT, brown, moist, very stiff	3.4	113.7	
5		20 21 15	R2	SM	Silty f-c SAND, red-brown, moist, medium dense	5.7	110.7	
		7 12 20	R3					
10		8 11 13	R4		brown, trace gravel	3.5	115.5	
15		14 34 40	R5		red-brown, very dense			
20		12 17 24	R6		Silty f-c SAND with gravel, red-brown, slightly moist, medium dense	2.0	123.2	
					BORING TERMINATED AT 21.5 FEET No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/19/2021

BORING NO.: B-2

Depth (ft)	Sample Type	Blows/6 in.	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					Older Alluvium: Disced in upper one foot			
		16 33 40	R1	ML	Sandy SILT, brown, moist, very stiff	4.0		
		24 20 20	R2	SM	Silty f SAND, red-brown, slightly moist, medium dense	5.2	102.1	MD, EI, SH
5		9 9 11	R3	ML	Sandy SILT, red-brown, moist, medium stiff, slightly disturbed sample	5.9		
10		13 21 28	R4	SM	Silty f SAND, light brown, moist, dense			
15		20 37 50	R5		very dense	5.2	121.2	
20		40 50-5"	R6		Silty f-c SAND, red-brown, moist, very dense, trace caliche stringers			
					BORING TERMINATED AT 21 FEET No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/19/2021

BORING NO.: B-3

Depth (ft)	Sample Type	Blows/6 in.	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					Older Alluvium: Disced in upper one foot			
		15 40 50-5"	R1	ML	Sandy SILT, brown, moist, very stiff, trace caliche stringers and roothairs	4.6	112.8	
		33 24 16	R2	SM	Silty f-c SAND, brown, slightly moist, medium dense, trace caliche stringers	4.6	106.4	SR
5		7 7 11	R3		trace gravel	2.3	116.9	HC
10		9 11 17	R4	SP	F-c SAND with trace SILT, light brown, moist, medium dense			
15		18 34 50	R5	SM	Silty f-m SAND trace CLAY, dark red-brown, moist, very dense	9.9	124.2	
20		21 34 40	R6					
					BORING TERMINATED AT 21.5 FEET			
					No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/19/2021

BORING NO.: B-4

Depth (ft)	Sample Type	Blows/6 in.	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					Older Alluvium: Disced in upper one foot			
	■	13 24 34	R1	ML	Sandy SILT, brown, moist, hard, trace roothairs	5.2	116.7	
	■	28 40 41	R2		red-brown	5.2	118.1	
5	■	19 15 15	R3		very stiff			
	■	13 23 30	R4	SM	Silty f-c SAND, red-brown, moist, dense			
10	■	22 35 39	R5		trace caliche stringers	5.8	119.6	
15	■	23 35 45	R5					
20		50-5"			**BORING TERMINATED AT 20.5 FEET** No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

Sample type: ■ ---Ring ■ ---SPT ▨ ---Small Bulk ▩ ---Large Bulk □ ---No Recovery ▽ ---Water Table

Lab testing: AL = Atterberg Limits; SR = Sulfate/Resisitivity Test; EI = Expansion Index; SH = Shear Test; SA = Sieve Analysis; HC= Consolidation; RV = R-Value Test; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/24/2021

BORING NO.: B-5

Depth (ft)	Sample Type	Blows/6 in.	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					Older Alluvium: Disced in upper one foot			
		5 14 10	R1	ML	Sandy SILT, brown, slightly moist, very stiff, disturbed sample, roots	3.7		RV, SR
		4 9 20	R2			4.1	105.4	HC
5		12 13 13	R3	SM	Silty f-m SAND, red-brown, moist, medium dense	4.8	117.9	
		16 17 20	R4			3.1	113.8	
10		14 16 18	R5	SP	F-c SAND with trace SILT, dark red-brown, slightly moist, medium dense, friable			
15		24 34 50	R6	SM	Silty f-m SAND, dark reddish brown, moist, very dense	8.9	122.7	
20		20 27 50-5"	R7	SC	Clayey f-m SAND, red-brown, moist, very dense			
25		12 18 28	S1	ML	Sandy SILT, red-brown, moist, hard	11.8		
30		17 20 25	S2	SM	Silty f SAND, red-brown, moist, dense			

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/24/2021

BORING NO.: B-5 (cont.)

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/6 in.	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium:			
35		10 11 17	S3	SP	F-m SAND with trace SILT, red-brown, moist, medium dense		3.6	
40		9 18 20	S4	SP	F-c SAND, red-brown, grey-brown, moist, dense, trace interbedded clay lens (0.25 inch thick)			
45		16 21 21	S5		no clay lenses present		4.2	
50		20 23 23	S6					
					BORING TERMINATED AT 51.5 FEET No groundwater encountered Boring backfilled with soil cuttings			
55								
60								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/24/2021

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/6 in.	SAMPLES: Sample Number	USCS Symbol	BORING NO.: B-6 / MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium: Disced in upper one foot			
		9 12 16	R1	ML	Sandy SILT, brown, moist, very stiff	3.1	112.8	
5		6 20 30	R2		red-brown, moist, hard	7.3	117.2	
		20 18 14	R3	SM	Silty f SAND, red-brown, moist, medium dense	6.2	105.5	
10		3 5 8	R4	SP	F SAND with some SILT, olive brown, moist, medium dense	3.1	107.5	
15		17 27 36	R5	SM	Silty f SAND with trace CLAY, red-brown, moist, dense			
20		18 35 30	R6		Strace caliche stringers	6.9	104.2	
					BORING TERMINATED AT 21.5 FEET No groundwater encountered Boring backfilled with soil cuttings			
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/24/2021

BORING NO.: B-7

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/6 in.	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium: Disced in upper one foot			
	Ring	7 15 16	R1	ML	Sandy SILT, red-brown, moist, very stiff	3.9	110.9	
5	Ring	18 19 20	R2			4.4	107.1	
	Ring	9 10 11	R3	SM	Silty f-c SAND, brown, moist, medium dense, trace roothairs	2.4	115.7	HC
10	Ring	8 16 18	R4	SP	F-c SAND, light grey-brown, slightly moist, medium dense			
15	Ring	13 24 32	R5		dense	1.8	119.8	
20	Ring	15 37 47	R6	SC-SM	Silty to clayey SAND, red-brown, moist, very dense			
					BORING TERMINATED AT 21.5 FEET			
25					No groundwater encountered Boring backfilled with soil cuttings			
30								

LEGEND

Sample type: Ring, SPT, Small Bulk, Large Bulk, No Recovery, Water Table

Lab testing: AL = Atterberg Limits; SR = Sulfate/Resisitivity Test; EI = Expansion Index; SH = Shear Test; SA = Sieve Analysis; HC= Consolidation; RV = R-Value Test; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/24/2021

BORING NO.: B-8

Depth (ft)	Sample Type	Blows/6 in.	Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Water Content (%)	Dry Density (pcf)	Others
					Older Alluvium:			
					Disced in upper one foot			
		10 17 30	R1	ML	Sandy SILT, brown, moist, hard, trace caliche stringers	4.7	111.9	
		21 27 34	R2		red-brown	7.1	123.0	
5		6 9 11	R3	SM	Silty f-c SAND, red-brown, slightly moist, medium dense	2.2	109.2	
10		19 25 30	R5			8.2	114.9	
15		13 16 21	R5	SP	F-c SAND with trace SILT, gray-brown, slightly moist, medium dense			
20		50-6"		ML	Sandy SILT, red-brown, moist, hard, trace caliche stringers	8.5	116.4	
					BORING TERMINATED AT 20.5 FEET			
					No groundwater encountered			
					Boring backfilled with soil cuttings			
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/19/2021

BORING NO.: I-1

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/6 in.	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium: Disced in upper one foot			
				ML	Sandy SILT, brown, slightly moist to moist			
5					**BORING TERMINATED AT 5 FEET**			
					No groundwater encountered			
					Boring set with pipe, sock, and gravel			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/19/2021

BORING NO.: I-2

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/6 in.	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium: Disced in upper one foot			
				ML	Sandy SILT, brown, slightly moist to moist			
5					**BORING TERMINATED AT 5 FEET**			
					No groundwater encountered			
					Boring set with pipe, sock, and gravel			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/19/2021

BORING NO.: I-3

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/6 in.	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium:			
					Disced in upper one foot			
				ML	Sandy SILT, brown, slightly moist to moist			
					red-brown			
5					**BORING TERMINATED AT 5 FEET**			
					No groundwater encountered			
					Boring set with pipe, sock, and gravel			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/19/2021

BORING NO.: I-4

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/6 in.	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium:			
					Disced in upper one foot			
				ML	Sandy SILT, brown, slightly moist to moist			
					red-brown			
5					**BORING TERMINATED AT 5 FEET**			
					No groundwater encountered			
					Boring set with pipe, sock, and gravel			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/19/2021

BORING NO.: I-5

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/6 in.	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium: Disced in upper one foot			
				ML	Sandy SILT, brown, slightly moist to moist			
5					**BORING TERMINATED AT 5 FEET**			
					No groundwater encountered			
					Boring set with pipe, sock, and gravel			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/19/2021

BORING NO.: I-6

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/6 in.	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium:			
					Disced in upper one foot			
				ML	Sandy SILT, brown, slightly moist to moist			
5					**BORING TERMINATED AT 5 FEET**			
					No groundwater encountered			
					Boring set with pipe, sock, and gravel			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/19/2021

BORING NO.: I-7

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/6 in.	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium:			
					Disced in upper one foot			
				ML	Sandy SILT, brown, slightly moist to moist			
					red-brown			
5					**BORING TERMINATED AT 5 FEET**			
					No groundwater encountered			
					Boring set with pipe, sock, and gravel			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

GeoTek, Inc.

LOG OF EXPLORATORY BORING

CLIENT:	Oakdale Communities	**DRILLER:**	2R Drilling, Inc.	**LOGGED BY:**	JD
PROJECT NAME:	Darwin Court	**DRILL METHOD:**	HSA	**OPERATOR:**	Miguel
PROJECT NO.:	2950-CR	**HAMMER:**	140 lbs - 30 in	**RIG TYPE:**	CME 75
LOCATION:	Moreno Valley, CA			**DATE:**	11/19/2021

BORING NO.: I-8

Depth (ft)	SAMPLES: Sample Type	SAMPLES: Blows/6 in.	SAMPLES: Sample Number	USCS Symbol	MATERIAL DESCRIPTION AND COMMENTS	Laboratory Testing: Water Content (%)	Laboratory Testing: Dry Density (pcf)	Laboratory Testing: Others
					Older Alluvium:			
					Disced in upper one foot			
				ML	Sandy SILT, brown, slightly moist to moist			
					red-brown			
5					**BORING TERMINATED AT 5 FEET**			
					No groundwater encountered			
					Boring set with pipe, sock, and gravel			
10								
15								
20								
25								
30								

LEGEND

Sample type: ---Ring ---SPT ---Small Bulk ---Large Bulk ---No Recovery ---Water Table

Lab testing: AL = Atterberg Limits; EI = Expansion Index; SA = Sieve Analysis; RV = R-Value Test; SR = Sulfate/Resisitivity Test; SH = Shear Test; HC= Consolidation; MD = Maximum Density

Client:	**Oakdale Communities**
Project:	**Darwin Court Project**
Project No:	**2950-CR**
Date:	**12/10/2021**

Boring No. **I-1**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	44
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	40
Total Test Hole Depth, D_T =	60

Equation - $I_t = \dfrac{\Delta H\ (60r)}{\Delta t\ (r+2H_{avg})}$

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	16
$\Delta H = \Delta D = H_O - H_F$ =	4
$H_{avg} = (H_O+H_F)/2$ =	18

I_t = 0.80 **Inches per Hour**

GEOTEK

Client:	**Oakdale Communities**
Project:	**Darwin Court Project**
Project No:	**2950-CR**
Date:	**12/10/2021**

Boring No. **I-2**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	44
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	40
Total Test Hole Depth, D_T =	60

Equation - $I_t = \dfrac{\Delta H\ (60r)}{\Delta t\ (r+2H_{avg})}$

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	16
$\Delta H = \Delta D = H_O - H_F$ =	4
$H_{avg} = (H_O+H_F)/2$ =	18

I_t = 0.80 **Inches per Hour**

GEOTEK

Client:	**Oakdale Communities**
Project:	**Darwin Court Project**
Project No:	**2950-CR**
Date:	**12/10/2021**

Boring No. **I-3**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	44.25
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	40
Total Test Hole Depth, D_T =	60

Equation - $I_t = \dfrac{\Delta H\ (60r)}{\Delta t\ (r+2H_{avg})}$

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	15.75
$\Delta H = \Delta D = H_O - H_F$ =	4.25
$H_{avg} = (H_O+H_F)/2$ =	17.875

I_t = 0.86 **Inches per Hour**

GEOTEK

Client:	**Oakdale Communities**
Project:	**Darwin Court Project**
Project No:	**2950-CR**
Date:	**12/10/2021**

Boring No. **I-4**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	44.25
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	40
Total Test Hole Depth, D_T =	60

Equation - $I_t = \dfrac{\Delta H\,(60r)}{\Delta t\,(r+2H_{avg})}$

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	15.75
$\Delta H = \Delta D = H_O - H_F$ =	4.25
$H_{avg} = (H_O+H_F)/2$ =	17.875

I_t = 0.86 **Inches per Hour**

GEOTEK

Client:	**Oakdale Communities**
Project:	**Darwin Court Project**
Project No:	**2950-CR**
Date:	**12/10/2021**

Boring No. **I-5**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	43.5
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	40
Total Test Hole Depth, D_T =	60

Equation - $I_t = \dfrac{\Delta H\ (60r)}{\Delta t\ (r+2H_{avg})}$

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	16.5
$\Delta H = \Delta D = H_O - H_F$ =	3.5
$Havg = (H_O+H_F)/2$ =	18.25

I_t = 0.69 **Inches per Hour**

GEOTEK

Client:	**Oakdale Communities**
Project:	**Darwin Court Project**
Project No:	**2950-CR**
Date:	**12/10/2021**

Boring No. **I-6**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	44
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	40
Total Test Hole Depth, D_T =	60

Equation - $I_t = \dfrac{\Delta H\ (60r)}{\Delta t\ (r+2H_{avg})}$

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	16
$\Delta H = \Delta D = H_O - H_F$ =	4
$H_{avg} = (H_O+H_F)/2$ =	18

I_t = 0.80 **Inches per Hour**

GEOTEK

Client:	**Oakdale Communities**
Project:	**Darwin Court Project**
Project No:	**2950-CR**
Date:	**12/10/2021**

Boring No. **I-7**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	44.25
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	40
Total Test Hole Depth, D_T =	60

Equation - $I_t = \dfrac{\Delta H\,(60r)}{\Delta t\,(r+2H_{avg})}$

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	15.75
$\Delta H = \Delta D = H_O - H_F$ =	4.25
$H_{avg} = (H_O+H_F)/2$ =	17.875

I_t = 0.86 **Inches per Hour**

GEOTEK

Client:	**Oakdale Communities**
Project:	**Darwin Court Project**
Project No:	**2950-CR**
Date:	**12/10/2021**

Boring No. **I-8**

Infiltration Rate (Porchet Method)

Time Interval, Δt =	30
Final Depth to Water, D_F =	44.5
Test Hole Radius, r =	4
Initial Depth to Water, D_O =	40
Total Test Hole Depth, D_T =	60

Equation - $I_t = \dfrac{\Delta H\,(60r)}{\Delta t\,(r+2H_{avg})}$

$H_O = D_T - D_O$ =	20
$H_F = D_T - D_F$ =	15.5
$\Delta H = \Delta D = H_O - H_F$ =	4.5
$H_{avg} = (H_O+H_F)/2$ =	17.75

I_t = 0.91 **Inches per Hour**

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APPENDIX B

APPENDIX B
LABORATORY TESTING

In 2021, GeoTek performed laboratory testing consisting of collapse, R-value, maximum dry density and optimum moisture content, in-situ dry density and moisture content, expansion index, direct shear, and corrosivity. The 2021 GeoTek laboratory test results are presented herein.

SUMMARY OF LABORATORY TESTING

Classification

Soils were classified visually in general accordance with the Unified Soil Classification System (ASTM Test Method D 2487). The soil classifications are shown on the logs of borings in Appendix A.

Collapse Test

Collapse tests were performed on selected samples of the site soils in general accordance with ASTM D 5333 test procedures. The results of this test are presented graphically in Appendix B.

Direct Shear

Shear testing was performed in a direct shear machine of the strain-control type in general accordance with ASTM D 3080 test procedures. The rate of deformation was approximately 0.035 inch per minute. The sample was sheared under varying confining loads in order to determine the coulomb shear strength parameters, angle of internal friction and cohesion. The tests were performed on soil samples remolded to approximately 90 percent of maximum dry density as determined by ASTM D 1557 test procedures. The shear test results are presented in Appendix B.

Expansion Index

Expansion Index testing was performed two soil samples. Testing was performed in general accordance with ASTM Test Method D 4829. The results of the testing are provided below and in Appendix B.

Boring No.	Depth (ft.)	Description	Expansion Index	Classification
B-2	0-5	Sandy Silt (ML)	10	Very Low
B-8	0-5	Sandy Silt (ML)	3	Very Low

In-Situ Moisture and Density

The natural water content of sampled soils was determined in general accordance with ASTM D 2216 test procedures on samples of the materials recovered from the subsurface exploration. In addition, in-place dry density of the sampled soils was determined in general accordance with ASTM D 2937 test procedures on relatively undisturbed samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths in Appendix A.

Moisture-Density Relationship

Laboratory testing was performed on two samples collected during the subsurface exploration. The laboratory maximum dry density and optimum moisture content for the soil type was determined in general accordance with ASTM D 1557 test procedures. The results are presented in Appendix B.

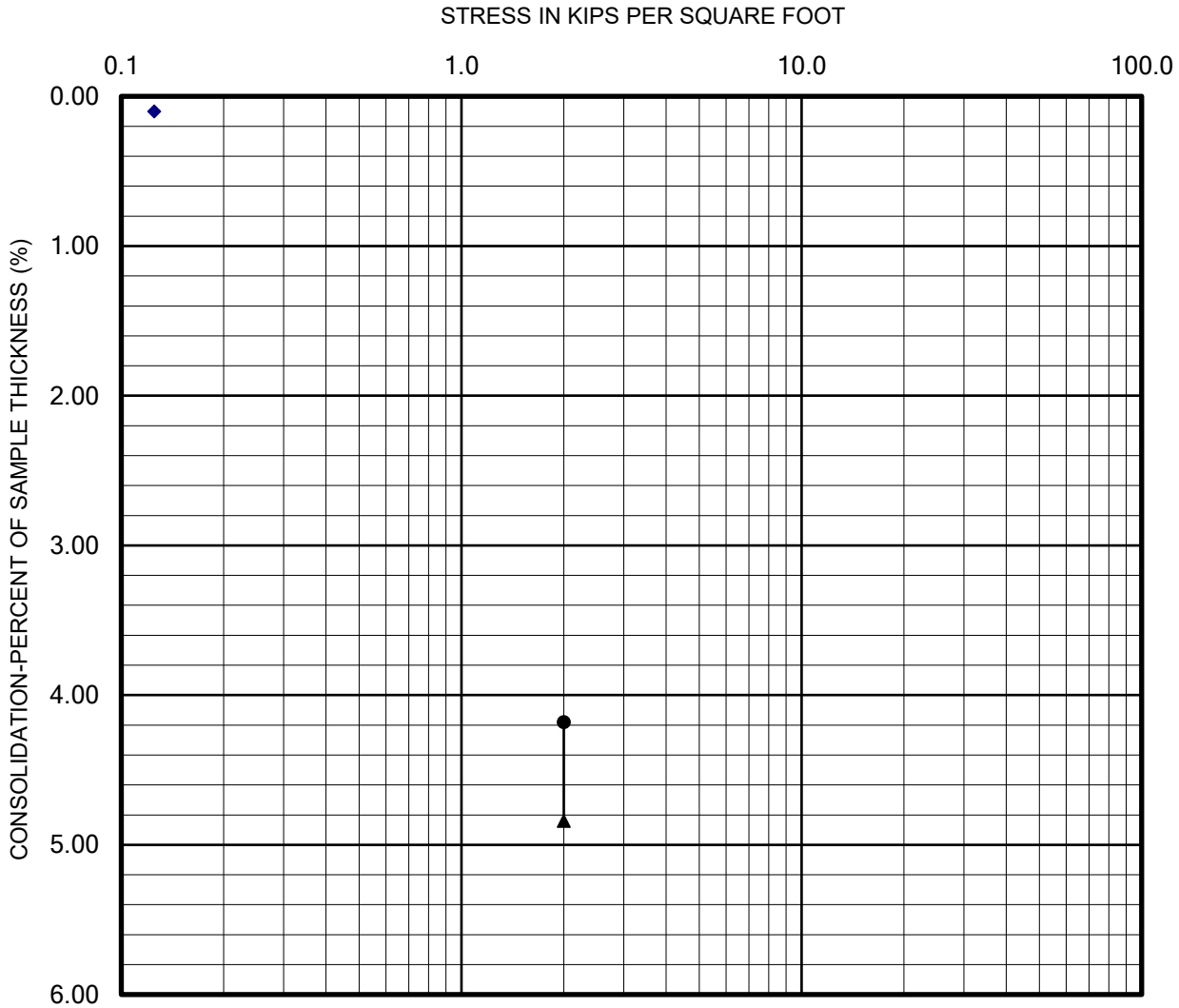
R-Value

One sample of the site soils was tested to determine its R-value characteristics in accordance with California Test Method 301. The test results are provided in Appendix B.

Sulfate Content, Resistivity and Chloride Content

Testing to determine the water-soluble sulfate content was performed by others in general accordance with ASTM D4327 test procedures. Resistivity testing was completed by others in general accordance with ASTM G187 test procedures. Testing to determine the chloride content was performed by others in general accordance with ASTM D4327 test procedures. The results of the testing are provided below and in Appendix B.

Boring No.	Depth (ft.)	pH ASTM D4972	Chloride ASTM D4327 (mg/kg)	Sulfate ASTM D4327 (% by weight)	Resistivity ASTM G187 (ohm-cm)
B-3	0-5	8.7	35.2	0.0001	4,020
B-5	0-5	7.9	63.9	0.0003	2,747



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

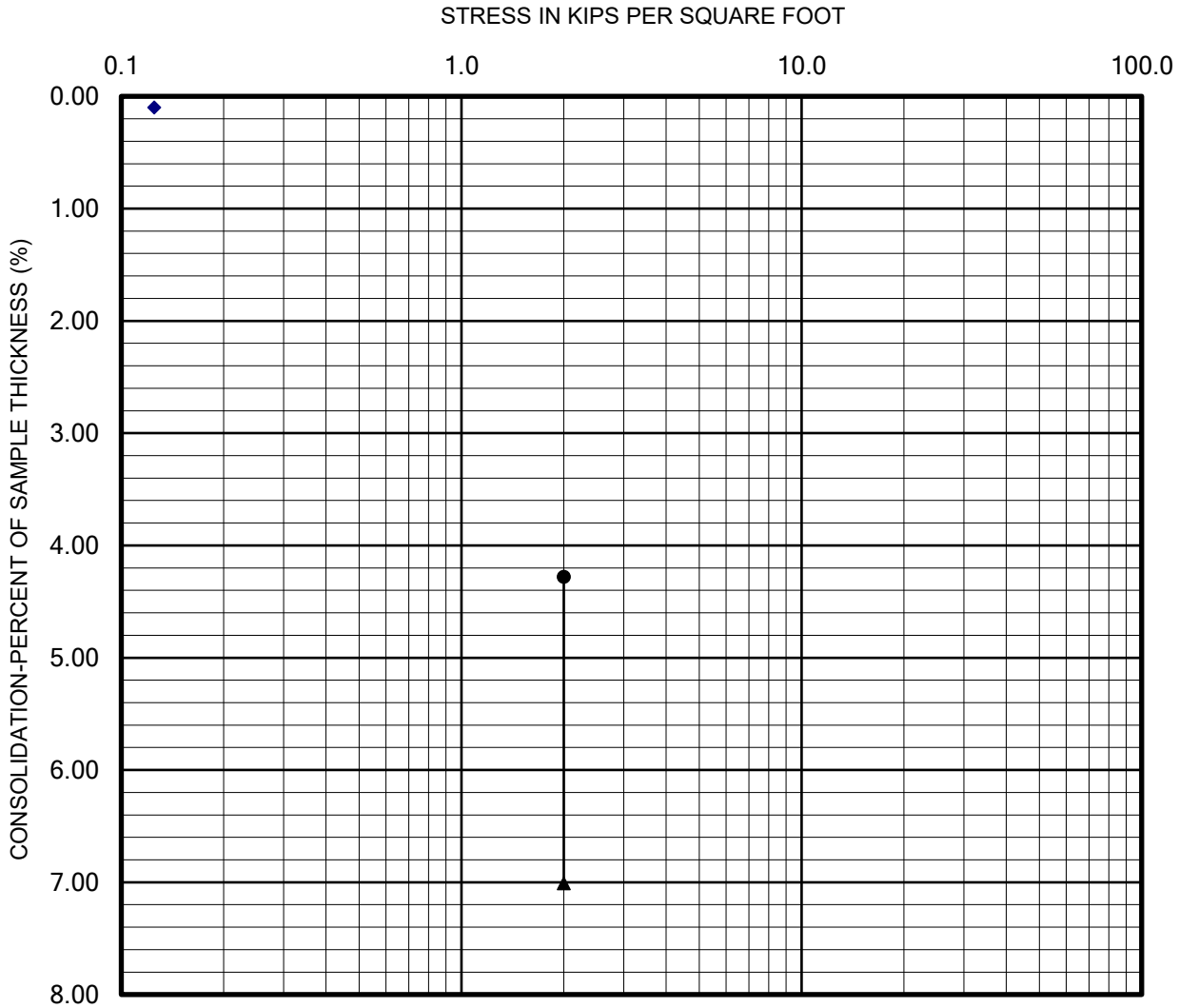
GEOTEK

CHECKED BY: DA	Lab: Corona
PROJECT NO.: 2950-CR	Date: 12/14/202

COLLAPSE REPORT

Sample: B-3 @ 5 feet

Plate C-1



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

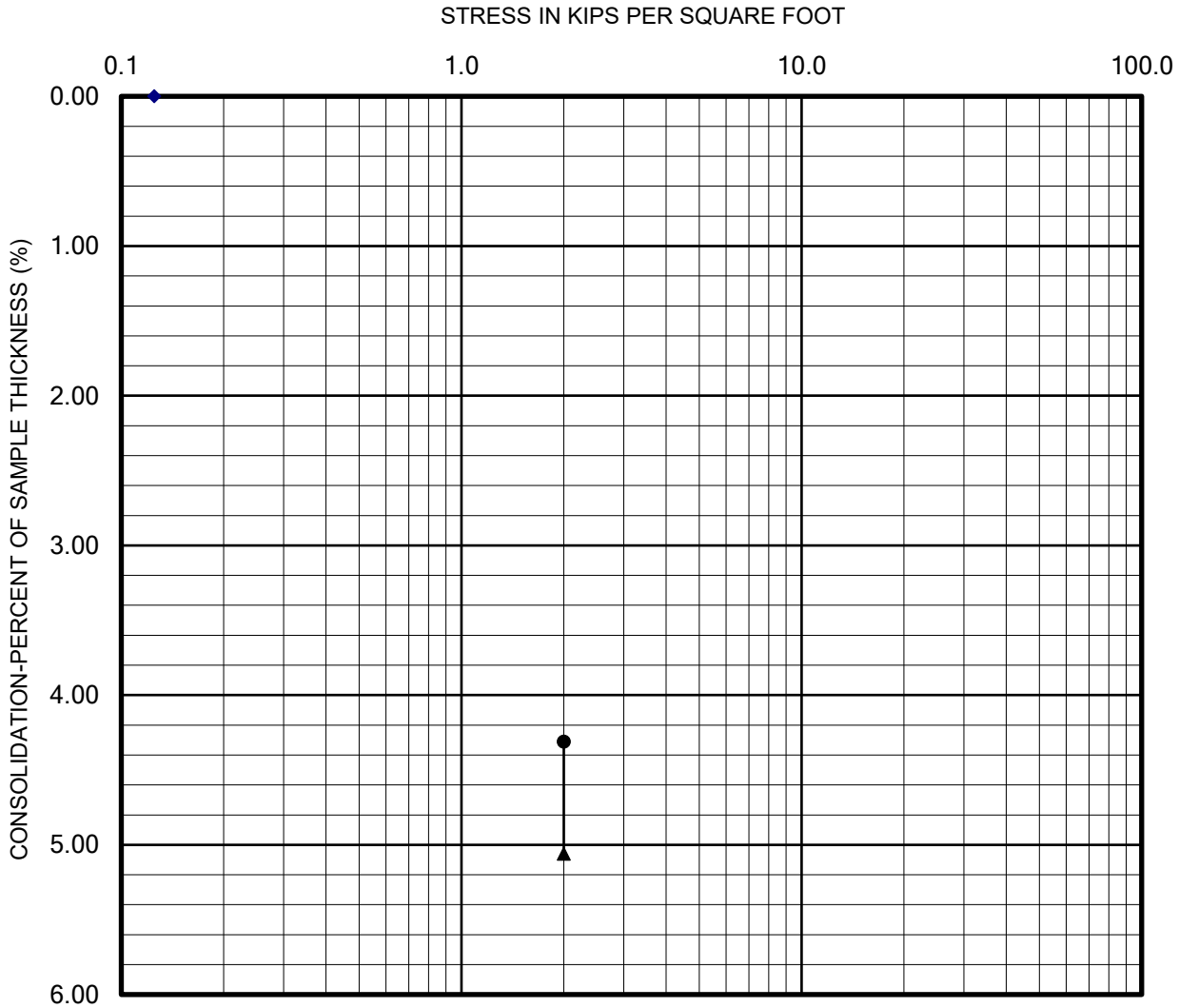
GEOTEK

COLLAPSE REPORT

Sample: B-5 @ 3 feet

Plate C-2

CHECKED BY: DA	Lab: Corona
PROJECT NO.: 2950-CR	Date: 12/14/202



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4546

GEOTEK

CHECKED BY: DA	Lab: Corona
PROJECT NO.: 2950-CR	Date: 12/14/202

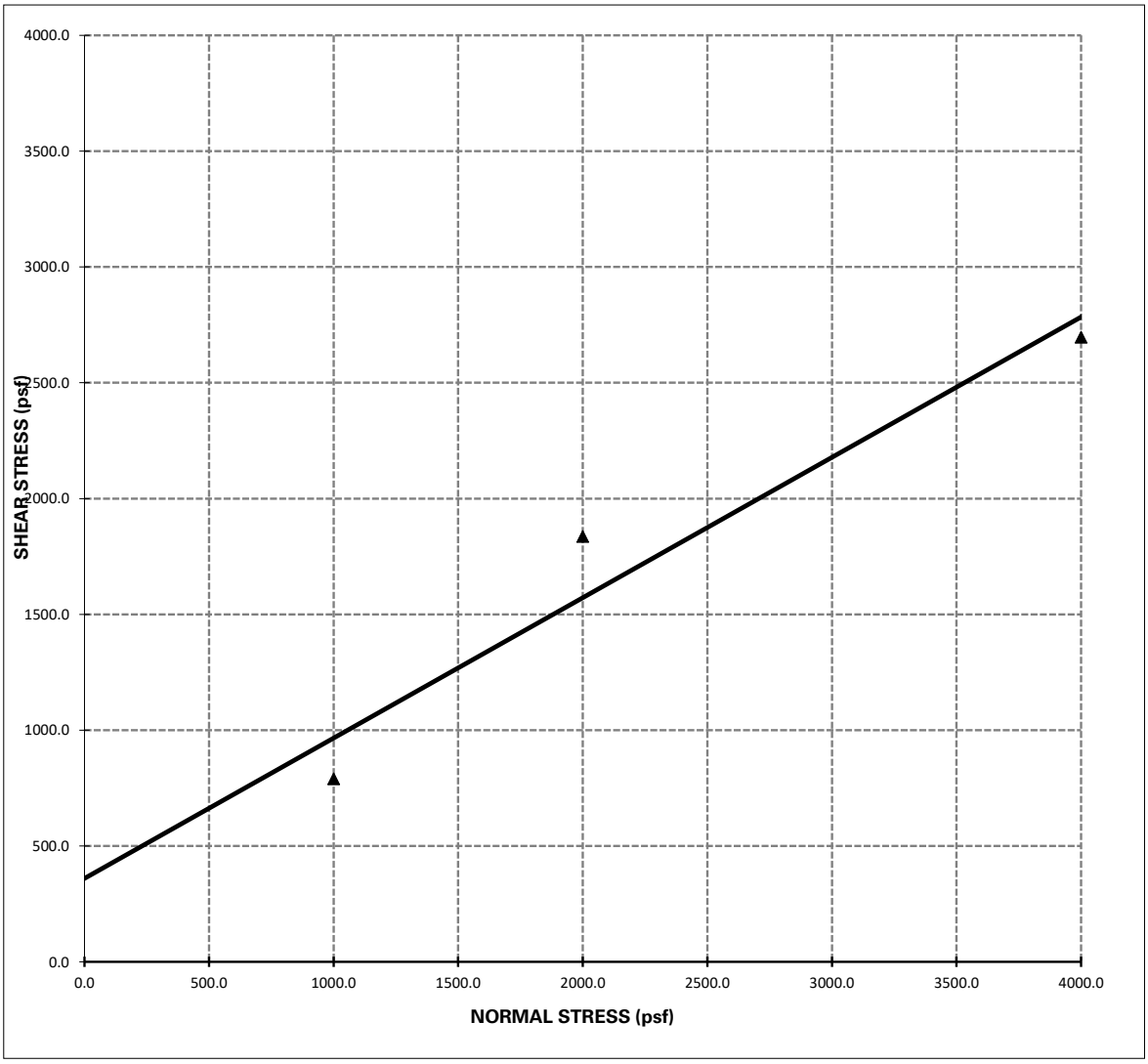
COLLAPSE REPORT

Sample: B-7 @ 6 feet

Plate C-3

DIRECT SHEAR TEST

Project Name:	Oakdale Communities	***Sample Location:***	B-8 @ 0-5 feet
Project Number:	Darwin Court Project	***Date Tested:***	12/14/2021



Shear Strength: Φ = **31°** **C = 360 psf**

Notes: 1 - The soil specimen used in the shear box was a ring sample remolded to approximately 90% relative compaction from a bulk sample collected during the field investigation.

2 - The above reflect direct shear strength at saturated conditions.

3 - The tests were run at a shear rate of 0.01 in/min.

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client:	Oakdale Communities	***Tested/ Checked By:***	ADLC Lab No Corona
Project Number:	2950-CR	***Date Tested:***	12/8/2021
Project Location:	Darwin Court Project, Moreno Valley	***Sample Source:***	B-2 @ 0-5 feet
		Sample Description:	

Ring #:_____ Ring Dia. : 4.01" Ring Ht.:1"

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	784.3
B	Weight of ring (gm)	362.3
C	Net weight of sample (gm)	**422.0**
D	Wet Density, lb / ft3 (C*0.3016)	**127.3**
E	Dry Density, lb / ft3 (D/1.F)	**118.2**

SATURATION DETERMINATION

F	Moisture Content, %	7.7
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20°C, (pcf)	**62.4**
I	% Saturation	**48.8**

READINGS

DATE	TIME	READING	
12/8/2021		0.2060	Initial
12/8/2021		0.2040	10 min/Dry
12/9/2021		0.2140	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
806.1	**12.9**

EXPANSION INDEX = 10

GEOTEK

EXPANSION INDEX TEST

(ASTM D4829)

Client:	Oakdale Communities	***Tested/ Checked By:***	ADLC Lab No Corona
Project Number:	2950-CR	***Date Tested:***	12/8/2021
Project Location:	Darwin Court Project, Moreno Valley	***Sample Source:***	B-8 @ 0-5 feet
		Sample Description:	

Ring #:_____ Ring Dia. : 4.01" Ring Ht.:1"

DENSITY DETERMINATION

A	Weight of compacted sample & ring (gm)	801.3
B	Weight of ring (gm)	366.4
C	Net weight of sample (gm)	**434.9**
D	Wet Density, lb / ft3 (C*0.3016)	**131.2**
E	Dry Density, lb / ft3 (D/1.F)	**122.4**

SATURATION DETERMINATION

F	Moisture Content, %	7.2
G	Specific Gravity, assumed	**2.70**
H	Unit Wt. of Water @ 20°C, (pcf)	**62.4**
I	% Saturation	**51.6**

READINGS

DATE	TIME	READING	
12/8/2021		0.8050	Initial
12/8/2021		0.8050	10 min/Dry
12/9/2021		0.8080	Final

FINAL MOISTURE

Final Weight of wet sample & tare	% Moisture
817.0	**10.8**

EXPANSION INDEX = 3

GEOTEK

MOISTURE/DENSITY RELATIONSHIP

Client: Oakdale Communities
Project: Darwin Court Project
Location: Moreno Valley
Material Type: Gray brown fine to medium sand with silt
Material Supplier: -
Material Source: -
Sample Location: B-2 @ 0-5 feet
-

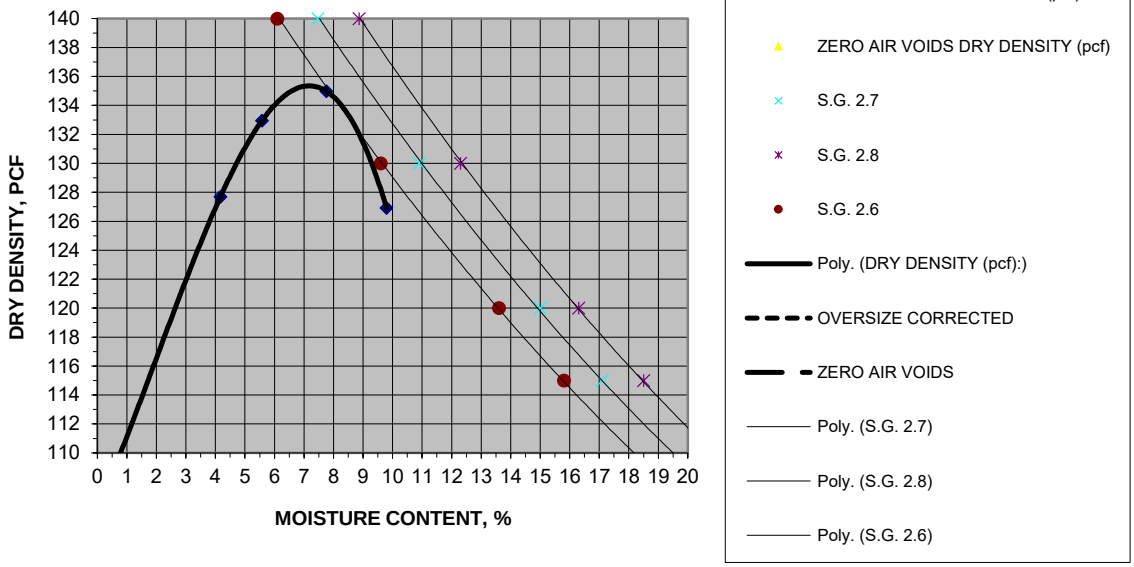
Job No.: 2950-CR
Lab No.: Corona

Sampled By: JD
Received By: ADLC
Tested By: JD/BL
Reviewed By: DA

Date Sampled: 12/9/2021
Date Received: 12/10/2021
Date Tested: 12/13/2021
Date Reviewed: 12/14/2021

Test Procedure: ASTM D1557 **Method:** A
Oversized Material (%): 1.9 **Correction Required:** [ ] yes [x] no

MOISTURE/DENSITY RELATIONSHIP CURVE



MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	135.5	**@ Optimum Moisture, %**	7.0
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

	% Gravel (retained on No. 4)
	% Sand (Passing No. 4, Retained on No. 200)
	% Silt and Clay (Passing No. 200)

Atterberg Limits:

	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %

Classification:

Unified Soils Classification:
AASHTO Soils Classification:

GEOTEK

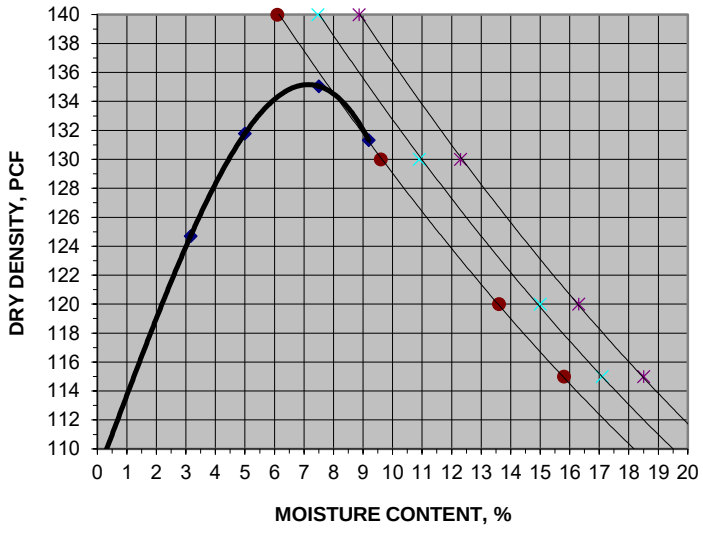
MOISTURE/DENSITY RELATIONSHIP

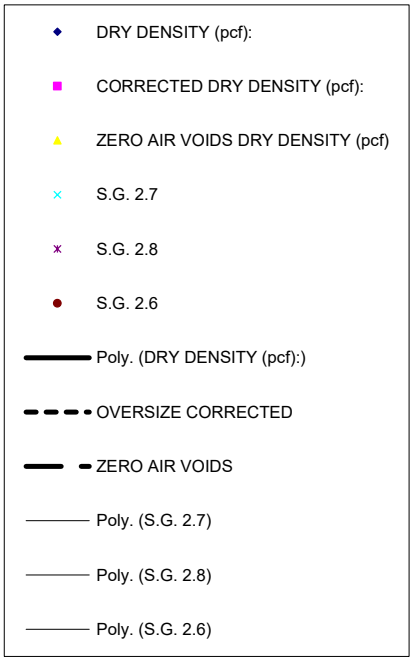
Client:	Oakdale Communities	**Job No.:**	2950-CR
Project:	Darwin Court Project	**Lab No.:**	Corona
Location:	Moreno Valley		
Material Type:	Gray brown silty sand		
Material Supplier:	-		
Material Source:	-		
Sample Location:	B-5 @ 0-5 feet		
	-		
Sampled By:	JD	**Date Sampled:**	12/9/2021
Received By:	ADLC	**Date Received:**	12/10/2021
Tested By:	JD/BL	**Date Tested:**	12/13/2021
Reviewed By:	DA	**Date Reviewed:**	12/14/2021

Test Procedure: ASTM D1557 **Method:** A

Oversized Material (%): 2.0 **Correction Required:** [ ] yes [x] no

MOISTURE/DENSITY RELATIONSHIP CURVE





MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	135.5	**@ Optimum Moisture, %**	7.0
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

	% Gravel (retained on No. 4)
	% Sand (Passing No. 4, Retained on No. 200)
	% Silt and Clay (Passing No. 200)

Atterberg Limits:

	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %

Classification:

Unified Soils Classification: _____

AASHTO Soils Classification: _____

GEOTEK

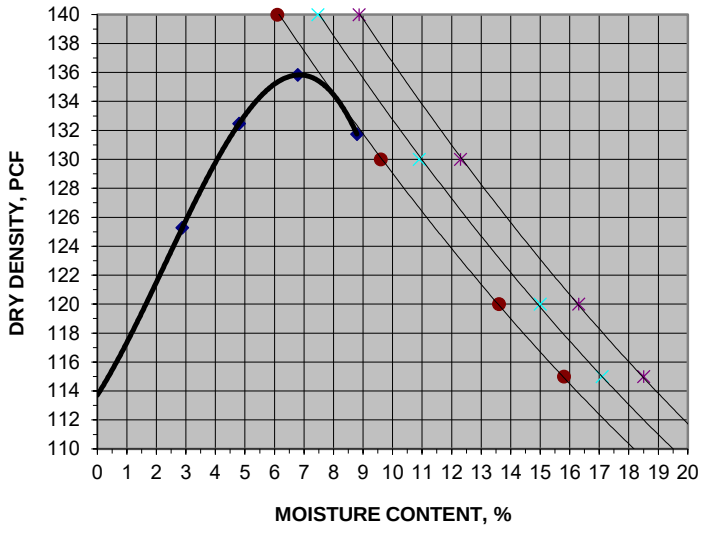
MOISTURE/DENSITY RELATIONSHIP

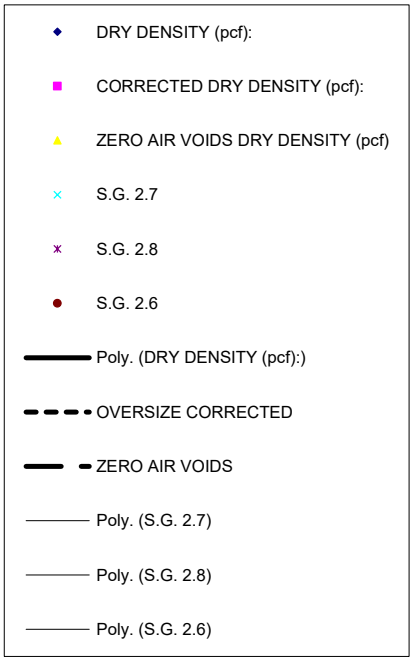
Client:	Oakdale Communities	**Job No.:**	2950-CR
Project:	Darwin Court Project	**Lab No.:**	Corona
Location:	Moreno Valley		
Material Type:	Gray brown silty sand		
Material Supplier:	-		
Material Source:	-		
Sample Location:	B-8 @ 0-5 feet		
	-		
Sampled By:	JD	**Date Sampled:**	12/9/2021
Received By:	ADLC	**Date Received:**	12/10/2021
Tested By:	JD/BL	**Date Tested:**	12/13/2021
Reviewed By:	DA	**Date Reviewed:**	12/14/2021

Test Procedure: ASTM D1557 **Method:** A

Oversized Material (%): 2.0 **Correction Required:** [ ] yes [x] no

MOISTURE/DENSITY RELATIONSHIP CURVE





MOISTURE DENSITY RELATIONSHIP VALUES

Maximum Dry Density, pcf	136.0	**@ Optimum Moisture, %**	7.0
Corrected Maximum Dry Density, pcf		**@ Optimum Moisture, %**	

MATERIAL DESCRIPTION

Grain Size Distribution:

	% Gravel (retained on No. 4)
	% Sand (Passing No. 4, Retained on No. 200)
	% Silt and Clay (Passing No. 200)

Atterberg Limits:

	Liquid Limit, %
	Plastic Limit, %
	Plasticity Index, %

Classification:

Unified Soils Classification:

AASHTO Soils Classification:

Results Only Soil Testing for Darwin Court Project, Moreno Valley

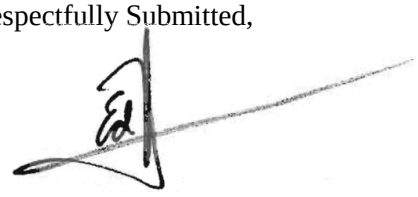
December 2, 2021

Prepared for:

Kyle McHargue
GeoTek, Inc.
1548 North Maple Street
Corona, CA 92280
kmchargue@geotekusa.com

Project X Job#: S211201D
Client Job or PO#: 2950-CR Oakdale Communities

Respectfully Submitted,



Eduardo Hernandez, M.Sc., P.E.
Sr. Corrosion Consultant
NACE Corrosion Technologist #16592
Professional Engineer
California No. M37102
ehernandez@projectxcorrosion.com



Soil Analysis Lab Results

Client: GeoTek, Inc.
Job Name: Darwin Court Project, Moreno Valley
Client Job Number: 2950-CR Oakdale Communities
Project X Job Number: S211201D
December 2, 2021

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM D4972	ASTM G200	ASTM D4658	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd | Minimum		**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^{--}	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
2950-CR B5 @	0-5	3.3	0.0003	63.9	0.0064	80,400	2,747	7.9	241	0.04	0.9	ND	0.03	53.9	3.2	36.4	38.7	2.6	323.8
2950-CR B3@	0-5	1.2	0.0001	35.2	0.0035	167,500	4,020	8.7	257	0.17	1.0	0.8	0.03	81.8	20.9	35.3	26.9	5.7	278.3

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract
PPM = mg/kg (soil) = mg/L (Liquid)

Project X
Corrosion Engineering
Corrosion Control - Soil, Water, and Metallurgy Lab

Lab Request Sheet Chain of Custody
Phone: (213) 928-7213 · Fax (951) 226-1720 · www.projectxcorrosion.com

Ship Samples To: 29990 Technology Dr, Suite 13, Murrieta, CA 92563

Project X Job Number: S211201D GEOTEK 2950-CR DARWIN 2FULL

IMPORTANT: Please complete Project and Sample Identification Data as you would like it to appear in report & include this form with samples.

Company Name:	Geotek USA	Contact Name:	Kyle McHargue
Mailing Address:	1548 N. Maple Ave. Corona, CA	Phone No:	951-206-5443
Accounting Contact:		Contact Email:	kmchargue@geotekusa.com
Client Project No:	Oakdale Communities	Invoice Email:	
P.O. #:		Project Name:	Darwin Court Project, Moreno Valley

(Business Days) Turn Around Time: 3-5 Day Standard ✓ | 3 Day Guarantee 50% mark-up | 2 Day RUSH 100% mark-up

Results By: ☐ Phone ☐ Fax ☑ Email

Date & Received by : RD 12-1-21 12/1/21

Default Method

Special Instructions:

ANALYSIS REQUESTED (Please circle)

	SAMPLE ID - BORE #		DEPTH (ft)	DATE COLLECTED
1	2950-CR	B5@	0-5	
2	2950-CR	B3@	0-5	
3				
4				
5				
6				
7				
8				
9				
10				
11				
12				
13				
14				
15				

7
8

Geo Quad				Full Corrosion Series														Full Corrosion Series	Reports										
Soil Resistivity	pH	Sulfate	Chloride	Redox Potential	Sulfide	Ammonia	Nitrate	Flouride	Phosphate	Lithium	Sodium	Potassium	Magnesium	Calcium	BiCarbonate				Soil Corrosivity Evaluation Report	Water Corrosivity Mini Report	Moisture Content	Total Alkalinity	Thermal Resistivity	(ANC) Alk Neutr Cap	(SRB) Sulfate Reducing	Puckorius Index	XRF Elemental Analysis	Water Hardness
Caltrans CTM643	Caltrans CTM643	Caltrans CTM417	Caltrans CTM422																		ASTM D2216							
AASHTO T288	AASHTO T 289	AASHTO T 290	AASHTO T 291	SM 2580B		SM 4500-NH3	SM 4500-NO3														SM 2320B		ASTM D5334				ASTM E1621	ASTM E132	ASTM A751
ASTM G187	ASTM G 51	ASTM D4327	ASTM D4327	ASTM G200	SM 4500-S2	ASTM D4327	ASTM D4327	SM 2320B	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	ASTM D4327	SM 2320B						IEEE 442		HACH BART				
																		✓											

*Req: Min. 3 Samples, site map, and groundwater info

• ANALYSIS
• DESIGN

LaBelle • Marvin
PROFESSIONAL PAVEMENT ENGINEERING
A CALIFORNIA CORPORATION

• SOILS, ASPHALT TECHNOLOGY

December 8, 2021

Mr. Kyle McHargue
GeoTek Inc.
1548 North Maple Street
Corona, California 92880

Project No. 47834

Attention Mr. McHargue:
Laboratory testing of the bulk soil sample delivered to our laboratory on 12/1/2021 has been completed.

Reference:	W.O. # 2950-CR
Project:	Oakdale Communities, Darwin Court
Sample:	B-5 @ 0'-5' Bulk

Data sheets are transmitted herewith for your use and information. Any untested portion of the samples will be retained for a period of sixty (60) days prior to disposal. The opportunity to be of service is appreciated, and should you have any questions, kindly call.

Very truly yours,

REGISTERED PROFESSIONAL ENGINEER
STEVEN R. MARVIN
NO. 30659
CIVIL
STATE OF CALIFORNIA

Steven R. Marvin
RCE 30659

SRM:tw
Enclosures

2700 S. GRAND AVENUE • SANTA ANA, CA 92705-5404 • (714) 546-3468 • FAX (714) 546-5841
INFO@LABELLEMARVIN.COM

R - VALUE DATA SHEET

LaBelle Marvin

PROJECT No. 47834

DATE: 12/8/2021

BORING NO. B-5 @ 0'-5' Bulk

Oakdale Communities, Darwin Court

W.O.# 2950-CR

SAMPLE DESCRIPTION: Brown Silty Fine Sand

R-VALUE TESTING DATA | CA TEST 301			
	SPECIMEN ID		
	a	b	c
Mold ID Number	7	8	9
Water added, grams	63	43	50
Initial Test Water, %	9.5	7.6	8.3
Compact Gage Pressure,psi	60	350	180
Exudation Pressure, psi	216	692	542
Height Sample, Inches	2.50	2.41	2.40
Gross Weight Mold, grams	3105	3084	2910
Tare Weight Mold, grams	1951	1947	1770
Sample Wet Weight, grams	1154	1137	1140
Expansion, Inches x 10exp-4	14	51	31
Stability 2,000 lbs (160psi)	27 / 60	18 / 33	20 / 38
Turns Displacement	4.98	3.91	4.22
R-Value Uncorrected	46	71	66
R-Value Corrected	46	70	64
Dry Density, pcf	127.7	132.8	132.9

DESIGN CALCULATION DATA

Traffic Index	Assumed:	4.0	4.0	4.0
G.E. by Stability		0.55	0.31	0.37
G. E. by Expansion		0.47	1.70	1.03

Equilibrium R-Value	**50** by EXPANSION	Examined & Checked: 12 /8/ 21
REMARKS:	Gf = 1.25 0.0% Retained on the 3/4" Sieve.	REGISTERED PROFESSIONAL ENGINEER STEVEN R. MARVIN NO. 30659 CIVIL STATE OF CALIFORNIA Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

LaBelle Marvin

R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 47834

DATE: 12 /8/ 2021

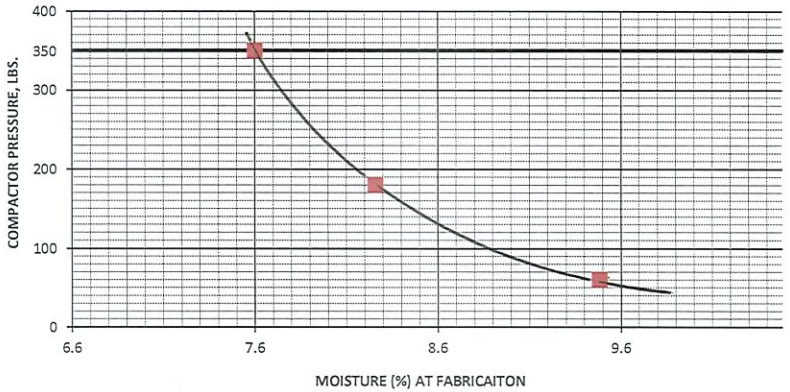
REMARKS:

BORING NO. B-5 @ 0'-5' Bulk

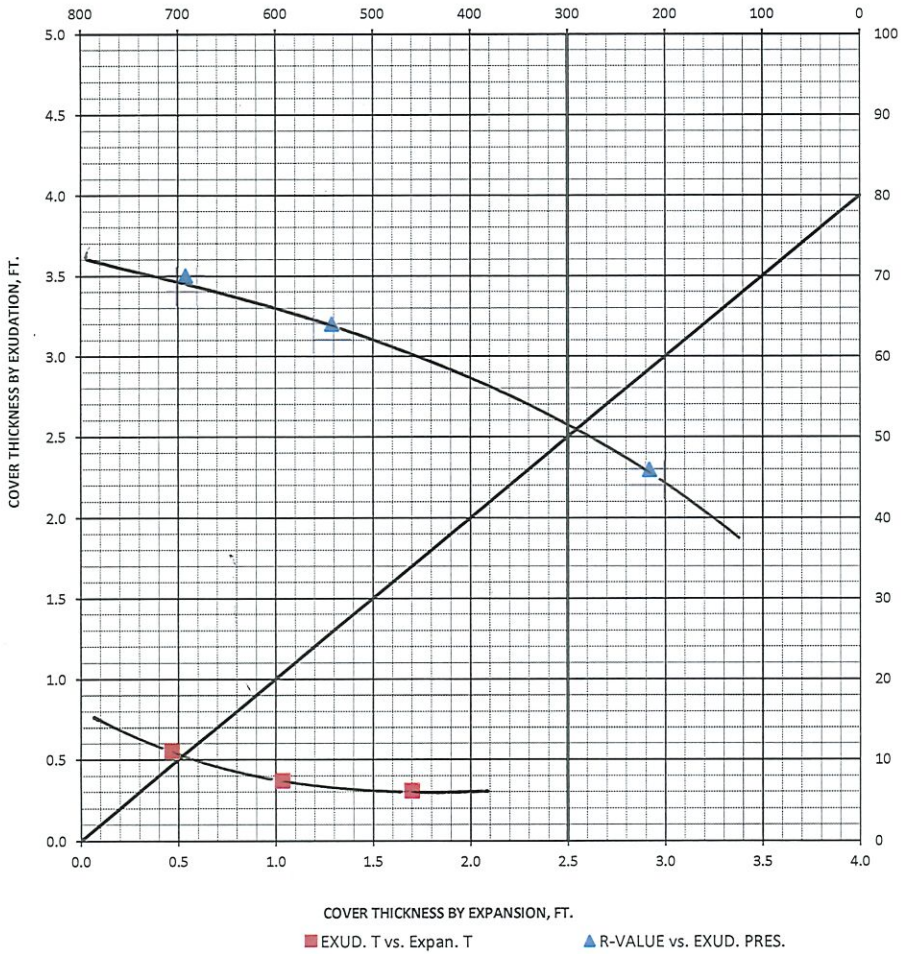
Oakdale Communities, Darwin Court

W.O.# 2950-CR

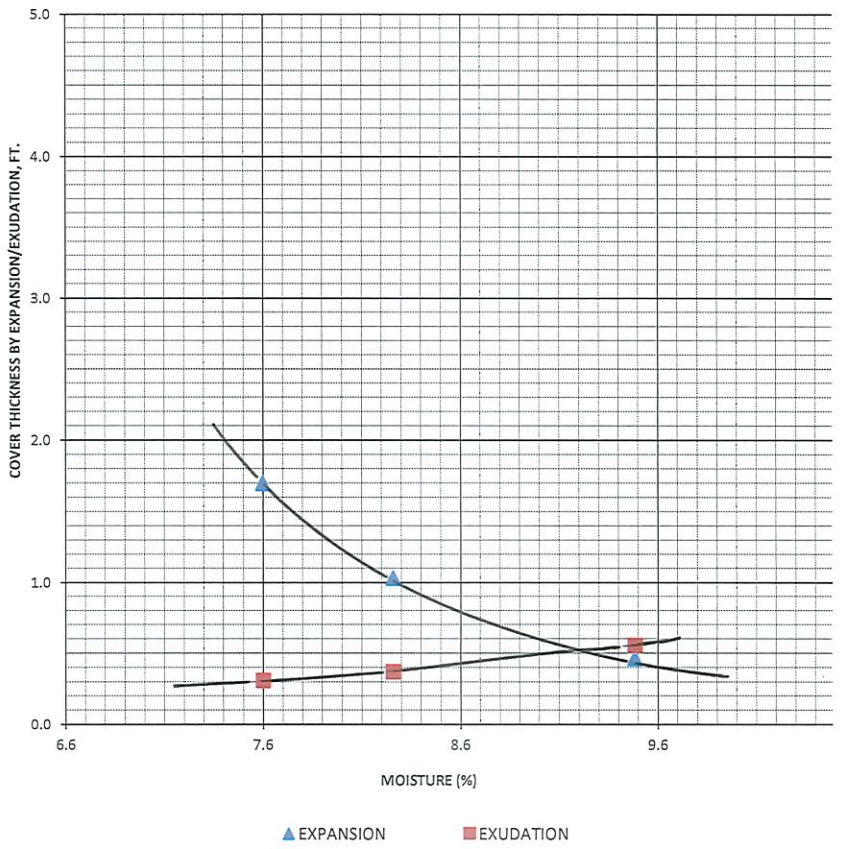
COMPACTOR PRESSURE vs MOISTURE %



COVER THICKNESS BY EXUDATION vs COVER THICKNESS BY EXPANSION



COVER THICKNESS vs MOISTURE %



APPENDIX C

APPENDIX C

RECOMMENDED GRADING SPECIFICATIONS

FOR

ROCAS GRANDES II
MULTI-FAMILY RESIDENTIAL DEVELOPMENT
SOUTHEAST OF ALESSANDRO BOULEVARD AND DARWIN DRIVE
MORENO VALLEY, CALIFORNIA

PROJECT NO. T2927-22-02A

RECOMMENDED GRADING SPECIFICATIONS

1. GENERAL

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

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

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

2. DEFINITIONS

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

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

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

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

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

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

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

3. MATERIALS

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

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

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

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

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

3.3 Materials used for fill, either imported or on-site, shall not contain hazardous materials as defined by the California Code of Regulations, Title 22, Division 4, Chapter 30, Articles 9

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

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

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

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

4. CLEARING AND PREPARING AREAS TO BE FILLED

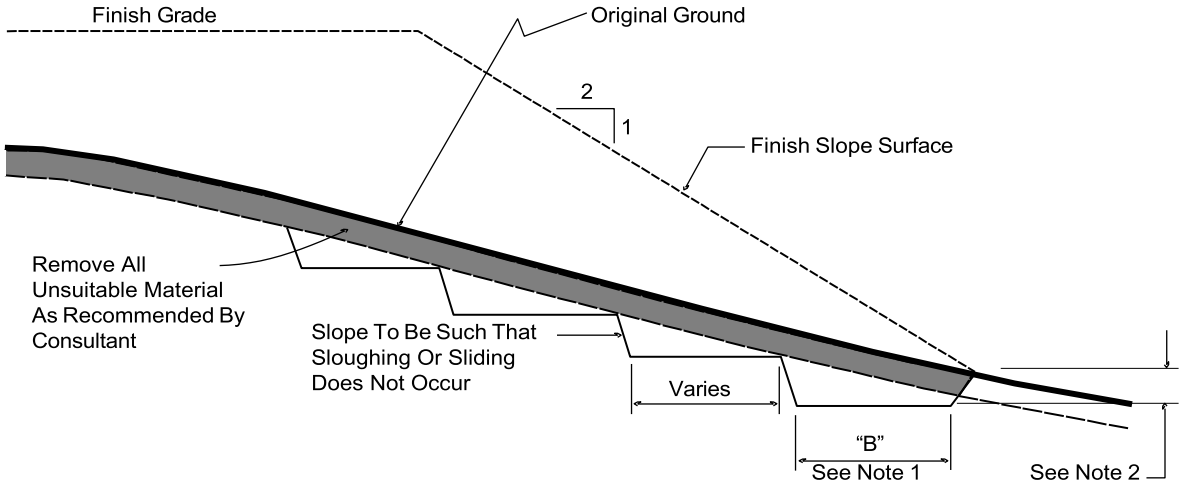
4.1 Areas to be excavated and filled shall be cleared and grubbed. Clearing shall consist of complete removal above the ground surface of trees, stumps, brush, vegetation, man-made structures, and similar debris. Grubbing shall consist of removal of stumps, roots, buried logs and other unsuitable material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

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

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

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

TYPICAL BENCHING DETAIL



No Scale

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

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

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

5. COMPACTION EQUIPMENT

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

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

6. PLACING, SPREADING AND COMPACTION OF FILL MATERIAL

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

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

6.3.3 Plate bearing tests, in accordance with ASTM D 1196, may be performed in both the compacted *soil* fill and in the *rock* fill to aid in determining the required minimum number of passes of the compaction equipment. If performed, a minimum of three plate bearing tests should be performed in the properly compacted *soil* fill (minimum relative compaction of 90 percent). Plate bearing tests shall then be performed on areas of *rock* fill having two passes, four passes and six passes of the compaction equipment, respectively. The number of passes required for the *rock* fill shall be determined by comparing the results of the plate bearing tests for the *soil* fill and the *rock* fill and by evaluating the deflection

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

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

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

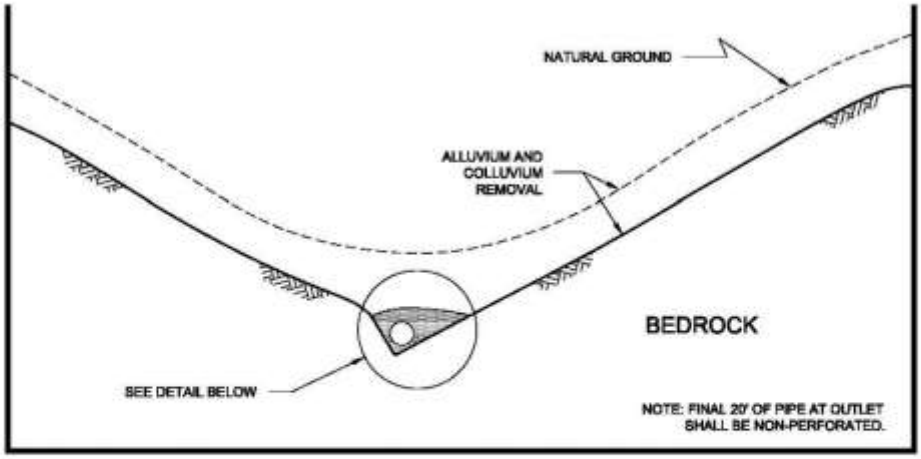
6.3.6 To reduce the potential for "piping" of fines into the *rock* fill from overlying *soil* fill material, a 2-foot layer of graded filter material shall be placed above the uppermost lift of *rock* fill. The need to place graded filter material below the *rock* should be determined by the Consultant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

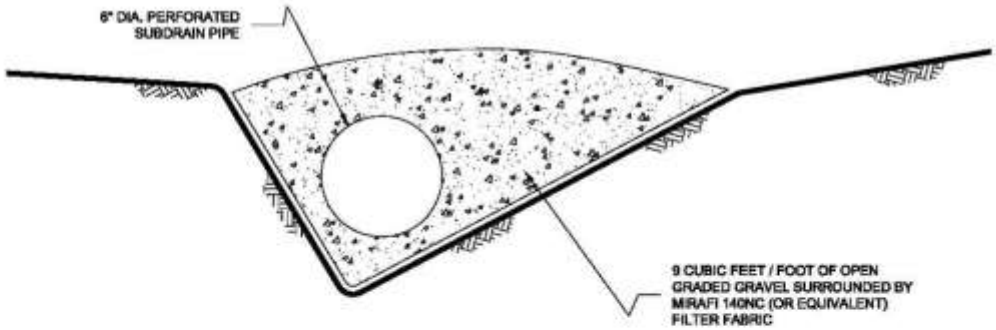
6.3.7 *Rock* fill placement should be continuously observed during placement by the Consultant.

7. SUBDRAINS

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

TYPICAL CANYON DRAIN DETAIL





NOTES:

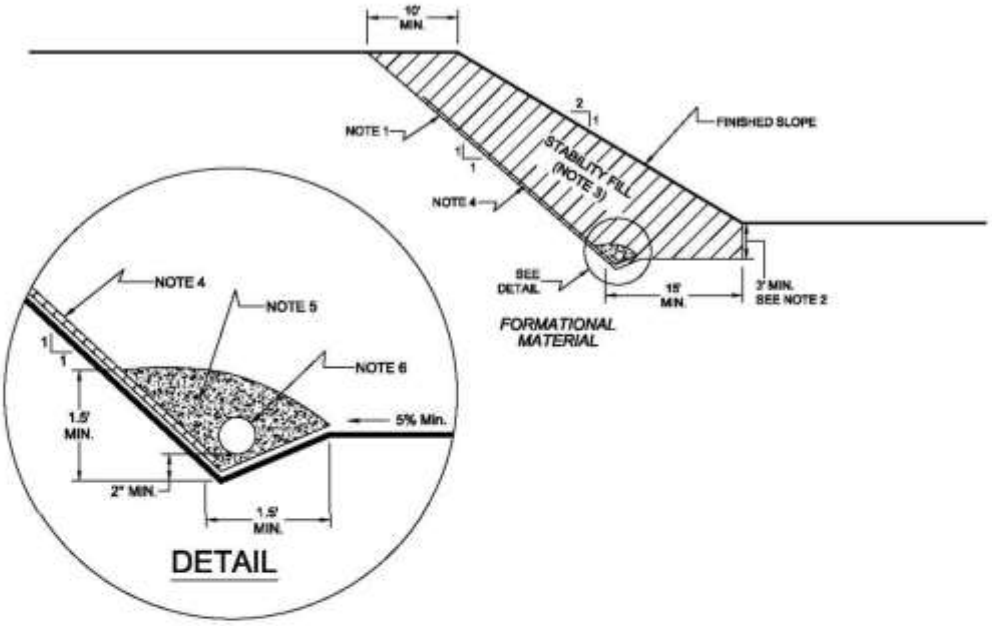
1.....8-INCH DIAMETER, SCHEDULE 80 PVC PERFORATED PIPE FOR FILLS IN EXCESS OF 100-FEET IN DEPTH OR A PIPE LENGTH OF LONGER THAN 500 FEET.

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

NO SCALE

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

TYPICAL STABILITY FILL DETAIL



NOTES:

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

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

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

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

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

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

NO SCALE

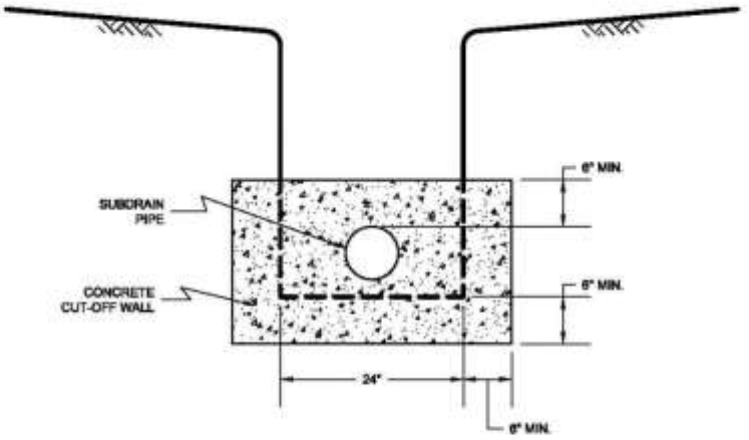
7.3 The actual subdrain locations will be evaluated in the field during the remedial grading operations. Additional drains may be necessary depending on the conditions observed and the requirements of the local regulatory agencies. Appropriate subdrain outlets should be evaluated prior to finalizing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

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

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

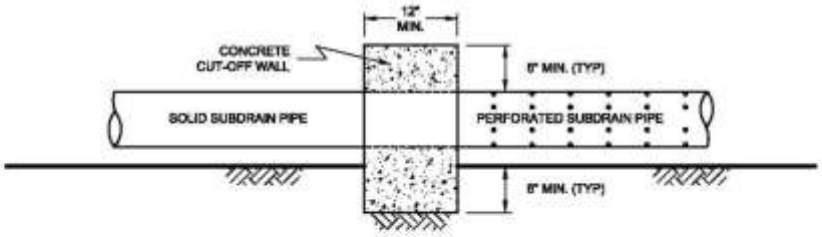
TYPICAL CUT OFF WALL DETAIL

FRONT VIEW



NO SCALE

SIDE VIEW

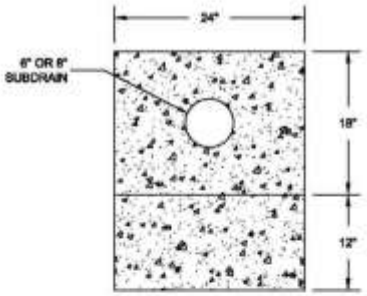


NO SCALE

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

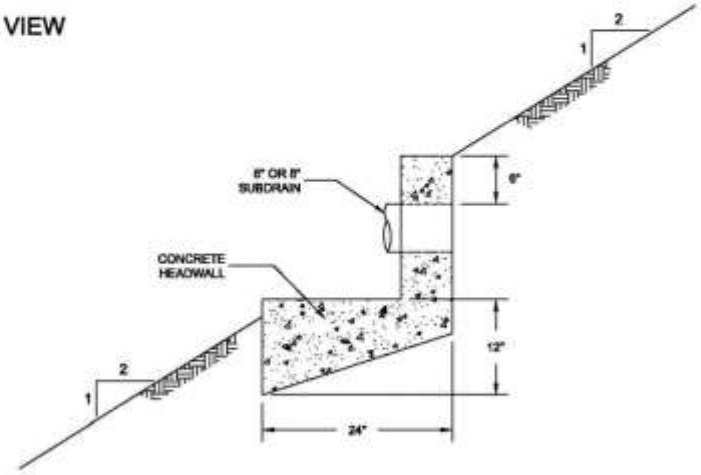
TYPICAL HEADWALL DETAIL

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.