Appendix C Limited Geotechnical Investigation

Appendix

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GEOCON
WEST, INC.

GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

LIMITED GEOTECHNICAL INVESTIGATION

PROPOSED IMPROVEMENTS TO EXISTING PARK
OAK CREEK COMMUNITY PARK
15616 VALLEY OAK DRIVE
IRVINE, CALIFORNIA

PREPARED FOR

SCHMIDT DESIGN GROUP, INC.
SAN DIEGO, CALIFORNIA

PROJECT NO. W1035-88-01

APRIL 14, 2022

Project No. W1035-88-01
April 14, 2022

Mr. Jeff Justus
Schmidt Design Group, Inc.
1310 Rosecrans Street, Suite G
San Diego, California 92106

Subject: LIMITED GEOTECHNICAL INVESTIGATION
PROPOSED IMPROVEMENTS TO EXISTING PARK
OAK CREEK COMMUNITY PARK
15616 VALLEY OAK DRIVE
IRVINE, CALIFORNIA 92618

Dear Mr. Justus:

In accordance with your authorization of our proposal, dated March 4, 2019, we have performed a limited geotechnical investigation for the proposed improvements to the Oak Creek Community Park located at 15616 Oak Valley Drive in the City of Irvine, California. The accompanying report presents the findings of our study, and our conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction. Based on the results of our investigation, it is our opinion that the project can be constructed as proposed provided the recommendations of this report are followed and implemented during design and construction.

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

Very truly yours,

GEOCON WEST, INC.

REGISTERED PROFESSIONAL ENGINEER
JELISA THOMAS ADAMS
GE3092
GEOTECHNICAL
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(EMAIL) Addressee

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LIMITED GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a limited geotechnical investigation for the proposed improvements to the Oak Creek Community Park located at 15616 Valley Oak Drive in the City of Irvine, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on March 7, 2022, by excavating five 3¼-inch diameter borings to depths ranging from approximately 5 to 6 feet below the existing ground surface using manual hand auger equipment and digging tools. An additional boring (B3A), was excavated near boring location B3 for percolation testing. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figures 2A and 2B). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The subject property is located at 15616 Valley Oak Drive in the City of Irvine, California. The property is bounded by Valley Oak Drive to the northwest, an asphalt parking lot to the east and northeast, Sand Canyon Avenue to the southeast, and Barranca Parkway to the west and southwest. The subject property is occupied by an existing roughly level SCE Easement with overhead powerline structures and the existing Oak Creek Community Park. Existing improvements within the park include a single-story restroom building, a single-story canopy structure, two soccer fields, an open grass area with a chainlink backstop for softball and baseball activities, playground equipment, asphalt paved parking lots, and concrete walkways. The area of proposed improvements is generally roughly level with no pronounced highs or lows. Within the open grass area, there are isolated, built-up mounds creating localized high areas. Additionally, there is a northwest to southeast trending berm that separates the SCE Easement from the existing park. The berm is roughly level with the existing soccer fields at the southeast extent and increases in height to approximately 5 feet at the northwest extent. Surface water drainage at the site appears to be by sheet flow along the existing ground contours to existing area drains and the city streets. Vegetation on site consists of shrubs, grass and trees.

Based on the information provided by the Client, it is our understanding that the proposed improvements will consist of: a new soccer field in the area of the existing open grass area with new field lighting; repurposing the southeast soccer field into a dog park; and a multipurpose field and asphalt parking lot within the SCE Easement. Additional improvements will consist of site lighting, fences, and concrete walkways. It is anticipated that the proposed field lighting for the soccer field will be supported on deepened foundations consisting of piles. This report assumes that the proposed athletic fields will be natural turf.

Based on the preliminary nature of the design at this time, design loads were not available. It is anticipated that the foundations supporting proposed improvements may require a bearing pressure up to 1,500 pounds per square foot.

Once information on the existing and proposed grades are available and the design phase and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Any changes in the design, location or elevation of any structure, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

3. SOIL AND GEOLOGIC CONDITIONS

Based on our field investigation and published geologic maps of the area, the site is underlain by artificial fill and unconsolidated Holocene age young alluvial fan deposits consisting of gravel, sand and silt (USGS, 1999). Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

3.1 Artificial Fill

Artificial fill was encountered in our explorations to a maximum depth of ½ foot below the existing ground surface. Artificial fill was not encountered in boring B1. The artificial fill generally consists of brown to dark brown sandy clay and clay with sand. Within the existing soccer fields, the artificial fill consists of a silty sand substrate, likely to support the growth of grass. The artificial fill is characterized as slightly moist and firm. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

3.2 Young Alluvial Fan Deposits

Holocene age young alluvial fan deposits were encountered beneath the fill. The alluvial deposits generally consist of brown to dark brown, sandy clay and clay with varying amounts of sand and calcium carbonate stringers. The alluvial deposits are slightly moist to moist and firm to stiff.

4. GROUNDWATER

Based on a review of the Seismic Hazard Zone Report of the Tustin Quadrangle, (California Division of Mines and Geology [CDMG], 2001), the historically highest groundwater level in the area is greater than 40 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900’s to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was not encountered in our field explorations, drilled to a maximum depth of 6 feet below the existing ground surface. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed, especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the immediate site vicinity. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the S*urface Drainage* section of this report (see Section 6.14).

5. SEISMIC DESIGN CRITERIA

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

2019 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2019 CBC Reference
Site Class	D	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.243g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.445g	Figure 1613.2.1(2)
Site Coefficient, F_A	1.003	Table 1613.2.3(1)
Site Coefficient, F_V	1.855*	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.247g	Section 1613.2.3 (Eqn 16-36)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.826g*	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	0.831g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.551g*	Section 1613.2.4 (Eqn 16-39)

Note:

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

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.

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE_G Peak Ground Acceleration, PGA	0.521g	Figure 22-9
Site Coefficient, F_{PGA}	1.1	Table 11.8-1
Site Class Modified MCE_G Peak Ground Acceleration, PGA_M	0.573g	Section 11.8.3 (Eqn 11.8-1)

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 General

6.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude the construction of the proposed improvements provided the recommendations presented herein are followed and implemented during design and construction.

6.1.2 Up to ½ feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. The existing fill, in its present condition, is not suitable for direct support of proposed foundations and slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 6.4).

6.1.3 The on-site native soils are considered to have a "high" expansion potential and are classified as "expansive". These soils may be subject to swelling and shrinking cycles following the introduction of water due to precipitation, irrigation, or other means. Design and maintenance of proper drainage will be critical to the future performance of this project in order to reduce the potential for adverse impacts due to expansive soil, such as paving offsets or differential soil movement.

Foundations for miscellaneous small structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area, or in the competent alluvial soils found at and below a depth of 1 foot below the ground surface. Where excavation and proper compaction cannot be performed, foundations may derive support directly in the undisturbed alluvial soils found at and below a depth of 1 foot, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials or a minimum 30 inch embedment below the lowest adjacent grade (whichever is deeper). 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 in writing by a Geocon representative.

6.1.4 Cast-in-place friction piles may be utilized for support of the proposed light pole structures, provided foundations derive support in the competent alluvium found at or below a depth of 2 feet. Recommendations for friction pile design are provided in Section 6.8 of this report.

6.1.5 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon). Recommendations for earthwork are provided in the *Grading* section of this report (see Section 6.4).

6.1.6 It is anticipated that stable excavations for the proposed improvements can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 6.12).

6.1.7 Where new paving or hardscape is to be placed, it is recommended that all existing fill be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill in the area of new paving is not required; however, paving constructed over existing uncertified fill may experience increased settlement and/or cracking and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of subgrade soil should be excavated, moisture conditioned to 2 to 3 percent above optimum moisture content, and properly compacted for paving support. Paving recommendations are provided in the *Exterior Concrete Slabs-on-Grade* section of this report (see Section 6.10).

6.1.8 Based on the results of the percolation testing performed at the site, a stormwater infiltration system is considered feasible for this project. Recommendations for infiltration are provided in *Stormwater Infiltration* section of this report (see Section 6.13).

6.1.9 Once the design and foundation loading configuration for the proposed improvements proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be re-evaluated by this office.

6.1.10 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

6.2 Soil and Excavation Characteristics

6.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular soils are present.

6.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of existing adjacent improvements.

6.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 6.12).

6.2.4 The upper 3 feet of the soils encountered during the field investigation are considered to have a "high" expansion potential and are classified as "expansive" (expansion index [EI] of 109 and 119) as defined by 2019 California Building Code (CBC) Section 1803.5.3. Recommendations presented herein assume that miscellaneous foundations and hardscape will derive support in these materials.

6.3 Water-Soluble Sulfate

6.3.1 Laboratory tests were performed on representative samples of the site materials to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B11) and indicate that the on-site materials possess a sulfate exposure class of "S2" to concrete structures as defined by 2019 CBC Section 1904 and ACI 318-19 Chapter 19. The table below presents a summary of concrete requirements set forth by 2019 CBC Section 1904 and ACI 318. The presence of water-soluble sulfates is not a visually discernible characteristic; therefore, other soil samples from the site could yield different concentrations. Additionally, over time landscaping activities (i.e., addition of fertilizers and other soil nutrients) may affect the concentration.

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	$0.10\leq SO_4<0.20$	II		0.50	4,000
S2	**$0.20\leq SO_4\leq 2.00$**	**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

6.4 Grading

6.4.1 Grading is anticipated to include preparation of existing site soils for pavement and hardscape construction, and excavations for miscellaneous foundations.

6.4.2 A preconstruction conference should be held at the site prior to the beginning of grading operations with the owner, contractor, civil engineer, geotechnical engineer, and, if applicable, building official in attendance. Special soil handling requirements can be discussed at that time.

6.4.3 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill encountered during exploration is suitable for re-use as an engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris is removed. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein.

6.4.4 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein.

6.4.5 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

6.4.6 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to 2 to 3 percent above optimum moisture content, and compacted to at least 90 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

6.4.7 Based on the laboratory testing of the in-situ moisture content, the grading contractor should be aware that the existing soils are currently several points above optimum moisture content. Conditions could change seasonally. If the soils are in excess of 3 to 5 percent above optimum moisture content at the time of construction the soils will likely require some spreading and drying activities in order to achieve proper compaction.

6.4.8 It is anticipated that stable excavations for construction of proposed improvements can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of the existing offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 6.12).

6.4.9 Foundations for miscellaneous small structures, such as block walls less than 6 feet high, planter walls or trash enclosures may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed, foundations may derive support directly in the undisturbed alluvial soils found at and below a depth of 1 foot, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials or a minimum 30 inch embedment below the lowest adjacent grade (whichever is deeper). 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.

6.4.10 Where new paving or hardscape is to be placed, it is recommended that all existing fill and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill and soft alluvial soils in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvial soil may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of subgrade soil should be scarified and properly compacted for paving support. *Preliminary Pavement Recommendations* section of this report (see Section 6.11).

6.4.11 Imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Rocks larger than 6 inches in diameter shall not be used in the fill. Import soils should have an expansion index less than 50, and corrosivity properties that are equally or less detrimental to that of the existing onsite soils (see Figure B11).

6.4.12 Utility trenches should be properly backfilled in accordance with the following requirements. The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry is also acceptable as backfill. Prior to placing any bedding material or pipes, the trench excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

6.4.13 All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding materials, fill, steel, gravel or concrete.

6.5 Shrinkage

6.5.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of up to 10 percent should be anticipated when excavating and compacting the upper 3 feet of existing earth materials on the site to an average relative compaction of 92 percent.

7.4.2 If import soils will be utilized within the area of proposed improvements, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.). Soils can be borrowed from non-improvement areas and later replaced with imported soils.

6.6 Miscellaneous Foundations

6.6.1 Foundations for miscellaneous small structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area, or in the competent alluvial soils found at and below a depth of 1 foot, and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials or a minimum 30 inch embedment below the lowest adjacent grade (whichever is deeper).

6.6.2 If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. 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. Miscellaneous foundations may be designed for a bearing value of 1,500 psf and should be a minimum of 12 inches in width, 30 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.

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

6.7 Lateral Design

6.7.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.25 may be used with the dead load forces in the undisturbed alluvial soils or engineered fill.

6.7.2 Passive earth pressure for the sides of foundations and slabs poured against the alluvial soils or properly compacted engineered fill may be computed as an equivalent fluid having a density of 180 pounds per cubic foot (pcf) with a maximum earth pressure of 1,800 pcf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

6.8 Friction Pile Design

6.8.1 Cast-in-place friction piles may be utilized for support of the proposed light pole structures, provided foundations derive support in the competent alluvium at or below a depth of 2 feet.

6.8.2 Friction piles should be a minimum of 18 inches in diameter and should be embedded a minimum of 10 feet into the recommended bearing materials. Where not protected from erosion or disturbance, the upper 18 inches of soil should be ignored when calculating axial and lateral capacity.

6.8.3 Friction piles may be designed based on a skin friction capacity of 160 psf. Uplift capacity may be assumed to be ⅔ the axial capacity in compression. Increases in frictional resistance may be available at deeper pile depths and Geocon should be contacted to provide updated values once a preliminary design is available.

6.8.4 Friction piles do not require the complete removal of all loose earth materials from the bottom of the excavation since the end-bearing capacity is not being considered for design. However, a cleanout of the excavation bottom will be required. A one-third increase in the capacity may be used for wind or seismic loads.

6.8.5 For design purposes, an allowable passive value for the soils may be assumed to be 180 psf per foot. To develop the full lateral value, provisions should be implemented to assure firm contact between the piles and the engineered fill and underlying alluvium. A one-third increase in the passive value may be used for wind or seismic loads. The allowable capacity may be doubled for isolated piles spaced more than three times the diameter on-center.

6.8.6 The maximum expected settlement for improvements supported on piles deriving support in the alluvial soils is expected to be less than ½ inch. The majority of settlement is anticipated to occur on initial application of loading during construction. Differential settlement is not expected to exceed ½ inch between adjacent foundations.

6.8.7 All drilled pile excavations should be continuously observed by personnel of this firm to verify adequate penetration into the recommended bearing materials. The capacity presented is based on the strength of the soils. The compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

6.9 Deepened Foundation Installation

6.9.1 Casing may be required if caving is experienced in the drilled excavation. The contractor should have casing available prior to commencement of pile excavation. If casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.

6.9.2 Friction piles do not require the complete removal of all loose earth materials from the bottom of the excavation since the end-bearing capacity is not being considered for design. However, a cleanout of the excavation bottom will be required.

6.9.3 Groundwater was not encountered in our field explorations, drilled to a maximum depth of 6 feet below the existing ground surface, and the reported historic high groundwater is greater than 40 feet below the ground surface. However, should groundwater or seepage be encountered during construction, pile excavations with more than 6 inches of standing water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie shall consist of a water-tight tube, with a hopper at the top. The tube shall be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie shall be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end shall be closed at the start of the work to prevent water entering the tube and shall be entirely sealed at all times, except when the concrete is being placed. The tremie tube shall be kept full of concrete. The flow shall be continuous until the work is completed, and the resulting concrete seal shall be monolithic and homogeneous. The tip of the tremie tube shall always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to ensure that the tip of the tremie tube is never raised above the surface of the concrete.

6.9.4 A special concrete mix should be used for concrete to be placed below water. The design shall provide for concrete with a strength of 1,000 pounds per square inch (psi) over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste shall be included. The slump shall be commensurate to any research report for the admixture, provided that it shall also be the minimum for a reasonable consistency for placing when water is present. Extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Continuous observation of the drilling and pouring of the piles by a representative of this firm is required.

6.9.5 Closely spaced piles should be drilled and filled alternately, with the concrete permitted to set at least 8 hours before drilling an adjacent hole. Pile excavations should be filled with concrete as soon after drilling and inspection as possible; the holes should not be left open overnight unless approved by the Geotechnical Engineer.

6.10 Exterior Concrete Slabs-on-Grade

6.10.1 Concrete slabs for walkways or flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to 2 to 3 percent above optimum moisture content and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. The project structural engineer should design construction joints as necessary.

6.10.2 Due to the expansive potential of the subgrade soils, the moisture content of the slab subgrade should be maintained and sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement. Furthermore, consideration should be given to doweling slabs into adjacent curbs and foundations to minimize movements and offsets which could lead to a potential tripping hazard. As an alternative, the upper 24 inches of subgrade soils could be replaced with granular, non-expansive soils which will reduce the potential for movements and offsets. It may be feasible to reduce the slab thicknesses and/or reinforcing where slabs are underlain by non-expansive materials.

6.10.3 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete 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.

6.11 Preliminary Pavement Recommendations

6.11.1 Where new paving is to be placed, it is recommended that all existing fill and soft or unsuitable alluvial materials be excavated and properly recompacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing unsuitable material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to 2 to 3 percent above optimum moisture content, and properly compacted to at least 92 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

6.11.2 The following pavement sections are based on site-specific R-Value of 4. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement. The use of a pavement interlayer or geogrid can help improve subgrade performance and reduce the pavement section design thickness. However, consideration should be given to future excavations that may occur in the paved area. Recommendations for the design of flexible pavement using a interlayer or geogrid can be provided upon request.

6.11.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking And Driveways	4.0	4.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	16.0

6.11.4 Asphalt concrete should conform to Section 203-6 of the "*Standard Specifications for Public Works Construction"* (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the "*Standard Specifications of the State of California, Department of Transportation"* (Caltrans). The use of Crushed Miscellaneous Base in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the "*Standard Specifications for Public Works Construction"* (Green Book).

6.11.5 Unless specifically designed and evaluated by the project structural engineer, where concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 4 steel reinforcing bars placed 16 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 92 and 95 percent relative compaction, respectively, as determined by ASTM Test Method D 1557 (latest edition).

6.11.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

6.12 Temporary Excavations

6.12.1 Excavations less than 5 feet in height are anticipated during construction of proposed improvements. The excavations are expected to expose artificial fill and alluvial soils, which are suitable for vertical excavations up to 5 feet in height where loose soils or caving sands are not present, and where not surcharged by adjacent traffic or structures.

6.12.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to maximum height of 10 feet. A uniform slope does not have a vertical portion.

6.12.3 Where temporary slopes are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction slopes are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The soils exposed in the slopes should be inspected during excavation by our personnel so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

6.13 Stormwater Infiltration

6.13.1 During our site exploration performed on March 7, 2022, borings B1 and B3A were used to perform percolation testing. Boring B3A was located adjacent to boring B3 and was excavated to a depth of 3 feet for the purpose of percolation testing. Slotted casing was placed in the borings, and the annular space between the casing and excavation was filled with gravel. The borings were then filled with water to pre-saturate the soils. The casing was refilled with water and percolation test readings were performed after repeated flooding of the cased excavations. Based on the test results, the average infiltration rate (adjusted percolation rate), for the earth materials encountered, is provided in the following table. The field-measured percolation rate has been adjusted to infiltration rates in accordance with the *County of Orange Technical Guidance Document (TGD) for the Preparation of Conceptual/Preliminary and/or Project Water Quality Management Plans (December 2013)*. Additional correction factors may be required and should be applied by the engineer in responsible charge of the design of the stormwater infiltration system and based on applicable guidelines. Percolation test results are provided on Figures 3 and 4.

Boring	Soil Type	Infiltration Depth (ft)	Average Infiltration Rate (in / hour)
B1	Clayey Sand (SC)	3-5	2.41
B3A	Sandy Clay (CL)	1-3	0.21

6.13.2 The Orange County TGD indicates that a minimum infiltration rate of 0.3 inches per hour is required for infiltration to be considered feasible. Additionally, based on the predominately expansive clay soil conditions we encountered during our site exploration, shallow infiltration may saturate expansive soils. Based on these considerations, while infiltration at the location of B1 is considered feasible, it is recommended that infiltration occur at a minimum distance of 40 feet from existing or proposed foundations. Additionally, the project owner should understand that it is not our intent to completely prevent any soil movement as a result of stormwater infiltration as doing so would be prohibitive to the proposed project.

6.13.3 It is our further opinion that infiltration of stormwater at the locations tested will not induce excessive hydro-consolidation (see Figure B5), will not create a perched groundwater condition, will not affect soil structure interaction of existing or proposed foundations due to expansive soils, will not saturate soils supported by existing or proposed retaining walls, and will not increase the potential for liquefaction. Resulting settlements are anticipated to be less than ¼ inch, if any.

6.13.4 The infiltration system must be located such that the closest distance between an adjacent foundation is at least 40 feet in all directions from the zone of saturation. The zone of saturation may be assumed to project downward from the discharge of the infiltration facility at a gradient of 1:1. Additional property line or foundation setbacks may be required by the governing jurisdiction and should be incorporated into the stormwater infiltration system design as necessary.

6.13.5 Subsequent to the placement of the infiltration system, it is acceptable to backfill the resulting void space between the excavation sidewalls and the infiltration system with minimum two-sack slurry provided the slurry is not placed in the infiltration zone. It is recommended that pea gravel be utilized adjacent to the infiltration zone so communication of water to the soil is not hindered.

6.13.6 Due to the preliminary nature of the project at this time, the type of stormwater infiltration system and location of the stormwater infiltration systems has not yet been determined. The design drawings should be reviewed and approved by the Geotechnical Engineer. The installation of the stormwater infiltration system should be observed and approved by the Geotechnical Engineer (a representative of Geocon).

6.14 Surface Drainage

6.14.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the foundation supporting soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage in building areas should be maintained at all times.

6.14.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2019 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within five feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within five feet of the building perimeter footings except when enclosed in protected planters.

6.14.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures. The building pad and pavement areas should be fine graded such that water is not allowed to pond.

6.14.4 Landscaping planters immediately 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. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or an impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

6.15 Plan Review

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

LIMITATIONS AND UNIFORMITY OF CONDITIONS

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

2. This report is issued with the understanding that it is the responsibility of the owner, or of his 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.

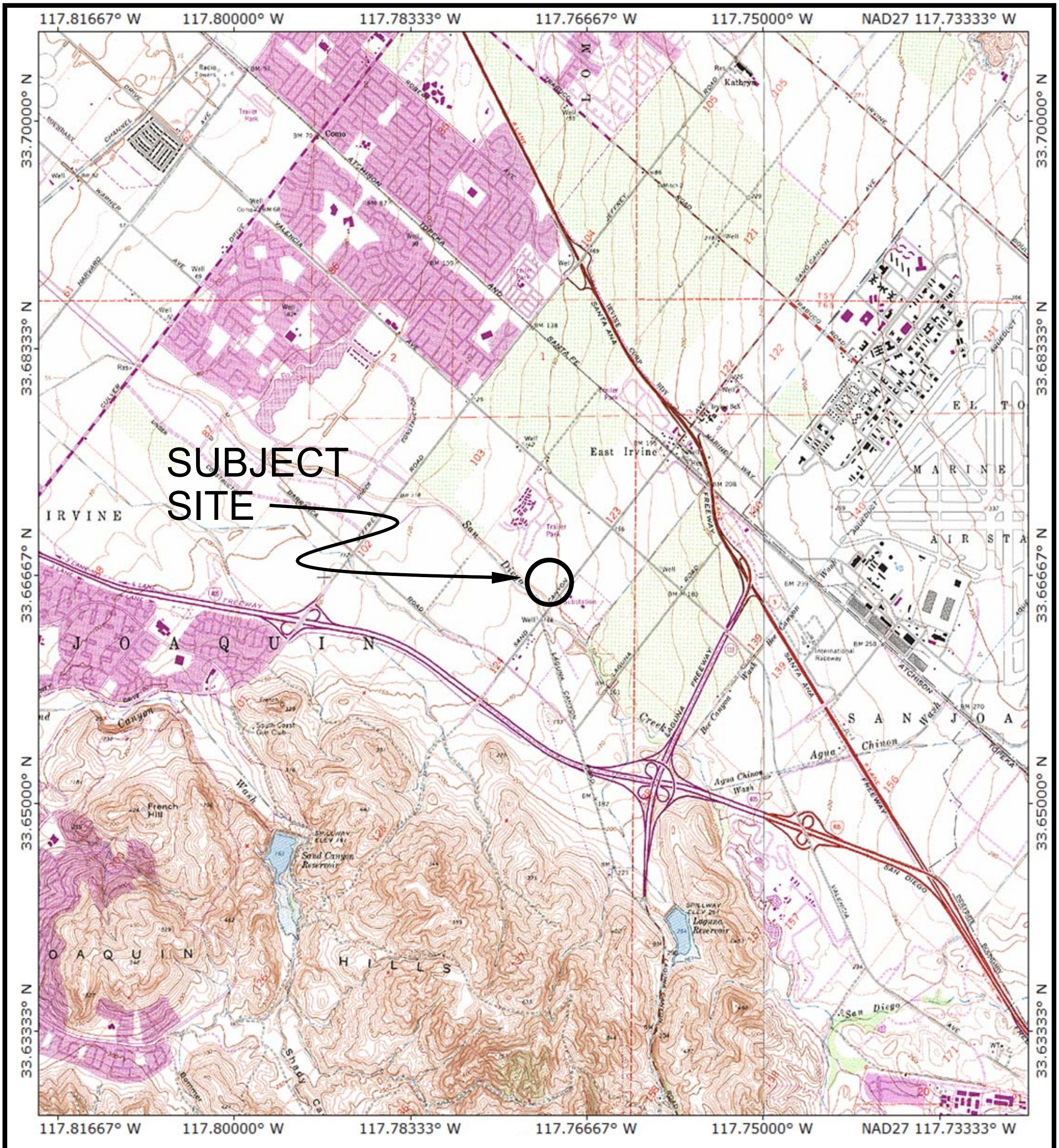
3. The findings of this report are valid as of the date of this report. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate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

4. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.

LIST OF REFERENCES

U.S. Geological Survey, 1999, *Preliminary Digital Geologic Map of the Santa Ana 30' x 60' Quadrangle, Southern California, Version 1.0,* Open File Report 99-172.

California Department of Conservation, Division of Mines and Geology, 2001, *Seismic Hazard Evaluation of the Tustin 7.5-Minute Quadrangle, Orange County, California,* Open File Report 97-20.



U.S.G.S. TOPOGRAPHIC MAPS, 7.5 MINUTE SERIES, TUSTIN, CA QUADRANGLE

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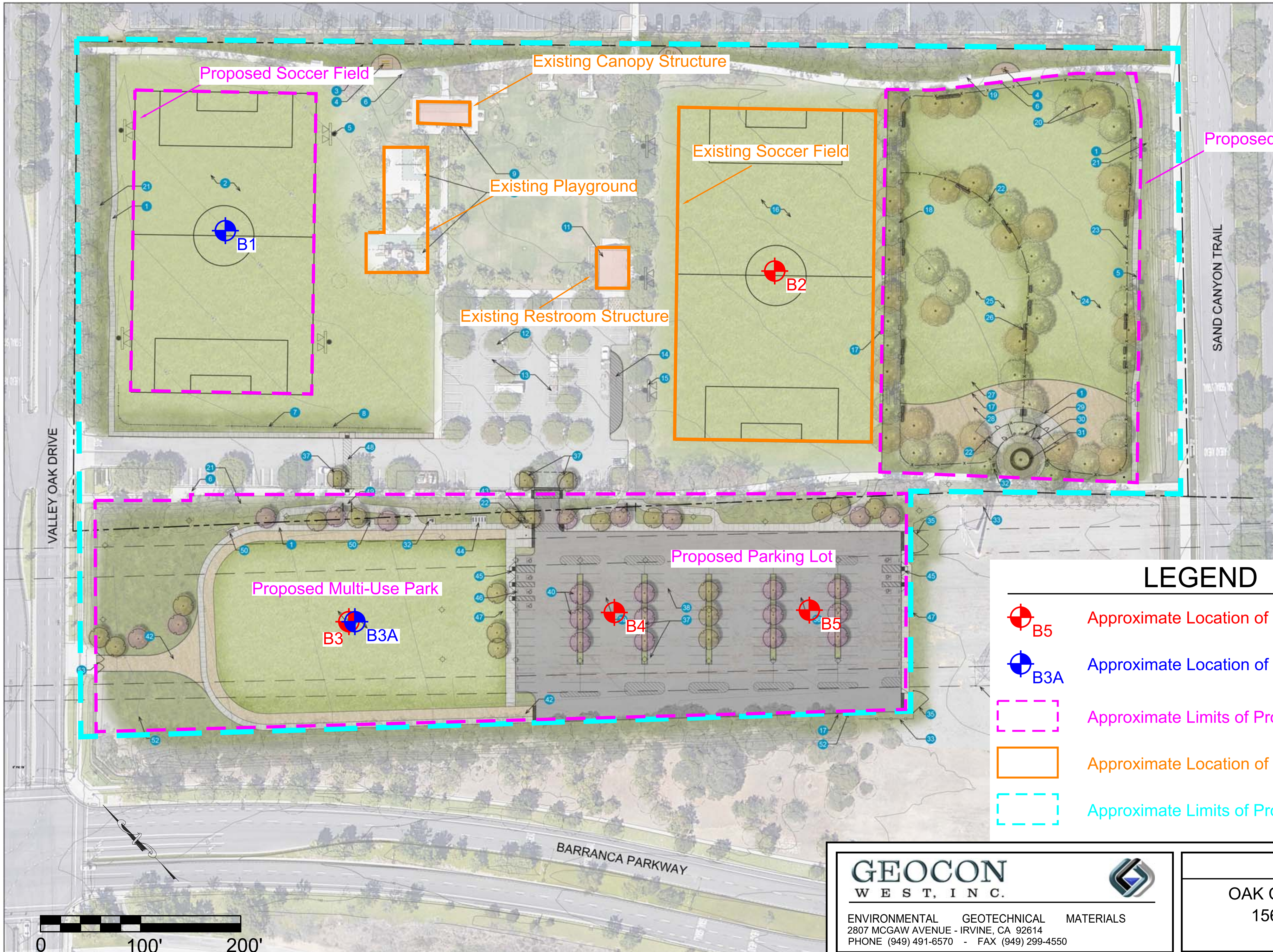
VICINITY MAP

OAK CREEK COMMUNITY PARK
15616 OAK VALLEY DRIVE
IRVINE, CALIFORNIA

APRIL 2022 | PROJECT NO. W1035-88-01 | FIG. 1



LEGEND

B5 Approximate Location of Boring

B3A Approximate Location of Percolation Test Boring

Approximate Limits of Proposed Improvements

Approximate Location of Existing Structures/Improvements

Approximate Limits of Proposed Project

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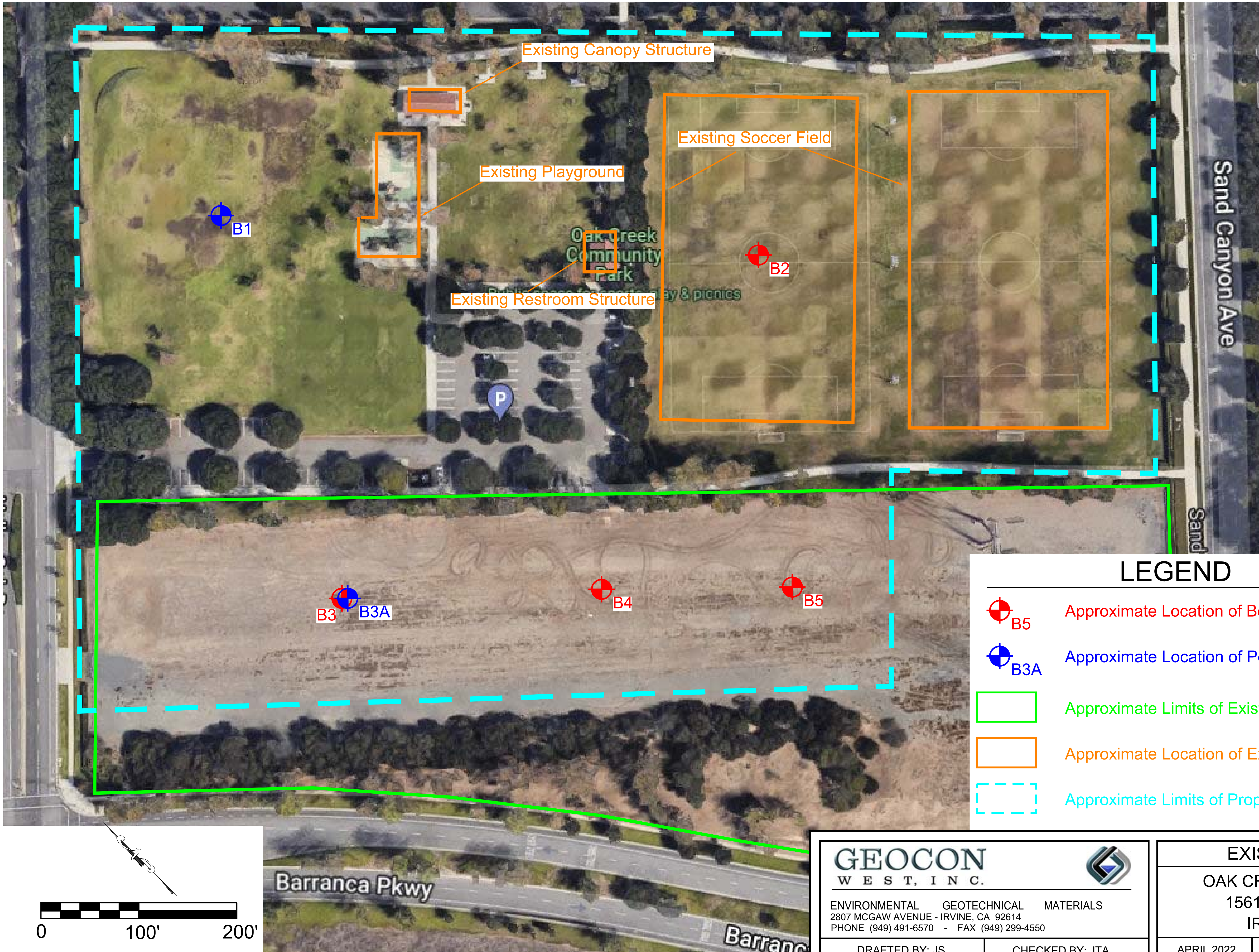
SITE PLAN

OAK CREEK COMMUNITY PARK
15616 OAK VALLEY DRIVE
IRVINE, CALIFORNIA

APRIL 2022 | PROJECT NO. W1035-88-01 | FIG. 2A



LEGEND

B5	Approximate Location of Boring
B3A	Approximate Location of Percolation Test Boring
	Approximate Limits of Existing SCE Easement
	Approximate Location of Existing Structures/Improvements
	Approximate Limits of Proposed Project

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EXISTING CONDITIONS

OAK CREEK COMMUNITY PARK
15616 OAK VALLEY DRIVE
IRVINE, CALIFORNIA

APRIL 2022 | PROJECT NO. W1035-88-01 | FIG. 2A

PERCOLATION TEST DATA SHEET

Project:	Oak Creek Park		Project No:	W1035-88-01		Date:	3/7/2022
Test Hole No:		B1	Tested By:	JS			
Depth of Test Hole, D_T:		5	USCS Soil Classification:		CL / SC		
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round) =		4	Sides (if rectangular) =		---	---	

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Greater than or Equal to 6"? (y/n)
1	11:59	12:24	25	37.8	53.6	15.8	y
2	12:32	12:57	25	39.0	52.7	13.7	y

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements, taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min/in)
1	12:59	13:09	10	39.0	46.9	7.9	1.26
2	13:11	13:21	10	36.6	46.0	9.4	1.07
3	13:23	13:33	10	36.6	45.7	9.1	1.10
4	13:35	13:45	10	36.6	45.4	8.8	1.14
5	13:46	13:56	10	36.7	45.4	8.6	1.16
6	13:59	14:09	10	36.6	44.8	8.2	1.23
7							
8							

Infiltration Rate Calculation:

Time Interval, Δt =	10	minutes	Ho =	23.4	inches
Final Depth to Water, Df =	44.8	inches	Hf =	15.2	inches
Test Hole Radius, r =	2	inches	ΔH =	8.2	inches
Initial Depth to Water, Do =	36.6	inches	Havg =	19.3	inches
Total Depth of Test Hole, DT =	60.0	inches			

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

Infiltration Rate, It = **2.41** inches/hour

Figure 3

PERCOLATION TEST DATA SHEET

Project:	Oak Creek Park		Project No:	W1035-88-01		Date:	3/7/2022
Test Hole No:		B3A	Tested By:	JS			
Depth of Test Hole, D_T:		3	USCS Soil Classification:		CL		
Test Hole Dimensions (inches)					Length	Width	
Diameter (if round) =		4	Sides (if rectangular) =		---	---	

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Greater than or Equal to 6"? (y/n)
1	9:42	10:12	30	15.8	22.2	6.4	y
2	10:14	10:44	30	16.4	20.5	4.1	n

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements, taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Δt Time Interval (min)	D_0 Initial Depth to Water (in)	D_f Final Depth to Water (in)	ΔD Change in Water Level (in)	Percolation Rate (min/in)
1	10:46	11:16	30	14.9	18.6	3.7	8.06
2	11:16	11:56	40	11.4	17.3	5.9	6.80
3	11:56	12:34	38	17.3	20.0	2.8	13.77
4	12:34	13:04	30	20.0	21.6	1.6	19.23
5	13:05	13:37	32	10.7	15.5	4.8	6.67
6	13:37	14:11	34	15.5	18.2	2.8	12.32
7	14:11	14:41	30	18.2	20.0	1.8	16.67
8	14:44	15:14	30	13.0	15.7	2.8	10.87
9	15:14	15:44	30	15.7	17.9	2.2	13.89

Infiltration Rate Calculation:

Time Interval, Δt =	30	minutes	Ho =	20.3	inches
Final Depth to Water, Df =	17.9	inches	Hf =	18.1	inches
Test Hole Radius, r =	2	inches	ΔH =	2.2	inches
Initial Depth to Water, Do =	15.7	inches	Havg =	19.2	inches
Total Depth of Test Hole, DT =	36.0	inches			

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

Infiltration Rate, It = **0.21** inches/hour

Figure 4

APPENDIX A

APPENDIX A

FIELD INVESTIGATION

The site was explored on March 7, 2022, by excavating five 3¼-inch diameter borings to depths ranging from approximately 5 to 6 feet below the existing ground surface using manual hand auger equipment and digging tools. An additional boring (B3A) was excavated near boring location B3 for percolation testing. Representative and relatively undisturbed samples were obtained by driving a 3-inch O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a slide hammer. The California Modified Sampler was equipped with 1-inch high by 2⅜-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A6. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the boring logs were revised based on subsequent laboratory testing. The locations of the borings are shown on the Site Plan (see Figures 2A and 2B).

BORING 1

ELEV. (MSL.) -- DATE COMPLETED 3/7/2022

EQUIPMENT HAND AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-3'				**ALLUVIUM**			
				CL	Clay with sand, firm, moist, brown, fine-grained, trace white stringers.			
2								
	B1@2.5'				-increase in sand, increase in white stringers.		107.0	19.6
4				SC	Clayey Sand, medium dense, moist, strong brown, fine- to medium-grained.			
	B1@4.5'				-increase in clay		111.8	14.7
					Total depth of boring: 5 feet No Fill. No groundwater encountered. Percolation testing performed. NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.			

Figure A1,
Log of Boring 1, Page 1 of 1

W1035-88-01 BORING LOGS.GPJ

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

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

GEOCON

BORING 2

ELEV. (MSL.) -- DATE COMPLETED 3/7/2022

EQUIPMENT HAND AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ARTIFICIAL FILL** Silty Sand Substrate			
				CL	**ALLUVIUM** Clay, firm, moist, brown, some fine-grained sand.			
2	B2@2.5'			SC	Clayey Sand, medium dense, moist, brown, fine- to medium-grained.		105.7	20.8
4	B2@5'			CL	Clay and Sand, firm, moist, dark yellowish brown, fine- to medium-grained sand, trace to some white stringers.		97.5	23.6
6								

Total depth of boring: 6 feet
Fill to 0.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

Figure A2,
Log of Boring 2, Page 1 of 1

W1035-88-01 BORING LOGS.GPJ

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

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

GEOCON

PROJECT NO. W1035-88-01

BORING 3

ELEV. (MSL.) -- DATE COMPLETED 3/7/2022

EQUIPMENT HAND AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-5'				**ARTIFICIAL FILL** Sandy Clay, firm, slightly moist, brown, fine-grained.			
2					**ALLUVIUM** Sandy Clay, firm to stiff, slightly moist, brown to dark brown, fine-grained, trace white stringers.			
	B3@2.5'			CL			101.6	15.3
4								
	B3@5'				-stiff to hard, dark yellowish brown, increase in white stringers.		107.3	13.3

Total depth of boring: 5'9" feet
Fill to 0.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

Figure A3,
Log of Boring 3, Page 1 of 1

W1035-88-01 BORING LOGS.GPJ

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

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

GEOCON

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 3A ELEV. (MSL.) -- DATE COMPLETED 3/7/2022 EQUIPMENT HAND AUGER BY: JS	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0					**ARTIFICIAL FILL** Sandy Clay, firm, slightly moist, dark brown, fine-grained.			
2					**ALLUVIUM** Sandy Clay, firm to stiff, moist, brown to dark brown, fine-grained.			
					Total depth of boring: 3 feet Fill to 0.5 feet. No groundwater encountered. Percolation testing performed. Backfilled with soil cuttings and tamped. NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.			

Figure A4,
Log of Boring 3A, Page 1 of 1

W1035-88-01 BORING LOGS.GPJ

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

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

GEOCON

PROJECT NO. W1035-88-01

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 4 ELEV. (MSL.) -- DATE COMPLETED 3/7/2022 EQUIPMENT HAND AUGER BY: JS	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	BULK 0-3'				**ARTIFICIAL FILL** Clay with Sand, firm, slightly moist, brown, fine-grained.			
2	B4@2.5'			CL	**ALLUVIUM** Clay with Sand, firm, moist, brown to dark brown, fine-grained. -increase in sand, some white stringers.		100.3	23.0
4					-clayey sand interbed.			
	B4@5'			CL	Sandy Clay, stiff, slightly moist, brown, fine-grained, some white stringers.		109.6	11.6
6					Total depth of boring: 6 feet Fill to 0.5 feet. No groundwater encountered. Backfilled with soil cuttings and tamped. NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.			

Figure A5,
Log of Boring 4, Page 1 of 1

W1035-88-01 BORING LOGS.GPJ

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

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

GEOCON

BORING 5

ELEV. (MSL.) -- DATE COMPLETED 3/7/2022

EQUIPMENT HAND AUGER BY: JS

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	MATERIAL DESCRIPTION	PENETRATION RESISTANCE (BLOWS/FT)*	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0	BULK 0-3'				**ARTIFICIAL FILL** Clay with Sand, firm, slightly moist, brown, fine-grained.			
2				CL	**ALLUVIUM** Clay with Sand, firm to stiff, moist, brown to dark brown, fine-grained.			
	B5@2.5'				-increase in sand, trace white stringers.		105.6	20.6
4								
	B5@5'			CL	Clay and Sand, stiff, slightly moist, yellowish brown, fine-grained, some white stringers.		105.5	12.6

Total depth of boring: 5'11" feet
Fill to 0.5 feet.
No groundwater encountered.
Backfilled with soil cuttings and tamped.

NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.

Figure A6,
Log of Boring 5, Page 1 of 1

W1035-88-01 BORING LOGS.GPJ

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

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

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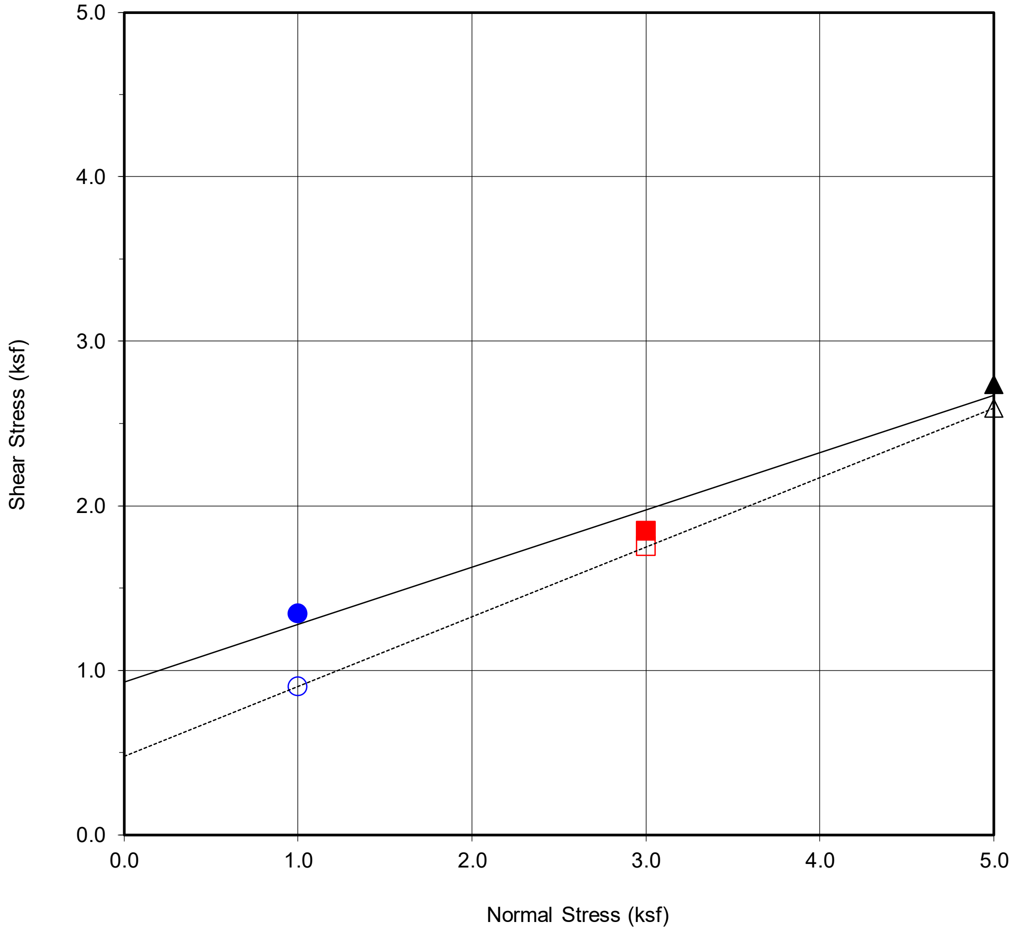
APPENDIX B

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the International ASTM, or other suggested procedures. Selected samples were tested for direct shear strength, consolidation characteristics, expansion index, water-soluble sulfate, R-Value, in-place dry density and moisture content. The results of the laboratory tests are summarized in Figures B1 through B1. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



Boring No.	**B-1**
Sample No.	**B1@2.5'**
Depth (ft)	**2.5'**
Sample Type:	Ring

Soil Identification:		
Clay with Sand (CL)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	932	19.2
Ultimate	479	22.9

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.34	■ 1.85	▲ 2.74
Shear Stress @ End of Test (ksf)	○ 0.90	□ 1.75	△ 2.59
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	20.3	20.4	20.9
Initial Dry Density (pcf)	107.1	107.4	104.6
Initial Degree of Saturation (%)	95.3	96.8	92.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	23.2	22.4	22.9

GEOCON

DIRECT SHEAR TEST RESULTS

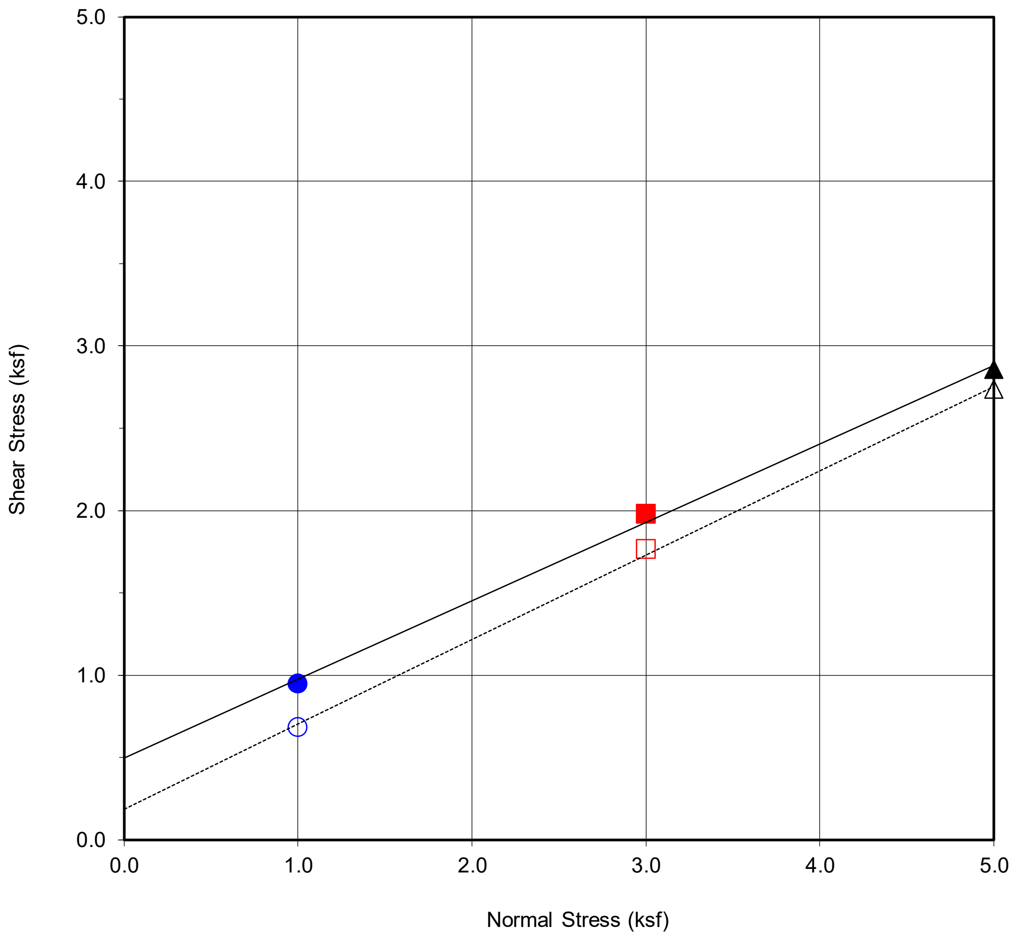
Consolidated Drained ASTM D-3080

Checked by: JS

Project No.: W1035-88-01

Oak Creek Community Park
15616 Oak Valley Drive
Irvine, California

April 2022 Figure B1



Boring No.	**B-1**
Sample No.	**B1@4'**
Depth (ft)	**4'**
Sample Type:	Ring

Soil Identification:		
Sandy Clay (CL)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	497	25.5
Ultimate	189	27.2

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.95	■ 1.98	▲ 2.86
Shear Stress @ End of Test (ksf)	○ 0.68	□ 1.76	△ 2.74
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	19.9	21.6	17.5
Initial Dry Density (pcf)	107.0	105.2	108.9
Initial Degree of Saturation (%)	93.3	96.8	86.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	25.3	24.2	20.6

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DIRECT SHEAR TEST RESULTS

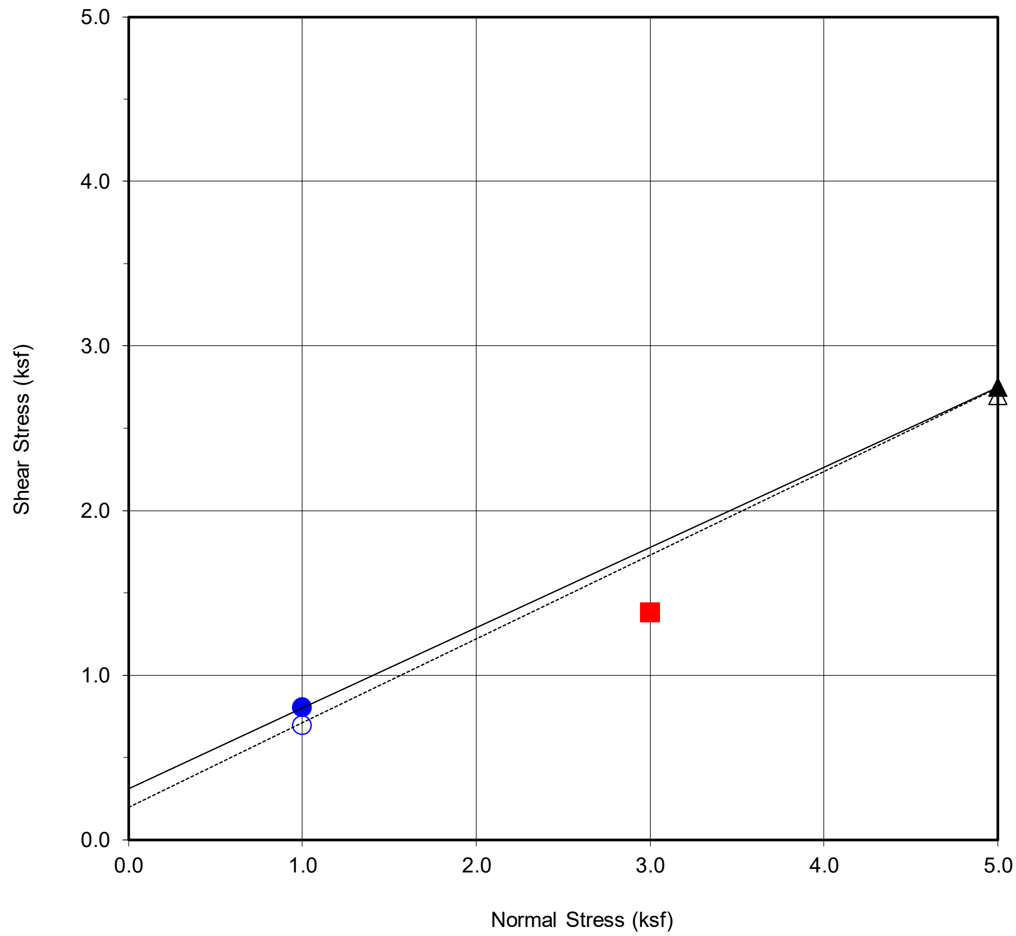
Consolidated Drained ASTM D-3080

Checked by: JS

Project No.: W1035-88-01

Oak Creek Community Park
15616 Oak Valley Drive
Irvine, California

April 2022 Figure B2



Boring No.	B-4
Sample No.	B4@5'
Depth (ft)	5'
Sample Type:	Ring

Soil Identification:		
Sandy Clay (CL)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	313	26.0
Ultimate	200	27.0

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.80	■ 1.38	▲ 2.75
Shear Stress @ End of Test (ksf)	○ 0.70	□ 1.38	△ 2.70
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	19.0	21.8	21.1
Initial Dry Density (pcf)	101.3	94.0	101.1
Initial Degree of Saturation (%)	77.3	74.2	85.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	27.8	27.1	22.6

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

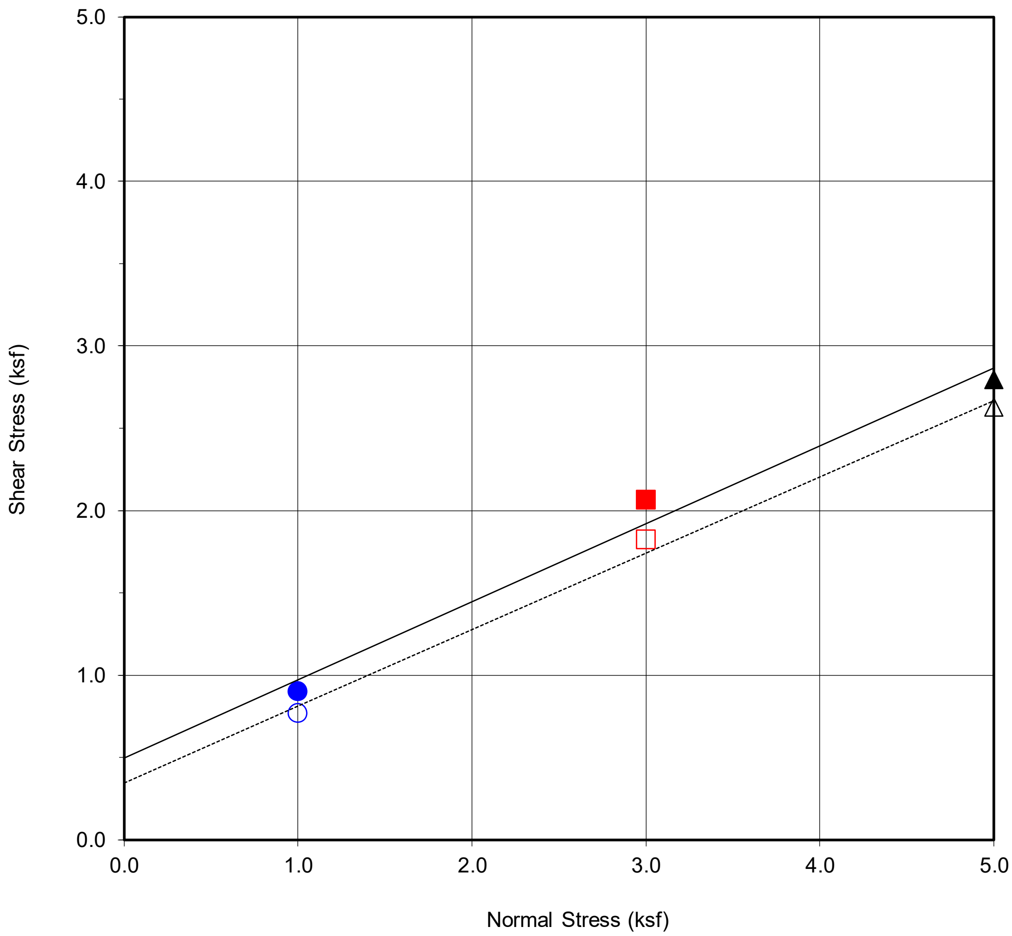
Checked by: JS

Project No.: W1035-88-01

Oak Creek Community Park
15616 Oak Valley Drive
Irvine, California

April 2022

Figure B3



Boring No.	B-5
Sample No.	B5@2.5'
Depth (ft)	2.5'
Sample Type:	Ring

Soil Identification:		
Clay and Sand (CL)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	498	25.4
Ultimate	345	24.9

Normal Stress (kip/ft2)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.90	■ 2.06	▲ 2.80
Shear Stress @ End of Test (ksf)	○ 0.77	□ 1.82	△ 2.63
Deformation Rate (in./min.)	0.01	0.01	0.01
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	21.2	22.1	20.8
Initial Dry Density (pcf)	105.1	103.3	104.5
Initial Degree of Saturation (%)	94.9	94.3	91.7
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	24.3	24.4	22.6

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DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: JS

Project No.: W1035-88-01

Oak Creek Community Park
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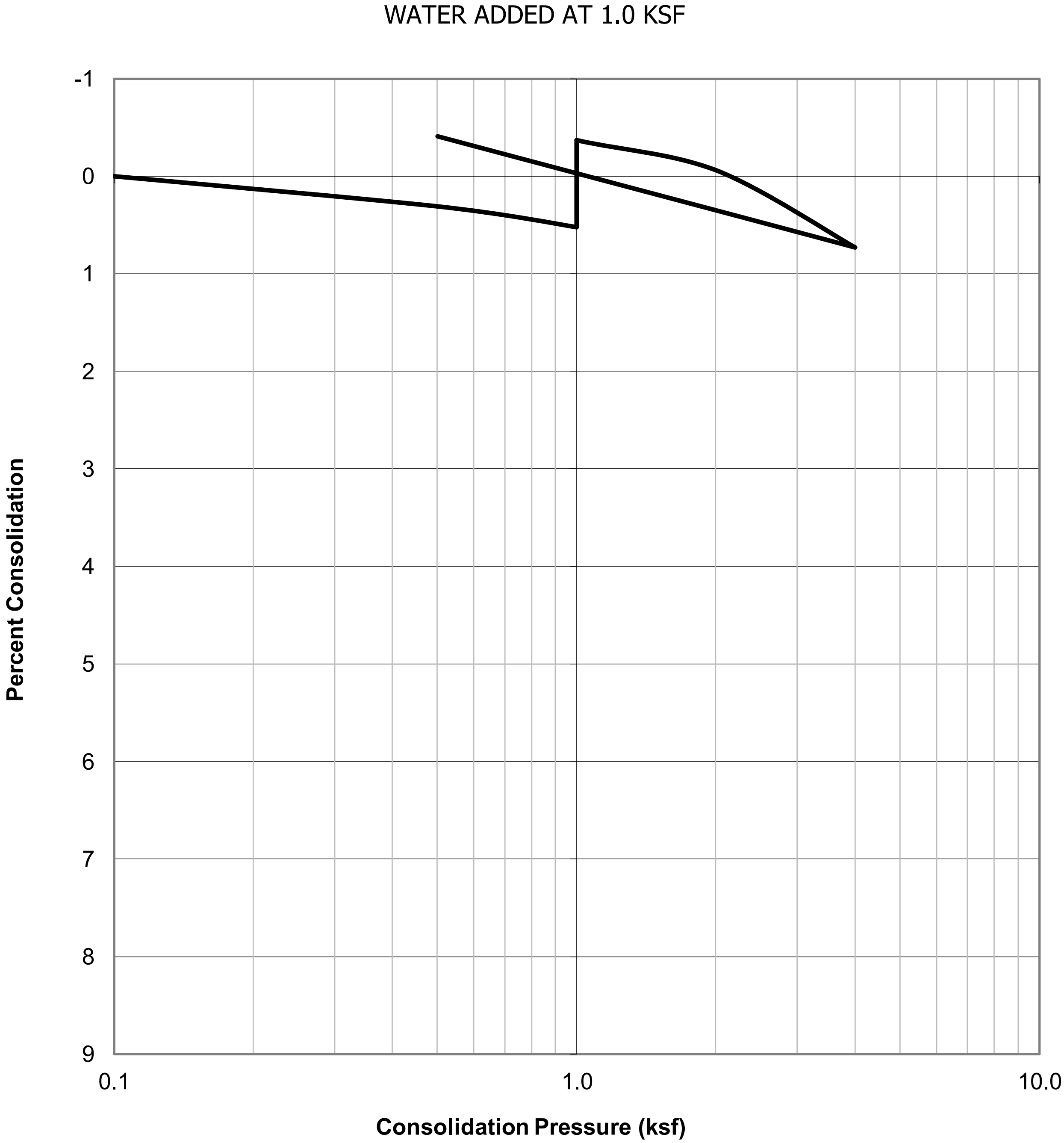
April 2022

Figure B4



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B1@4.5'	Clay and Clayey Sand (CL)	109.2	18.9	21.7

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CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: JS

Project No.: W1035-88-01

Oak Creek Community Park
15616 Oak Valley Drive
Irvine, California

April 2022

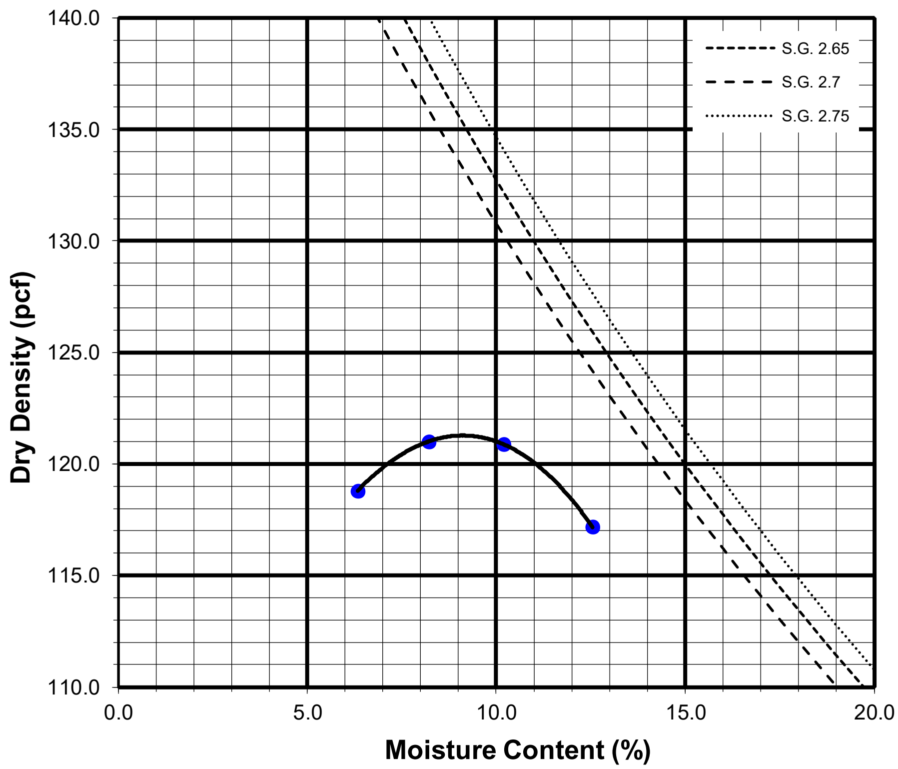
Figure B5

Sample No:

B1@0-3'	Clay with Sand (CL), brown

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6274	6294	6260	6190		
Weight of Mold	(g)	4282	4282	4282	4282		
Net Weight of Soil	(g)	1992	2012	1978	1908		
Wet Weight of Soil + Cont.	(g)	2401.2	2389.5	2389.1	2371.3		
Dry Weight of Soil + Cont.	(g)	2179.2	2203.5	2239.0	2254.5		
Weight of Container	(g)	409.8	378.7	411.8	409.8		
Moisture Content	(%)	12.5	10.2	8.2	6.3		
Wet Density	(pcf)	131.9	133.2	131.0	126.3		
Dry Density	(pcf)	117.2	120.9	121.0	118.8		

Maximum Dry Density (pcf)	121.8	Optimum Moisture Content (%)	9.2



Preparation Method: A

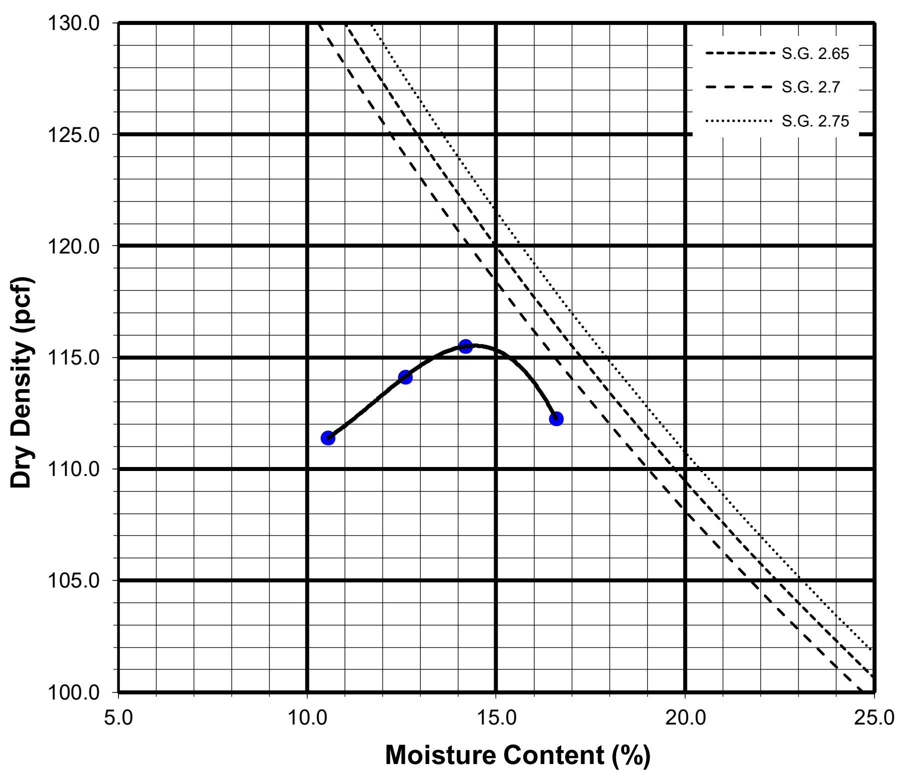
GEOCON	**COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS** ASTM D-1557	Project No.: W1035-88-01 Oak Creek Community Park 15616 Oak Valley Drive Irvine, California
	Checked by: JS	April 2022 Figure B6

Sample No:

B4+B5@0-3'	Clay with Sand (CL), dark brown

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6160	6241	6292	6277		
Weight of Mold	(g)	4300	4300	4300	4300		
Net Weight of Soil	(g)	1860	1941	1992	1977		
Wet Weight of Soil + Cont.	(g)	500.0	500.0	500.0	500.0		
Dry Weight of Soil + Cont.	(g)	452.3	444.1	437.9	428.9		
Weight of Container	(g)	0.0	0.0	0.0	0.0		
Moisture Content	(%)	10.5	12.6	14.2	16.6		
Wet Density	(pcf)	123.1	128.5	131.9	130.9		
Dry Density	(pcf)	111.4	114.1	115.5	112.3		

Maximum Dry Density (pcf) **116.5** **Optimum Moisture Content (%)** **14.2**



Preparation Method: A

GEOCON	**COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS** ASTM D-1557	Project No.: W1035-88-01
		Oak Creek Community Park 15616 Oak Valley Drive Irvine, California
	Checked by: JS	April 2022 Figure B7

B1@0-3'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.1
Wt. Comp. Soil + Mold	(gm)	557.6	610.5
Wt. of Mold	(gm)	176.5	176.5
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	707.8	610.5
Dry Wt. of Soil + Cont.	(gm)	677.1	342.1
Wt. of Container	(gm)	407.8	176.5
Moisture Content	(%)	11.4	26.9
Wet Density	(pcf)	115.0	130.7
Dry Density	(pcf)	103.2	103.1
Void Ratio		0.6	0.8
Total Porosity		0.4	0.5
Pore Volume	(cc)	80.3	104.9
Degree of Saturation	(%) [S_{meas}]	49.0	87.6

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
3/29/2022	10:00	1.0	0	0.3632
3/29/2022	10:10	1.0	10	0.3626
Add Distilled Water to the Specimen				
3/30/2022	10:00	1.0	1430	0.4814
3/30/2022	11:00	1.0	1490	0.4814

Expansion Index (EI meas) =	118.8
Expansion Index (Report) =	**119**

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

* Reference: 2019 California Building Code, Section 1803.5.3

** Reference: 1997 Uniform Building Code, Table 18-I-B.

GEOCON

EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: JS

Project No.: W1035-88-01

Oak Creek Community Park
15616 Oak Valley Drive
Irvine, California

April 2022

Figure B8

B4+B5@0-3'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.1
Wt. Comp. Soil + Mold	(gm)	574.9	631.3
Wt. of Mold	(gm)	199.4	199.4
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	499.4	631.3
Dry Wt. of Soil + Cont.	(gm)	465.1	332.6
Wt. of Container	(gm)	199.4	199.4
Moisture Content	(%)	12.9	29.9
Wet Density	(pcf)	113.3	130.1
Dry Density	(pcf)	100.3	100.2
Void Ratio		0.7	0.9
Total Porosity		0.4	0.5
Pore Volume	(cc)	83.8	106.3
Degree of Saturation	(%) [S_{meas}]	51.6	93.4

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
3/29/2022	10:00	1.0	0	0.2625
3/29/2022	10:10	1.0	10	0.2625
Add Distilled Water to the Specimen				
3/30/2022	10:00	1.0	1430	0.3712
3/30/2022	11:00	1.0	1490	0.3712

Expansion Index (EI meas) =	108.7
Expansion Index (Report) =	**109**

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

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

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EXPANSION INDEX TEST RESULTS

ASTM D-4829

Checked by: JS

Project No.: W1035-88-01

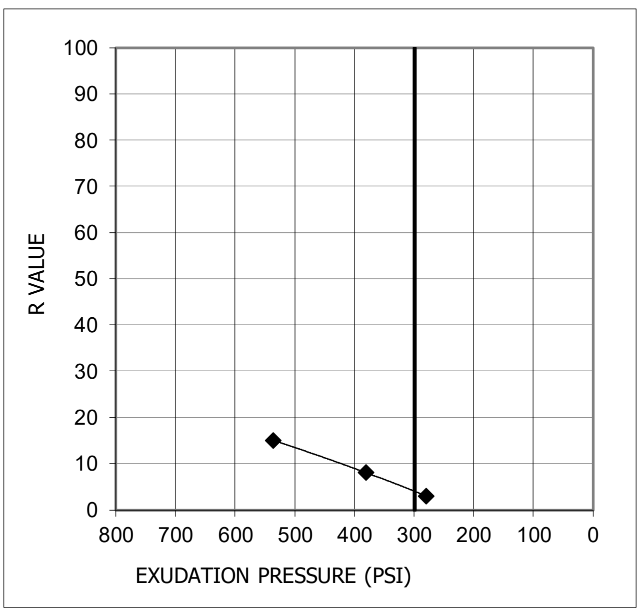
Oak Creek Community Park
15616 Oak Valley Drive
Irvine, California

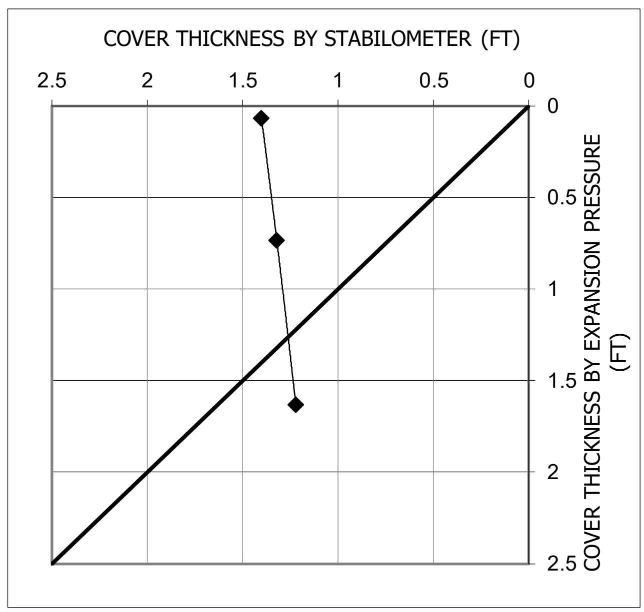
April 2022 Figure B9

Mold ID		A	B	C
Exudation Pressure	(psi)	536	380	280
Expansion Dial	(.0001")	49	22	2
Expansion Pressure	(psf)	212.2	95.3	8.7
Resistance 'R' Value	(psi)	15	8	3
Moisture Content	(%)	21.1	23.2	26.4
Dry Density	(pcf)	106	103.4	98.0

Sample ID:
B4+B5@0-3'
Sample Description:
Clay with Sand (CL), dark brown

R-Value by Expansion:	12
R-Value by Exudation:	4
R-Value by Equilibrium:	**4**





GEOCON	**R-VALUE TEST RESULTS** ASTM D-2844 Checked by: JS	Project No.: W1035-88-01 Oak Creek Community Park 15616 Oak Valley Drive Irvine, California April 2022 Figure B10

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
CALIFORNIA TEST NO. 417

Sample No.	**Water Soluble Sulfate (% SQ_4)**	**Sulfate Exposure***
B1@0-3'	0.247	S2

GEOCON

CORROSIVITY TEST RESULTS

Checked by: JS

Project No.: W1035-88-01

Oak Creek Community Park
15616 Oak Valley Drive
Irvine, California

April 2022

Figure B11