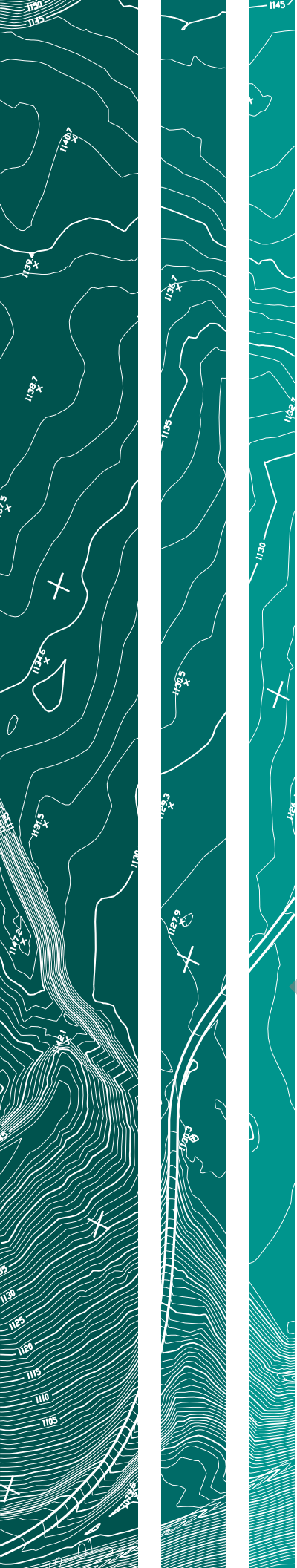
APPENDIX E
LIMITED GEOTECHNICAL INVESTIGATION



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GEOCON

LIMITED GEOTECHNICAL INVESTIGATION

GLENDORA FAMILY BIKE PARK PROJECT
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

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JANUARY 2025
PROJECT NO. W1983-06-01

PREPARED FOR:
IMEG Consultants Corp.
Ontario, California

Project No. W1983-06-01
January 2025

Mr. John Thompson
IMEG Consultants Corp.
901 Via Piemonte, Suite 400
Ontario, CA 91764

Subject: LIMITED GEOTECHNICAL INVESTIGATION
GLENDORA FAMILY BIKE PARK PROJECT
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

Mr. Thompson:

In accordance with your authorization of our proposal dated April 23, 2024, we have performed a limited geotechnical investigation in support of the Glendora Family Bike Project located at South Hills Park in the City of Glendora, 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 site can be developed 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.

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Disclaimer: This DRAFT is intended for the use of the project team to help with the on-going design of the project and provided as a courtesy for review only. This DRAFT should not be relied upon for final design nor produced and submitted to regulatory agencies until a FINAL document is completed with the signature and stamps of the design professionals.

Rex Panoy	Joe Hicks, M.S.	Michael A. Cazeneuve, GE, CEG
Senior Staff Engineer	PE 93183	GE 3114, CEG 2546

(email) Addressee

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LIMITATIONS AND UNIFORMITY OF CONDITIONS

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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation in support of the Glendora Family Bike Park Project located at South Hills Park, in the City of Glendora, California (see Vicinity Map, Figure1). 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 proposed 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 October 9, 2024, by excavating seven 7-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine between the depths of approximately 5½ and 37 feet below the existing ground surface. Additionally, the site was explored on November 22, 2024, by excavating one 30-inch-diameter boring using a bucket auger drilling machine to a maximum depth of approximately 36 feet. Refusal was encountered at depths of approximately 36 and 37 feet during the site explorations. The approximate locations of the exploratory borings are depicted on the Site Plan and Geologic Map (see Figure 2). 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.

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2. SITE AND PROJECT DESCRIPTION

The subject area of proposed improvements is located within South Hills Park, in the City of Glendora, California. The project is bounded by Glendora Avenue to the west, by the Heritage Oaks Senior Apartment development to the north, by undeveloped sections of South Hills Park to the east, and by the Glendora Avenue Access Road that traverses from the southwest to the southeast. The site slopes to the north and west with approximately 180 feet of vertical relief across the site. Surface water drainage occurs by sheet flow along the local topography towards city streets.

South Hills Park is a 318-acre public park located east of Glendora Avenue that currently serves hikers, bikers, and equestrian riders. It is our understanding that the main core of the Glendora Family Bike Park will be located along the west side of the park, adjacent to Glendora Avenue. Proposed bike park amenities include ADA accessible walkways, parking areas, vault style restrooms, gabion walls, a shade pavilion, seating and viewing areas, a bicycle pump track, and climbing and biking trails. The limits of the proposed improvements are indicated on the Site Plan and Geologic Map (see Figure 2).

Based on the preliminary nature of the design at this time, structural loads for project are not currently available. However, available manufacturer specifications for typical vault restrooms indicate the structures typically weigh about 40 kips and are embedded approximately 5 feet below the ground surface. Earthwork for the project is expected to consist of site preparations, miscellaneous cutting and filling (on the order of 5 feet or less) to achieve the desired site elevations and bike path contours, and removal and compaction of existing unsuitable site soils. As discussed herein, earthwork may also include trimming some of the existing slopes for stability purposes.

Once the design phase and 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.

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3. SOIL AND GEOLOGIC CONDITIONS

Based on the site exploration and published geologic maps of the area, much of the site is underlain by artificial fill and bedrock of the Topanga Formation. Limited natural residual soils were also observed above the bedrock in some areas. Although not encountered in our borings, Glendora Volcanic Rocks make up the hillside offsite to the east and south (Dibblee and Minch, 2002). Quaternary-age landslide deposits are mapped on the northern portion of the subject site and offsite to the east by S.S. Tan (Tan, 2000). However, landslide deposits were not encountered in our explorations or observed on the subject site during Geocon's investigation. Additional discussion regarding landsliding is provided in Section 6 of this report. 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 7 feet below existing ground surface. The material generally consists of brown silty sand that can be characterized as dry to slightly moist and medium dense. The fill is likely the result of past grading or construction activities at the site. Deeper fill likely exists between excavations and in other portions of the site that were not directly explored.

3.2 Natural Residual Soil

Occasional residual soil, or topsoil, was encountered below the fill and above the Topanga Formation where the underlying bedrock material has completely weathered until only soil texture remains. Residual soil thicknesses between approximately 1.5 and 3 feet were observed in borings B1, B3, and B5. It is noted that the residual soils were encountered to the maximum depth of exploration in borings B1 and B3.

3.3 Topanga Formation

Tertiary age Topanga Formation was encountered beneath the fill. The bedrock formation is generally a yellowish brown to reddish brown sandstone that is fine to coarse grained, thickly bedded to massive, medium hard to hard, moderately to completely weathered, and slightly fractured. The formation occasionally includes thin interbeds of siltstone and claystone and has various amounts of oxidation.

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3.4 Glendora Volcanic Rocks

Although the Tertiary-age Glendora Volcanic Rocks formation was not encountered during our exploration, it was observed to make up the hillside immediately east and south of the site. The volcanics are generally extrusive rocks and mapped as basaltic palagonitic tuff and pillow lavas (Dibblee and Minch, 2002). Onsite, they were observed to be fine grained, massive, hard, slightly weathered, and moderately fractured.

4. GROUNDWATER

Review of the Seismic Hazard Zone Report for the San Dimas 7.5 Minute Quadrangle, Los Angeles County, California (CDMG, 1998, Revised 2006) indicates that the historically highest groundwater level in the area is between 100 and 150 feet beneath the ground surface. Groundwater level information in the CDMG publication is based on data collected from the early 1900's to the late 1990's. Based on current groundwater basin management practices, it is unlikely that the groundwater levels will ever exceed the historic high levels.

Groundwater was not encountered in any of our exploratory borings to the maximum depth explored of 37 feet below the existing ground surface. In addition, the underlying bedrock is not considered water bearing. Based on the lack of groundwater in our borings and the depth of proposed construction, static groundwater will likely not be encountered during construction and is not expected to have a detrimental effect on the project. 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), groundwater seepage levels encountered during construction may be actually higher than those encountered during our investigation. In addition, recent requirements for storm water infiltration could result in shallower seepage conditions in the region. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the *Surface Drainage* section of this report (see Section 8.16).

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5. SEISMIC DESIGN CRITERIA

The following table summarizes the 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. The data was calculated using the online application U.S. Seismic Design Maps, provided by the Structural Engineers Association of California (SEAOC). The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2022 CBC and Table 20.3-1 of ASCE 7-16. The values presented below are for the risk-targeted maximum considered earthquake (MCER).

2022 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2022 CBC Reference
Site Class	C	Section 1613.2.2
MCE_R Ground Motion Spectral Response Acceleration – Class B (short), S_S	1.663g	Figure 1613.2.1(1)
MCE_R Ground Motion Spectral Response Acceleration – Class B (1 sec), S_1	0.627g	Figure 1613.2.1(3)
Site Coefficient, F_A	1.2	Table 1613.2.3(1)
Site Coefficient, F_V	1.4	Table 1613.2.3(2)
Site Class Modified MCE_R Spectral Response Acceleration (short), S_{MS}	1.995g	Section 1613.2.3 (Eqn 16-20)
Site Class Modified MCE_R Spectral Response Acceleration – (1 sec), S_{M1}	0.877g*	Section 1613.2.3 (Eqn 16-21)
5% Damped Design Spectral Response Acceleration (short), S_{DS}	1.33g	Section 1613.2.4 (Eqn 16-22)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.585g*	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 (S1) 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 SM1 is increased by 50% for all applications of SM1. The values for parameters SM1 and SD1 presented above have not been increased in accordance with Supplement 3 of ASCE 7-16.

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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 MCEG Peak Ground Acceleration, PGA	0.707g	Figure 22-9
Site Coefficient, F_{PGA}	1.2	Table 11.8-1
Site Class Modified MCEG Peak Ground Acceleration, PGA_M	0.849g	Section 11.8.3 (Eqn 11.8-1)

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 7.16 magnitude event occurring at a hypocentral distance of 11.59 kilometers from the site.

Deaggregation was also performed for the Design Earthquake (DE) peak ground acceleration, and the result of the analysis indicates that the predominant earthquake contributing to the DE peak ground acceleration is characterized as a 7.03 magnitude occurring at a hypocentral distance of 16.84 kilometers from the site.

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

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6. LANDSLIDE EVALUATION

According to the State of California – Earthquake Zones of Required Investigation Map (CGS, 2025), portions of the site are located within a zone of required investigation for earthquake-induced landslides. Published geologic maps of the project area and the surrounding South Hill indicate the site is predominantly underlain by the Topanga Formation bedrock and the Glendora Volcanic Rocks (CGS, 2003; Dibblee and Minch, 2002; Tan, 2000). However, the geologic map by Tan shows landslide deposits mapped on the northern portion of the subject site and offsite to the east. It is noted that the mapped landslides are queried, indicating that their existence is uncertain.

Based on these considerations, this limited geotechnical investigation includes an evaluation for the presence of landslide deposits in the area of the proposed bike park. The evaluation consisted of a site reconnaissance, excavation and logging of a large diameter boring, and review of historical aerial photographs taken of the site to observe both current and previous conditions of the existing slopes and drainages within project area. No specific laboratory testing was performed as part of this landslide evaluation. Stability analyses of the slopes depicted in Cross Sections A-A' and B-B' (see Figure 2) are provided in Section 7 below.

The limits of the queried landslide encompass most of the bike park area including the slope that ascends to the southwest. In general, slope gradients range between 2:1 and 4:1 (H:V), with isolated areas that have been cut at steeper gradients up to approximately 3/4 :1 (H:V). The total slope height in the area of the queried landslide area is estimated to be approximately 130 feet. The slope is undeveloped and heavily vegetated with trees and bushes. Based on Geocon's visual site reconnaissance, surficial evidence of landsliding (such as a head scarp, bulging or terracing of the slope, or a break in vegetation) was not observed on the site.

A large diameter boring (B8) was excavated to provide direct observation of the geologic materials on site near the central portion of the queried landslide. Artificial fill was found to extend to 5 feet below existing ground surface and is underlain by the Topanga Formation. The Topanga Formation was observed to be predominantly sandstone and massive, however, contained siltstone and claystone interbeds that were between 1 and 3 inches thick, respectively. Measurements of the structural orientation of these beds were taken and indicated a strike to the northwest and dip at inclinations between 9 and 10 degrees to the northeast. Additional measurements of the Topanga were taken at the surface, where able (see Figure 2), and found the dip inclination to be between 11 and 16 degrees. These orientations are generally neutral to the slopes on site, with the exception of the north to northeast-facing slope, where they are slightly adverse. The downhole observations of the bedrock did not reveal landslide deposits or evidence of landsliding.

Historical aerial photographs of the project area taken between 1928 and 1956 were reviewed using a stereoscope. Historical aerial photos can be a useful tool for locating potential landslides due to the lack of development in an area, as natural features have yet to be covered by housing and infrastructure. Features indicative of landslides were not definitively observed in our historical aerial photograph review, however, evidence of heavy erosion and wasting was seen immediately north of the site. The photos showed the project area to consist of a relatively uniform ridge trending and descending to the north-northwest.

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Based on the surficial site observations, downhole observations within the large diameter boring, review of the historical aerial photographs, and the published geologic map by (Dibblee and Minch, 2002), it is Geocon's opinion that existing landslides do not exist within the area of the proposed bike park, and the potential for existing landslides to affect the site is considered low.

7. SLOPE STABILITY

A slope stability evaluation has been performed to determine the stability of the existing site configuration. Based on the results of the landslide hazard evaluation in the section above, no historic landslides were observed within the project area. Cross sections determined to be most critical to the proposed development are shown on the Geologic Map (see Figure 2) and illustrated on the Geologic Sections (see Figure 3). The scope of our analyses addresses the southeastern ascending slope above the proposed bike park improvements and the descending slope to the northwest.

Based on our geologic mapping and subsurface conditions encountered in our borings, the upper portion of the sections consists of the Glendora volcanics and the lower portion of the slope consists of Topanga formation sedimentary bedrock overlain by artificial fill. The descending western slope is generally inclined from to a 2H:1V and flattens to an approximate 7H:1V and the upper portion of the slope varies from an over steepened ¾H:1V condition to a flatter 4:1H:1V.

Prior to performing the slope stability analyses, review of laboratory data and historic geotechnical reports were performed to determine appropriate shear strength parameters to be applied in our slope stability models. Soil samples from the small diameter borings were obtained during excavation by driving a California Modified Sampler into the soil/rock mass. Samples of the bedrock were collected in boring B6 at the depths of approximately 9½ and 24½ feet by driving brass rings into the bedrock. These samples were utilized for determining residual strength parameters of the bedrock.

Samples of the Glendora Volcanics were not obtained during our investigation. In the absence of accurate laboratory data that reflects the true strength of the undisturbed bedrock, review of published documents was performed to determine more appropriate shear strength parameters. A review of the *Seismic Hazard Zone Report for the San Dimas Quadrangle* (California Geological Survey[CGS], 1998) was performed to determine regional shear strength parameters commonly associated with the Glendora Volcanics. The reported shear strengths are indicated in the table below. The Glendora Volcanics are generally extrusive rocks and mapped as basaltic palagonitic tuff and pillow lavas (Dibblee and Minch, 2002). Onsite, they were observed to be fine grained, massive, hard, slightly weathered, moderately fractured.

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Shear Strength Associated with Landslide Slip Surface Materials

Group ID	Formation Name	Mean/Median Phi (degrees)	Mean/Median C (psf)
Group 1	Tg	36.6 / 35.5	354 / 312

Note: Strengths are taken from San Dimas Open File Report for Group 1 Rock (testing was performed on Tpsq-Tps(crse) samples) (CGS, 1998)

Direct shear tests were performed on ring samples obtained during the site exploration. Residual shear strengths were obtained by running direct shear tests with multiple passes at a slower rate on ring samples of the Topanga Formation bedrock obtained from the hollow stem auger boring explorations. These test results are presented in Appendix B.

We performed deterministic slope stability analyses using the using the computer program Slide2 (Build Version 9.037 by Rocscience Inc.) for static and pseudo-static conditions using the Cuckoo method of limit-equilibrium analysis considering non-circular modes of failure. The slope depicted in Geologic Section A-A' and Section B-B' (Figure 3) was analyzed for gross static stability, as well as pseudo-static stability. Stability Analyses are presented in Appendix C.

Shear strength parameters are based on review of published documents, laboratory test data as well as our experience with similar soil conditions. The values used in the slope stability analyses are summarized in the tables below.

SUMMARY OF SOIL PROPERTIES USED FOR STATIC AND SURFICIAL SLOPE STABILITY ANALYSES

Material	Wet Density (pcf)	Friction Angle (degrees)	Cohesion (psf)	Remarks
Artificial Fill	120	34	60	Saturated, Ultimate
Glendora Volcanics	130	36	350	(CGS, 1998)
Topanga Formation (Cross Bedding)	125	36	200	Saturated, Ultimate
Topanga Formation (Along Bedding)	125	31	100	Saturated, Residual

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The apparent dip of the Topanga Formation bedding planes has been estimated between 9 and 15 degrees below the horizontal with respect Section A-A'. The bedding is considered neutral with respect to Section B-B'. Based on these considerations, when evaluating Section A-A' an anisotropic function was applied to the shear strength parameters of the Topanga Formation to model both along bedding and across bedding shear strength conditions. Where failure surfaces were aligned with along bedding conditions (between 9 and 15 degrees below the horizontal), the residual shear strength is utilized. Where the failure surface inclines outside of the adverse bedding range, an across bedding strength is applied. Isotropic parameters using cross bedded shear strengths were utilized when analyzing Section B-B'.

In accordance with the current standard of practice, as outlined in the "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Landslide Hazards in California" and "Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California", slopes should demonstrate the following conditions when analyzed for stability:

- Global Static Stability - minimum factor of safety of 1.5
- Global Pseudo-Static Stability - minimum factor of safety of 1.0 at screening criteria

Static slope stability analyses indicate multiple failure surfaces for Section A-A' with a factor of safety of less than 1.5, locally found at the over-steepened portion of the slope with a critical failure surface indicating a factor of safety of 1.35. Failure surfaces analyzed for Section B-B were all over the minimum factor of safety of 1.5 with the critical failure surface being 1.74. Slope configurations and critical surfaces are indicated on the slope stability printouts included in Appendix C.

In accordance with the "Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Evaluating and Mitigating Landslide Hazards in California (2002)" and the "Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A (2008)", seismic slope performance may be evaluated by first applying a screening analysis based on the seismic coefficient (keq). The seismic coefficient (keq) is based on the maximum horizontal acceleration (MHAr) and feq, a factor related to the seismicity of the site. Graphs to estimate the value of feq based on slope displacement, earthquake magnitude, rupture distance, and MHAr are available in SP117A.

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The MHAr corresponding to the Design Earthquake hazard level was taken as two-thirds of the PGA_M. A value of 0.566g was used as MHAr in the analysis. The modal magnitude and distance of the Design Earthquake (DE) ground motion was utilized for determination of keq which correspond to a 7.9 magnitude occurring at a hypocentral distance of 32.6 kilometers from the site.

Based on these input values, the seismic coefficient (keq) was evaluated as 0.275g. This value of ground acceleration was applied in the pseudo-static slope stability analyses. Analysis of geologic section A-A' indicates a factor of safety of 0.91 for gross stability when subject to the seismic coefficient acceleration. Analysis of geologic section B-B' indicates a factor of safety of 1.06 for gross stability when subject to the seismic coefficient acceleration. Since the pseudo-static stability analysis results in a factor of safety of greater than 1.0, the slope passes the screening criteria and analysis of slope deformations is not required. Slope configurations and critical surfaces are indicated on the slope stability printouts included in Appendix C.

Based on the results of our analyses, the FS against failure under the specified conditions for Section B-B' are 1.74 for static conditions and 1.06 for seismic conditions, which indicate "stable" conditions. However, both the static and seismic analyses indicate an "unstable" condition for Section A-A' due to the presence of an over steepened portion of the slope directly above the proposed bike park.

It is recommended that remedial grading be performed at the site to trim back the over steepened slope to a gradient of 2H:1V or flatter. Subsequent to the recommended grading, the remediated slopes are expected to have a satisfactory factor of safety. To confirm this assumption, a second analysis of section A-A' was performed using revised ground geometry. When the over steepened portion of the slope is trimmed to a 2H:1V, no failure surfaces below a factor of safety of 1.5 were found for static conditions, and a factor of safety of 1 or greater is achieved for seismic conditions. Critical failure surfaces for the revised ground geometry are indicated on the slope stability printouts included in Appendix C.

Program input and calculations are presented in the report included in Appendix C.

This office should be contacted once a formal grading plan for the project has been established to evaluate the stability of the proposed slope configurations.

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8. CONCLUSIONS AND RECOMMENDATIONS

8.1 General

8.1.1 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.

8.1.2 The results of the slope stability analyses indicate that unstable slope configurations are currently present at the site. It is recommended that the over steepened slope above and to the southeast of the proposed bike park be trimmed to at least a 2H:1V or flatter. Preliminarily, it is recommended that other slopes on the site be trimmed to 2H:1H, or flatter. Geocon should be contacted once a grading plan becomes available to further evaluate this recommendation and evaluate the stability of the proposed slopes.

8.1.3 Based on site explorations, the area of proposed improvement appears to be mantled by 1 to 4½ feet of existing artificial fill. In addition, deeper accumulations of artificial fill on the order of 5 to 7 feet in depth were encountered at borings B2, B5, and B7. 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 considered suitable for direct support of proposed foundations and slabs. The existing fill soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 8.4).

8.1.4 Occasional residual soil, or topsoil, was encountered below the fill and above the Topanga Formation bedrock. The residual soil is not considered suitable for direct support of proposed foundations and slabs. The residual soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 8.4).

8.1.5 Based on manufacturer research for pre-fabricated vault restroom structures, it is our understanding that vault restrooms are typically founded up to 5 feet below the existing grade. Excavations for the proposed vault restroom structure is anticipated to extend through the existing fill soils and penetrate into bedrock. Any remaining residual, loose, or disturbed soils should be compacted to a dense state or penetrated by proposed excavations at the direction of the Geotechnical Engineer (a representative of Geocon). All excavations must be observed and approved in writing by the Geotechnical Engineer or a representative of Geocon. Recommendations for vault restroom foundations are provided in section 8.5.

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8.1.6 At a minimum, it is recommended that where new foundations are proposed for the shade pavilion, the upper 4 feet of existing earth materials within the proposed footprint areas be excavated and properly compacted for foundation and slab support. All existing fill and/or residual soils should be removed from the excavated area. The excavation should extend laterally a minimum distance of 3 feet beyond the structure footprint area or a distance equal to the depth of fill below the foundation, whichever is greater. Proposed foundations should be underlain by a minimum of 2 feet of newly placed engineered fill. The limits of existing fill and/or soft alluvial soils removal will be verified by the Geocon representative during site grading activities. Recommendations for earthwork are provided in the *Grading* section of this report (see Section 8.4).

8.1.7 Subsequent to the recommended grading, the proposed shade pavilion may be supported on a conventional foundation system, consisting of shallow spread footings and a slab-on-grade deriving support from newly placed engineered fill. Alternatively, the proposed foundations may be deepened to derive support in undisturbed bedrock. However, transitioning the foundation across bedrock to soil is not recommended due to the potential for differential settlement. The structure should be designed for all foundations to derive support in a uniform material. Once a foundation plan becomes available, supplemental borings can be performed to determine the depth to bedrock to help inform the design teams decision on whether or not to deepen foundations into bedrock. Recommendations for conventional foundations are provided in section 8.6.

8.1.8 Additional grading should be conducted as necessary to maintain the required 2 feet of newly placed engineered fill below foundations. The grading contractor should verify all bottom of footing elevations prior to commencement of grading activities to ensure that grading is conducted deep enough to provide the required two foot of engineering fill below foundations.

8.1.9 It is our understanding that potential gabion wall improvements may be utilized at areas along the edge of the proposed parking area. However, anticipated finished elevations for the proposed parking lot and the proposed height and inclination of the gabion wall have not been determined. It is recommended that once the design phase and configuration of proposed gabion wall improvements proceeds to a more finalized plan, recommendations can be provided under a separate cover, if necessary.

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8.1.10 Excavations up to 5 feet in vertical height are anticipated for the vault restroom structure installation and foundation excavations. Sloping and/or shoring measures will be required in order to provide a stable excavation. Recommendations for *Temporary Excavations* are provided in Section 8.13 of this report.

8.1.11 Where new paving is to be placed, it is recommended that all existing fill soils and soft soils 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 or unsuitable soils 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 soil should be scarified and properly compacted for paving support. Paving recommendations are provided in the *Preliminary Pavement Recommendations* section of this report (see Section 8.10).

8.1.12 Based on the results of percolation testing performed at the site, a stormwater infiltration system is not considered feasible for this project. It is recommended that stormwater be retained, filtered, and discharged in accordance with the requirements of the local governing agency. Discussion of the percolation testing and results are provided in the *Stormwater Infiltration* section of this report (see Section 8.14).

8.1.13 Once the design of proposed improvements proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary.

8.1.14 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.

8.2 Soil and Excavation Characteristics

8.2.1 The in-situ fill soils, residual soils, and weathered portions of the Topanga Formation can be excavated with moderate effort using conventional excavation equipment. Caving and sloughing should be anticipated in unshored vertical excavations, especially where loose, granular, or uncemented soils are encountered. The contractor should be aware concretions or well cemented areas may be encountered in the bedrock which could make excavation conditions difficult. The excavation contractor should be prepared for these difficult conditions.

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8.2.2 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.

8.2.3 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 8.13).

8.2.4 The upper five feet of existing site soils encountered during the investigation are considered to have a "very low" expansive potential (EI = 11) and are classified as "non-expansive" in accordance with the 2022 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that proposed improvements and slabs will derive support in these materials.

8.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

8.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered "moderately corrosive" with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B17) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that PVC, ABS or other approved plastic piping be utilized in lieu of cast-iron when in direct contact with the site soils.

8.3.2 Laboratory tests were performed on representative samples of the on-site soil to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B15 and indicate that the on-site soil possess a sulfate exposure class of "S0" to concrete structures as defined by 2022 CBC Section 1904A and ACI 318-19 Chapter 19.

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8.3.3 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

8.4 Grading

8.4.1 Grading is anticipated to include excavation of site soils for the installation vault restroom structure, foundation excavations, utility trenches, and placement of backfill for proposed improvements.

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

8.4.3 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, as well as the recommendations presented herein and on the approved plans.

8.4.4 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and alluvial soil encountered during exploration are suitable for re-use as an engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris are removed.

8.4.5 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. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

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8.4.6 The results of the slope stability analyses indicate that unstable slope configurations are currently present at the site. It is recommended that the over steepened slope above and to the southeast of the proposed bike park be trimmed to at least a 2H:1V or flatter. Preliminarily, it is recommended that other slopes on the site be trimmed to 2H:1H, or flatter. Geocon should be contacted once a grading plan becomes available to further evaluate this recommendation and evaluate the stability of the proposed slopes.

8.4.7 As a minimum, it is recommended that the upper 4 feet of existing earth materials within the proposed shade pavilion area be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as necessary to remove deeper artificial fill or soft residual soil at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 3 feet beyond the foundation footprint area, or a distance equal to the depth of fill below the foundation, whichever is greater. Proposed building foundations should be underlain by a minimum of 2 feet of newly placed engineered fill. The limits of existing fill and/or soft alluvial soils removal will be verified by the Geocon representative during site grading activities.

8.4.8 Additional grading should be conducted as necessary to maintain the required 2 feet of newly placed engineered fill below foundations. The grading contractor should verify all bottom of footing elevations prior to commencement of grading activities to ensure that grading is conducted deep enough to provide the required two foot of engineering fill below foundations.

8.4.9 The proposed vault restroom structure may derive support in undisturbed bedrock. Foundations should be deepened to penetrate through the existing artificial fill and soils to derive support exclusively in bedrock. All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

8.4.10 The area below the proposed pre-fabricated vault restroom structure should be excavated per the manufacturer's requirements. We expect that the bottom of the vault excavations will be founded in undisturbed competent bedrock. However, per typical manufacturer's requirements, a minimum 2-inch-thick leveling course of ¾-inch or ½-inch crushed rock may be required below the vault. Although not anticipated, if wet and unstable excavation bottoms are encountered, the crushed rock layer should be thickened to 12 to 18 inches. In addition, the crushed rock should be surrounded by a durable geotextile filter fabric, such as Mirafi 140N or equal. The crushed rock leveling should be consolidated and smoothed using a vibroplate compactor.

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8.4.11 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to optimum moisture content, and properly compacted to a minimum 90 percent of the maximum dry density in accordance with ASTM D 1557 (latest edition).

8.4.12 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Import fill should consist of the characteristics presented in the table below.

SUMMARY OF IMPORT FILL RECOMMENDATIONS

Soil Characteristic	Values
Expansion Potential	"Very Low" (Expansion Index of 20 or less)
Particle Size	Maximum Dimension Less Than 6 Inches
	Free of Debris
Corrosivity	Less Detrimental Than Existing Onsite Soils (see Figure B15)

8.4.13. Where new paving is to be placed, it is recommended that all existing fill and soft soils be excavated and properly compacted for paving support. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to optimum moisture content, and compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 8.10).

8.4.14 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 as backfill is also acceptable. Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).

8.4.15 All 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.

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8.5 Vault Restroom Foundation Design

8.5.1 It is our understanding that the restroom buildings may consist of pre-fabricated concrete vault restroom structures, such as CXT or similar. The pre-fabricated vault restroom structure should penetrate through the existing artificial fill and derive support in undisturbed, competent bedrock. Any soils disturbed should be compacted to a dense state or penetrated by proposed excavations at the direction of the Geotechnical Engineer (a representative of Geocon).

8.5.2 The recommended maximum allowable bearing value for the design of the vault restroom foundation is 2,000 pounds per square foot (psf). The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.

8.5.3 The maximum settlement for vault restroom structure deriving support in the recommended bearing materials and designed with a maximum bearing pressure of 2,000 psf is estimated to be less than ½ inch and occur below the heaviest loaded structural element. Settlement of the vault restroom structure is expected to occur on initial application of loading. Differential settlement is not expected to exceed ¼ inch over a distance of 20 feet.

8.5.4 It is recommended that a modulus of subgrade reaction of 150 pounds per cubic inch (pci) be utilized for the design of the foundation bearing in undisturbed bedrock. This value is a unit value for use with a one-foot square footing. The modulus should be reduced in accordance with the following equation when used with larger foundations:

$$K_R = K\left[\frac{B+1}{2B}\right]^2$$

where:
K_R = reduced subgrade modulus
K = unit subgrade modulus
B = foundation width (in feet)

8.5.5 The thickness of and reinforcement of the vault restroom foundation should be designed by the manufacturer or project structural engineer.

8.5.6 For seismic design purposes, a coefficient of friction of 0.4 may be utilized between concrete slab and undisturbed competent residual soil and/or bedrock, or newly placed engineered fill without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

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8.5.7 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 exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.

8.5.8 This office should be provided a copy of the final construction plans so that the recommendations presented herein could be properly reviewed and revised if necessary.

8.6 Conventional Foundation Design – Shade Pavilion

8.6.1 Subsequent to the recommended grading, a conventional foundation system may be utilized for support of the proposed shade pavilion structure provided foundations derive support in newly placed engineered fill. Foundations should be underlain by a minimum of 2 feet of newly placed engineered fill.

8.6.2 Continuous footings may be designed for an allowable bearing capacity of 1,500 pounds per square foot (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.

8.6.3 Isolated spread foundations may be designed for an allowable bearing capacity of 1,800 psf, and should be a minimum of 24 inches in width, 18 inches in depth below the lowest adjacent grade, and 12 inches into the recommended bearing material.

8.6.4 The allowable soil bearing pressure above may be increased by 250 psf and 500 psf for each additional foot of foundation width and depth, respectively, up to a maximum allowable soil bearing pressure of 2,500 psf.

8.6.5 The allowable bearing pressures may be increased by one-third for transient loads due to wind or seismic forces.

8.6.6 The maximum settlement for shade pavilion structure deriving support in the recommended bearing materials and designed with a maximum bearing pressure of 2,500 psf is estimated to be less than ¾ inch and occur below the heaviest loaded structural element. Settlement of the shade pavilion structure is expected to occur on initial application of loading. Differential settlement is not expected to exceed ½ inch over a distance of 20 feet.

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8.6.7 This office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary. Additional grading should be conducted as-needed in order to maintain the required 2-foot thick blanket of engineered fill below proposed foundations.

8.6.8 Continuous footings should be reinforced with four No. 4 steel reinforcing bars, two placed near the top of the footing and two near the bottom. Reinforcement for spread footings should be designed by the project structural engineer.

8.6.9 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only, and are not intended to be used in lieu of those required for structural purposes.

8.6.10 No special subgrade presaturation is required prior to placement of concrete. However, the slab and foundation subgrade should be sprinkled as necessary; to maintain a moist condition as would be expected in any concrete placement.

8.6.11 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. If unanticipated soil conditions are encountered, foundation modifications may be required.

8.6.12 This office should be provided a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

8.7 Miscellaneous Foundations

8.7.1 Foundations for small outlying structures, such as non-retaining 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. Where excavation and compaction cannot be performed, foundations may derive support directly in the competent undisturbed bedrock and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.

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8.7.2 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, 24 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.

8.7.3 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.

8.8 Lateral Design

8.8.1 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 0.4 may be used with the dead load forces in undisturbed, competent residual soil and/or bedrock, or newly placed engineered fill.

8.8.2 Passive earth pressure for the sides of foundations and slabs poured against properly compacted engineered fill may be computed as an equivalent fluid having a density of 270 pcf with a maximum earth pressure of 2,700 psf. Passive earth pressure for the sides of foundations and slabs poured against undisturbed competent residual soil or bedrock may be computed as an equivalent fluid having a density of 300 pcf with a maximum earth pressure of 3,000 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

8.9 Exterior Slabs-on-Grade

8.9.1 Exterior concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the *Preliminary Pavement Recommendations* section of this report (Section 8.10).

8.9.2 For seismic design purposes, a coefficient of friction of 0.4 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.

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8.9.3 Exterior slabs for walkways and flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 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 near optimum moisture content and properly compacted to at least 95 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. Construction joints should be designed by the project structural engineer.

8.9.4 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.

8.9.5 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, stucco walls, 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.

8.10 Preliminary Pavement Recommendations

8.10.1 Where new paving is to be placed, it is recommended that all existing fill and unsuitable soil be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and unsuitable soil materials in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable soil materials 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 near optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).

8.10.2 The following pavement sections are based on an assumed R-Value of 25. 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.

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8.10.3 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	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	11.0

8.10.4 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 (CMB) 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).

8.10.5 Unless specifically designed and evaluated by the project structural engineer, where exterior 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. 3 steel reinforcing bars placed 18 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 95 percent relative compaction, respectively, as determined by ASTM Test Method D 1557 (latest edition).

8.10.6 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.

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8.11 Retaining Wall Design (Vault Restroom Structure)

8.11.1 The recommendations presented below are generally applicable to the design of rigid concrete retaining walls having a maximum height of 5 feet. In the event that walls higher than 5 feet are planned, Geocon should be contacted for additional recommendations.

8.11.2 Retaining wall foundations may be designed in accordance with the recommendations provided in the *Vault Restroom Foundation Design* sections of this report (see Sections 8.5).

8.11.3 Retaining walls with a level backfill surface that are not restrained at the top should be designed utilizing a triangular distribution of pressure (active pressure). Restrained walls are those that are not allowed to rotate more than 0.001H (where H equals the height of the retaining portion of the wall in feet) at the top of the wall. Where walls are restrained from movement at the top, walls may be designed utilizing a triangular distribution of pressure (at-rest pressure). The table below presents recommended pressures to be used in retaining wall design, assuming that proper drainage will be maintained.

RETAINING WALL WITH LEVEL BACKFILL SURFACE

HEIGHT OF WALL (Feet)	ACTIVE PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)	AT-REST PRESSURE EQUIVALENT FLUID PRESSURE (Pounds Per Cubic Foot)
Up to 5	30	57

8.11.4 The wall pressures provided above assume that the proposed retaining walls will support newly placed engineered fill derived from onsite soils. If import soil will be used to backfill proposed retaining walls, revised earth pressures may be required to account for the geotechnical properties of the import soil used 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.

8.11.5 The wall pressures provided above assume that the retaining wall will be properly drained preventing the buildup of hydrostatic pressure. If wall drainage is not implemented, typical of vault structure improvements, the equivalent fluid pressure to be used in design of undrained walls is 90 pcf.

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8.11.6 Additional active 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. The surcharge pressure should be evaluated in accordance with the recommendations in Section 8.15 of this report.

8.11.7 In addition to the recommended earth pressure, the upper 10 feet of the vault 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 wall due to normal street traffic. If the traffic is kept back at least 10 feet from the wall, the traffic surcharge may be neglected.

8.11.8 Seismic lateral forces will be required for any retaining walls in excess of 6 feet. Recommendations for seismic lateral forces can be provided under a separate cover, if necessary.

8.12 Retaining Wall Drainage

8.12.1 Retaining walls not designed for hydrostatic pressures should be provided with a drainage system extended at least two-thirds the height of the wall. At the base of the drain system, a subdrain covered with a minimum of 12 inches of gravel should be installed, and a compacted fill blanket or other seal placed at the surface (see Figure 4). The clean bottom and subdrain pipe, behind a retaining wall, should be observed by the Geotechnical Engineer (a representative of Geocon), prior to placement of gravel or compacting backfill.

8.12.2 As an alternative, a plastic drainage composite such as Miradrain or equivalent may be installed in continuous, 4-foot-wide columns along the entire back face of the wall, at 8 feet on center. The top of these drainage composite columns should terminate approximately 18 inches below the ground surface, where either hardscape or a minimum of 18 inches of relatively cohesive material should be placed as a cap (see Figure 5). These vertical columns of drainage material would then be connected at the bottom of the wall to a collection panel or a 1-cubic-foot rock pocket drained by a 4-inch subdrain pipe.

8.12.3 Subdrainage pipes at the base of the retaining wall drainage system should outlet to an acceptable location via controlled drainage structures. Drainage should not be allowed to flow uncontrolled over descending slopes.

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8.12.4 Moisture affecting below grade walls is one of the most common post-construction complaints. Poorly applied or omitted waterproofing can lead to efflorescence or standing water. Particular care should be taken in the design and installation of waterproofing to avoid moisture problems, or actual water seepage into the structure through any normal shrinkage cracks which may develop in the concrete walls, floor slab, foundations and/or construction joints. The design and inspection of the waterproofing is not the responsibility of the geotechnical engineer. A waterproofing consultant should be retained in order to recommend a product or method, which would provide protection to subterranean walls, floor slabs and foundations.

8.13 Temporary Excavations

8.13.1 Excavations on the order of 5 feet in height may be required for excavation and construction of proposed improvements, including foundation depths. The excavations are expected to expose artificial fill, residual soils, and bedrock which are subject to caving where granular soils are encountered. Vertical excavations up to 5 feet in height may be attempted where not surcharged by adjacent traffic or structures. The bedrock encountered during this investigation is predominantly massive. However, some adverse geologic conditions could occur, depending on the nature of the excavation. All excavations should be observed by a Project Geologist (a representative of Geocon).

8.13.2 Excavations into bedrock, including the trimming of over steepened slopes, should be observed by the Project Geologist (a representative of Geocon West, Inc.) during excavation to check for the presence of jointing or bedding which may require special recommendations for sloping and/or shoring. Any recommendations deemed necessary will be provided at that time.

8.13.3 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 a maximum of 10 feet in height. A uniform slope does not have a vertical portion. Where space is limited, shoring measures will be required.

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8.13.4 Where temporary construction 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 cut 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.

8.14 Stormwater Infiltration

8.14.1 During the site explorations on October 9, 2024, borings B2 and B3 were utilized to determine the feasibility of stormwater infiltration at the site. The borings were excavated to the depths listed in the table below. Percolation testing was conducted at the depths listed in the table below. Slotted casing was placed in the borings, and the annular space between the casing and excavation were filled with filter pack. The borings were then filled with water to pre-saturate the soils. The casings were refilled with water and percolation test readings were performed after repeated flooding of the cased excavation.

8.14.2 Based on the test results, the field-measured percolation rate and the design infiltration rate are provided in the following table. The Reduction Factor (Rf), to convert the field-measured percolation rate to an infiltration rate, is also shown in the table below. This value has been calculated in accordance with the Small Diameter Boring Infiltration Test Procedure in the County of Los Angeles Department of Public Works GMED *Guidelines for Design, Investigation, and Reporting Low Impact Development Stormwater Infiltration* (June 2021). Calculations of the percolation rate, reduction factor, and infiltration rate are provided on Figures 6 and 7.

Boring	Infiltration Depth (ft)	Field Measured Infiltration Rate (in/hour)	Reduction Factor (Rf)	Design Infiltration Rate (in / hour)
B2	3 – 5	0.48	3	0.16
B3	2 – 5	0.33	3	0.11

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8.14.3 Based on the test method utilized (Small Diameter Boring), the reduction factor RF_t may be taken as 1.0 in the infiltration system design. Based on the number of tests performed and consistency of the soils throughout the site, it is suggested that the reduction factor RF_v be taken as 1.0. In addition, provided proper maintenance is performed to minimize long-term siltation and plugging, the reduction factor RF_s may be taken as 1.0. 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.

8.14.4 The results of the percolation testing indicate that the infiltration rate of the soils at the depths and locations indicated in the table above are not conducive to infiltration. It is recommended that stormwater be retained, filtered, and discharged in accordance with the requirements of the local governing agency.

8.15 Surcharge from Adjacent Structures and Improvements

8.15.1 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.

8.15.2 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.

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8.15.3 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, *Qp* 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*.

8.16 Surface Drainage

8.16.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the 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 should be maintained at all times.

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8.16.2 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 2022 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 5 feet of building perimeters. 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 5 feet of footings except when enclosed in protected planters.

8.16.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures.

8.16.4 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 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.

8.17 Plan Review

8.17.1 Grading, foundation, and shoring plans (if necessary) should be reviewed by the Geotechnical Engineer (a representative of Geocon West, Inc.), 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.

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LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. 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.

2. 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 Incorporated 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 Incorporated.

3. This report is issued with the understanding that it is the responsibility of the owner or 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.

4. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they be 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.

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LIST OF REFERENCES

1. California Division of Mines and Geology, 1999, *State of California Seismic Hazard Zones, South Gate Quadrangle, Official Map,* Released: March 25, 1999.

2. California Division of Mines and Geology, 1998, *Revised 2006, Seismic Hazard Zone Report for the San Dimas 7.5-Minute Quadrangle, Los Angeles County, California,* Seismic Hazard Zone Report 032.

3. California Geological Survey, 2003, *Geologic Map of the San Bernardino 30' x 60' Quadrangle, California,* Regional Geologic Map Series, 1:100,000 Scale.

4. California Geological Survey, 2025, Earthquake Zones of Required Investigation, https://maps.conservation.ca.gov/cgs/EQZApp/app/.

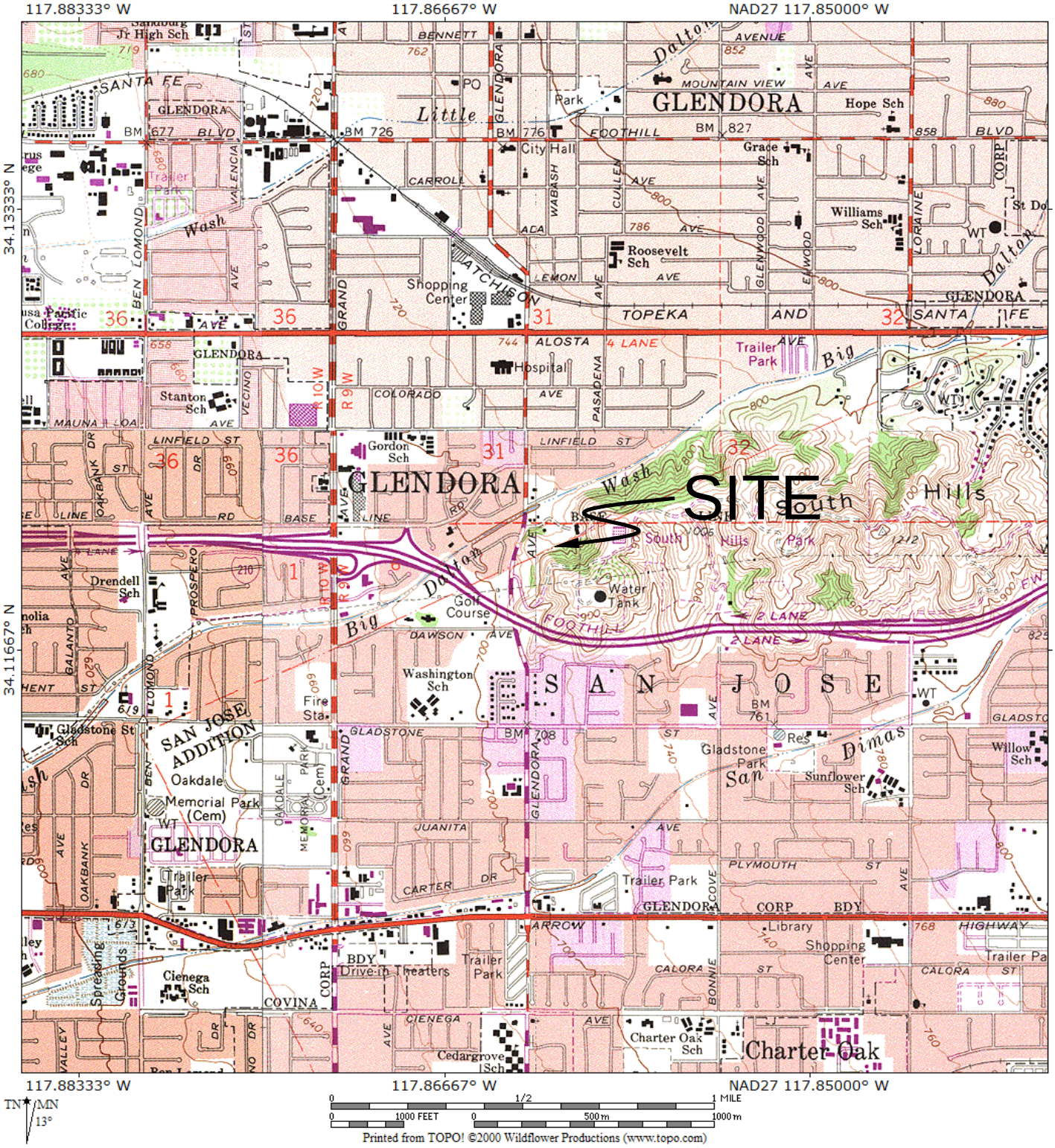
5. Dibblee, T.W. and Minch, J.A., 2002, *Geologic map of the San Dimas and Ontario Quadrangles, Los Angeles and San Bernardino Counties, California,* Dibblee Geological Foundation, Dibblee Foundation Map DF-91, 1:24,000.

6. OSHPD Seismic Design Maps Web Application, *https://seismicmaps.org/*

7. Tan, S.S., 2000, *Geologic map of the San Dimas 7.5 quadrangle, Los Angeles County, California*, California Division of Mines and Geology, Open-File Report 98-31, 1:24,000.

8. U.S. Geological Survey Landslide Hazard Program, 2024a, U.S. Landslide Inventory, accessed November, 2024, from USGS web site: https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=ae120962f459434b8c904b456c82669d.Risk Engineering, *EZ-FRISK*, 2016.

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

GEOCON WEST, INC.

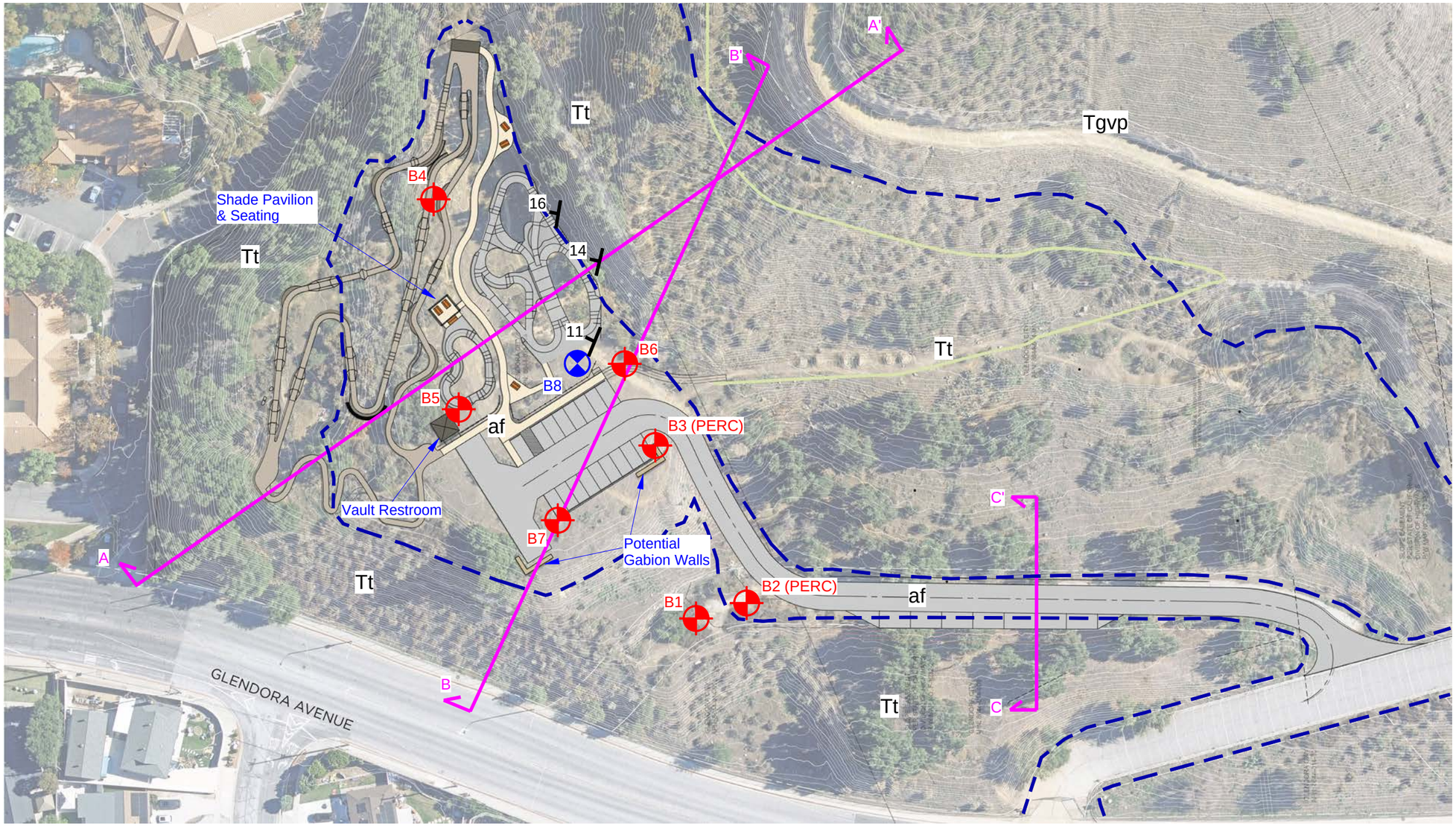
ENVIRONMENTAL GEOTECHNICAL MATERIALS
500 N. VICTORY BLVD. - BURBANK, CA 91502
PHONE (818) 841-8388 - FAX (818) 841-1704

VICINITY MAP

GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

DRAFTED BY: LW	CHECKED BY: GAK	JAN. 2025	PROJECT NO. W1983-06-01	FIG. 1

NOTE: DRAWING BY AVID TRAILS - ILUSTRATION PURPOSE ONLY. NOT FOR CONSTRUCTION



LEGEND

B7 — Number and Location of Hollow Stem Auger Boring

B8 — Number and Location of Bucket Auger Boring

A A' — Cross Section (see Figure 3)

Tgvp	Glendora Volcanic Rocks
Tt	Topanga Formation
af	Artificial Fill
16	Strike and Dip

0 80' 160'

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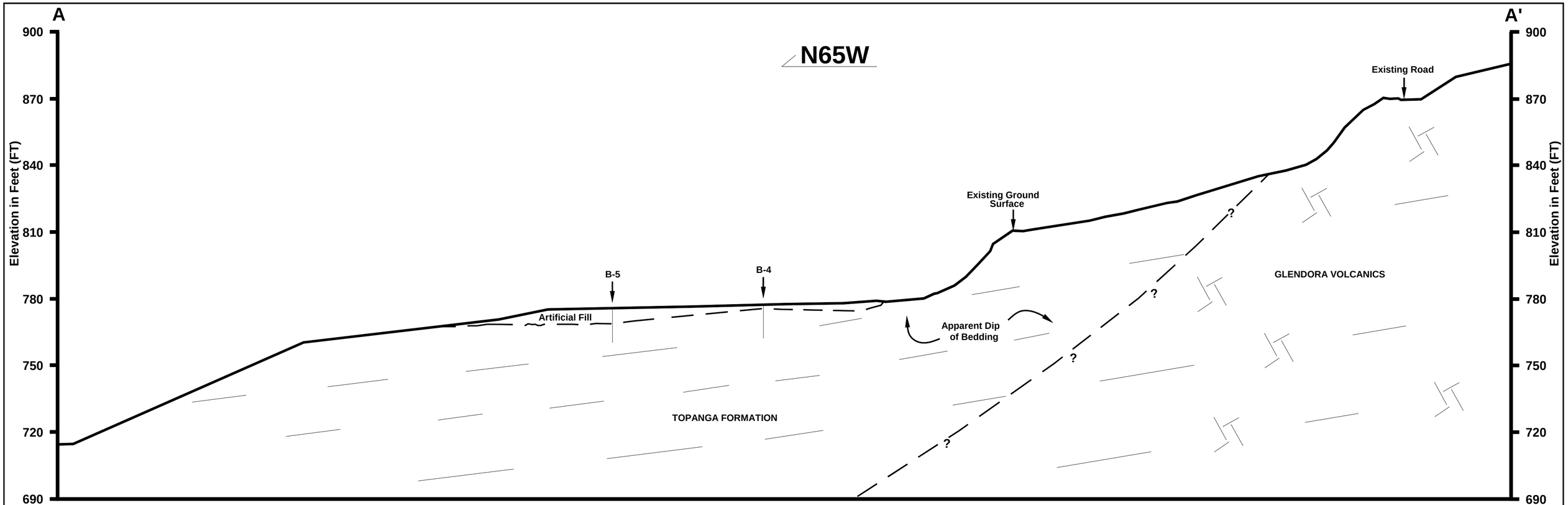
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PHONE (818) 841-8388 - FAX (818) 841-1704

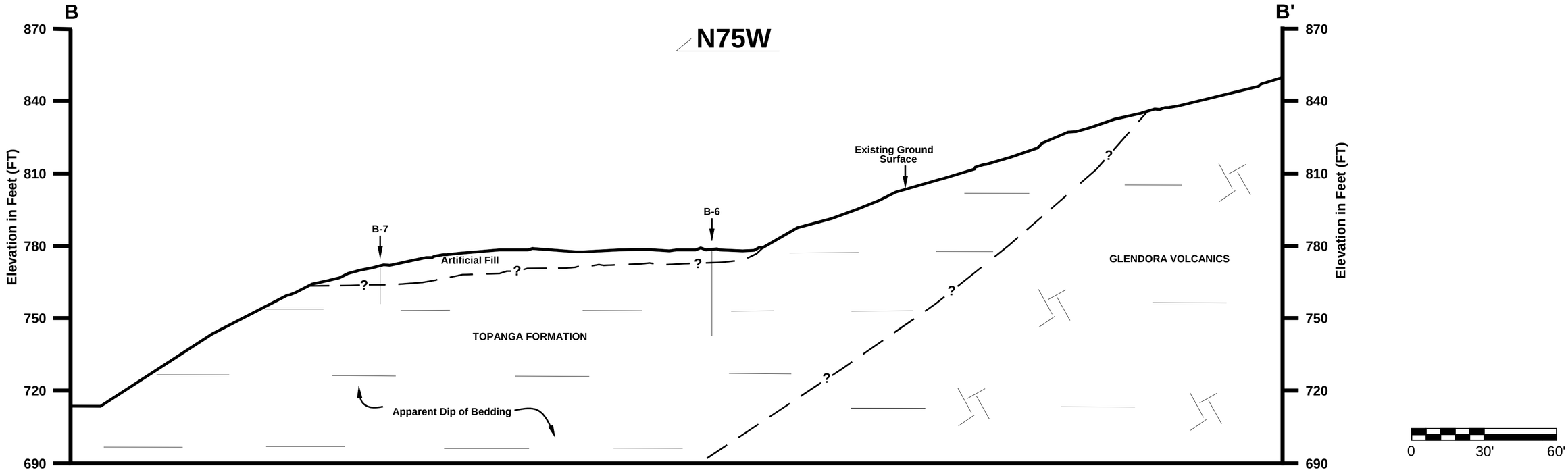
DRAFTED BY: RP | CHECKED BY: MC

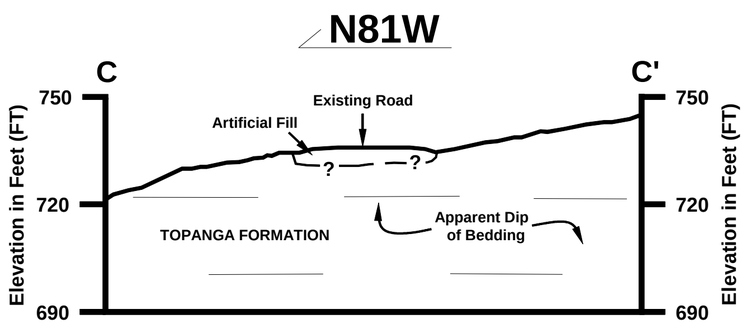
SITE PLAN AND GEOLOGIC MAP

GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

JAN. 2025 | PROJECT NO. W1983-06-01 | FIG. 2







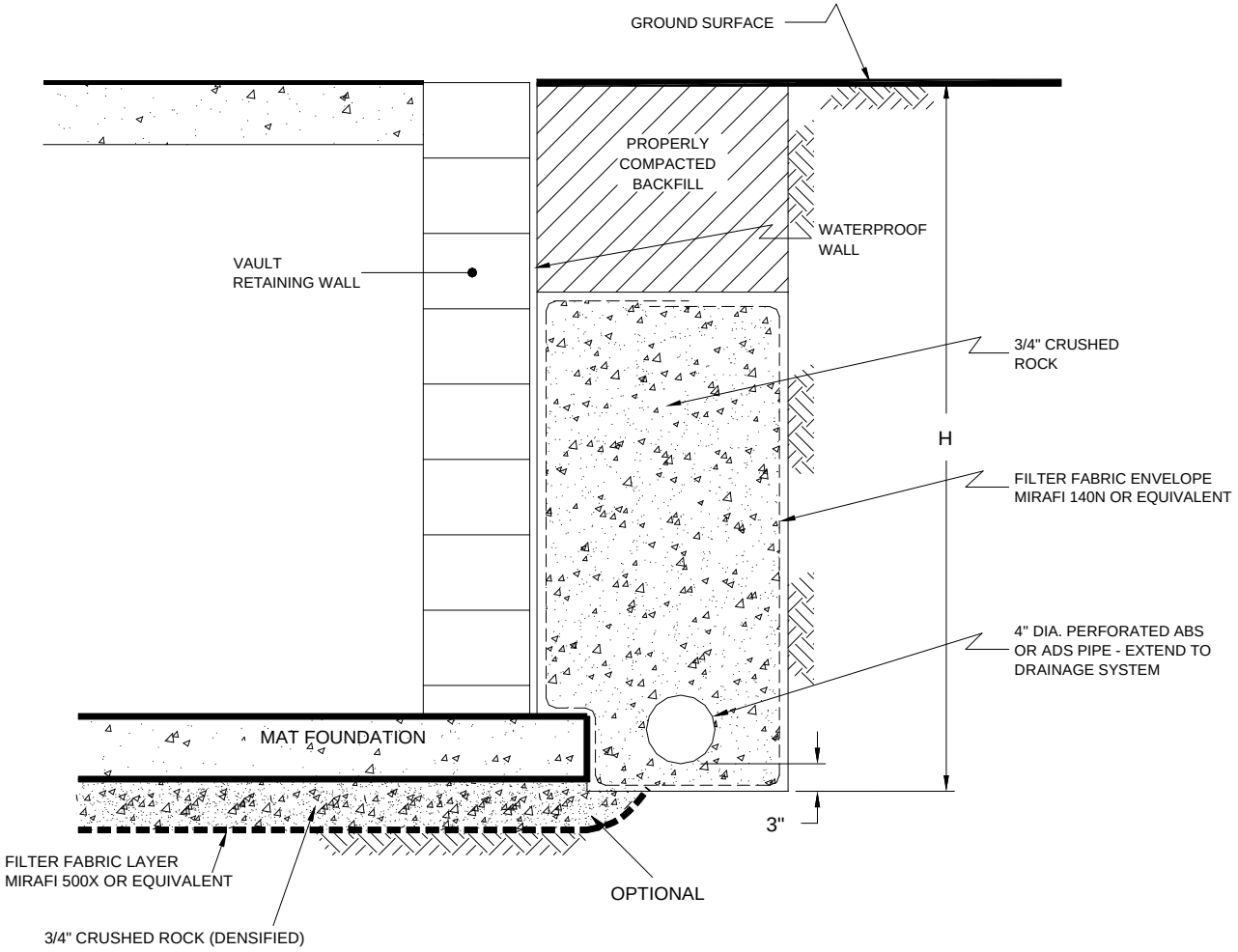
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WEST, INC.
ENVIRONMENTAL GEOTECHNICAL MATERIALS
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PHONE (818) 841-8388 - FAX (818) 841-1704

DRAFTED BY: JAO	CHECKED BY: GK

CROSS SECTIONS

GLENDORE FAMILY BIKE PARK
SOUTH HILLS WILDERNESS PARK
GLENDORA, CALIFORNIA

JAN. 2025	PROJECT NO. W1983-06-01	FIG. 3



NO SCALE

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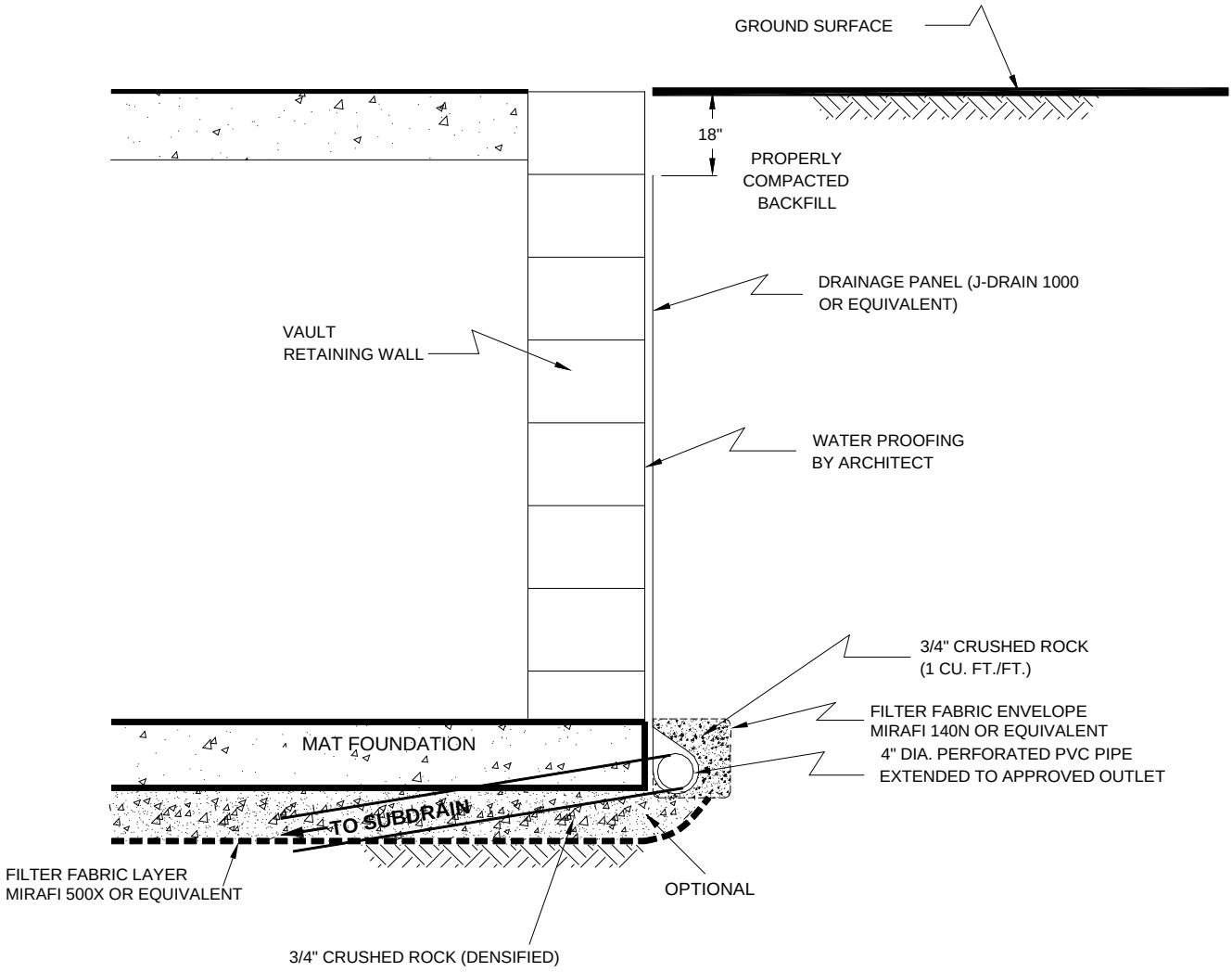
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RETAINING WALL DRAIN DETAIL

GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

DRAFTED BY: RP	CHECKED BY: JTA/NDB

JAN. 2025	PROJECT NO. W1983-06-01	FIG. 4



NOTE: TOP OF DRAINAGE PANEL NOT MORE THAN 18 INCHES FROM GROUND SURFACE

NO SCALE

GEOCON WEST, INC.

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PHONE (818) 841-8388 - FAX (818) 841-1704

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RETAINING WALL DRAIN DETAIL

GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

JAN. 2025 | PROJECT NO. W1983-06-01 | FIG. 5

BORING PERCOLATION TEST FIELD LOG

Date:	Wednesday, October 9, 2024	**Boring/Test Number:**	B2	
Project Number:	W1983-06-01	**Diameter of Boring:**	7	inches
Project Location:	Glendora Bike Park	**Diameter of Casing:**	2	inches
Earth Description:	SM	**Depth of Boring:**	5.5	feet
Tested By:	JJK	**Depth to Invert of BMP:**	3	feet
Liquid Description:	Water	**Depth to Water Table:**	n/a	feet
Measurement Method:	Sounder	**Depth to Initial Water Depth (d_1):**	36	inches
Start Time for Pre-Soak:	8:05 AM	**Water Remaining in Boring (Y/N):**	Yes	
Start Time for Standard:	9:05 AM	**Standard Time Interval Between Readings:**	30 min	

Reading Number	Time Start (hh:mm)	Time End (hh:mm)	Elapsed Time Δtime (min)	Water Drop During Standard Time Interval, Δd (in)	Soil Description Notes Comments
1	9:10 AM	9:40 AM	30	5.6	
2	9:42 AM	10:12 AM	30	4.3	
3	10:15 AM	10:45 AM	30	4.2	
4	10:50 AM	11:20 AM	30	4.9	
5	11:30 AM	12:00 PM	30	5.0	
6	12:10 PM	12:40 PM	30	4.3	Stabilized Readings
7	12:50 PM	1:20 PM	30	4.3	Achieved with Readings
8	1:30 PM	2:00 PM	30	4.3	6, 7, and 8

MEASURED PERCOLATION RATE & DESIGN INFILTRATION RATE CALCULATIONS*

* Calculations Below Based on Stabilized Readings Only

Boring Radius, r: 3.5 inches

Test Section Height, h: 30.0 inches

$$Test\ Section\ Surface\ Area, A = 2\pi rh + \pi r^2$$

A = **698** in^2

$$Discharged\ Water\ Volume, V = \pi r^2 \Delta d$$

$$Percolation\ Rate = \left(\frac{V/A}{\Delta T}\right)$$

Reading 6	V =	**166**	in^3	Percolation Rate =	**0.48** inches/hour
Reading 7	V =	**166**	in^3	Percolation Rate =	**0.48** inches/hour
Reading 8	V =	**166**	in^3	Percolation Rate =	**0.48** inches/hour

Measured Percolation Rate = **0.48** inches/hour

Reduction Factors

Small Diameter Boring, RF_t = 1

Site Variability, RF_v = 1

Long Term Siltation, RF_s = 1

$$Total\ Reduction\ Factor, RF = RF_t + RF_v + RF_s$$

Total Reduction Factor = 3

Design Infiltration Rate

$$Design\ Infiltration\ Rate = Measured\ Percolation\ Rate\ /RF$$

Design Infiltration Rate = **0.16** inches/hour

FIGURE 6

BORING PERCOLATION TEST FIELD LOG

Date:	Wednesday, October 9, 2024	**Boring/Test Number:**	B3	
Project Number:	W1983-06-01	**Diameter of Boring:**	7	inches
Project Location:	Glendora Bike Park	**Diameter of Casing:**	2	inches
Earth Description:	SM	**Depth of Boring:**	5.5	feet
Tested By:	JJK	**Depth to Invert of BMP:**	2	feet
Liquid Description:	Water	**Depth to Water Table:**	n/a	feet
Measurement Method:	Sounder	**Depth to Initial Water Depth (d_1):**	24	inches
Start Time for Pre-Soak:	9:08 AM	**Water Remaining in Boring (Y/N):**	Yes	
Start Time for Standard:	10:08 AM	**Standard Time Interval Between Readings:**	30 min	

Reading Number	Time Start (hh:mm)	Time End (hh:mm)	Elapsed Time Δtime (min)	Water Drop During Standard Time Interval, Δd (in)	Soil Description Notes Comments
1	10:10 AM	10:40 AM	30	6.1	
2	10:43 AM	11:13 AM	30	5.5	
3	11:15 AM	11:45 AM	30	5.2	
4	11:50 AM	12:20 PM	30	4.8	
5	12:23 PM	12:53 PM	30	4.3	
6	12:55 PM	1:25 PM	30	4.2	Stabilized Readings
7	1:27 PM	1:57 PM	30	4.2	Achieved with Readings
8	2:05 AM	2:35 AM	30	4.1	6, 7, and 8

MEASURED PERCOLATION RATE & DESIGN INFILTRATION RATE CALCULATIONS*

* Calculations Below Based on Stabilized Readings Only

Boring Radius, r: 3.5 inches

Test Section Height, h: 42.0 inches

$$Test\ Section\ Surface\ Area, A = 2\pi rh + \pi r^2$$

A = **962** in^2

$$Discharged\ Water\ Volume, V = \pi r^2 \Delta d$$

$$Percolation\ Rate = \left(\frac{V/A}{\Delta T}\right)$$

Reading 6	V =	**162**	in^3	Percolation Rate =	**0.34** inches/hour
Reading 7	V =	**162**	in^3	Percolation Rate =	**0.34** inches/hour
Reading 8	V =	**157**	in^3	Percolation Rate =	**0.33** inches/hour

Measured Percolation Rate = **0.33** inches/hour

Reduction Factors

Small Diameter Boring, RF_t = 1

Site Variability, RF_v = 1

Long Term Siltation, RF_s = 1

$$Total\ Reduction\ Factor, RF = RF_t + RF_v + RF_s$$

Total Reduction Factor = 3

Design Infiltration Rate

$$Design\ Infiltration\ Rate = Measured\ Percolation\ Rate\ /RF$$

Design Infiltration Rate = **0.11** inches/hour

FIGURE 7

APPENDIX A

APPENDIX A
FIELD INVESTIGATION

The site was explored on October 9, 2024, by excavating seven 7-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine between the depths of approximately 5 and 37 feet below the existing ground surface. Additionally, the site was explored on November 22, 2024, by excavating one 30-inch-diameter boring using a bucket auger drilling machine to a maximum depth of approximately 36 feet. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the "undisturbed" soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch by $2\frac{3}{8}$-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 A8. 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 logs were revised based on subsequent laboratory testing. The approximate locations of the borings are shown on Figure 2.

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SOIL BORING NUMBER: B - 1

PROJECT NAME Glendora South Hills Bike Park
LOGGED BY RP
PROJECT NUMBER W1983-06-01
LATITUDE / LONGITUDE 34.120221, -117.863983
BORING DATE 10/09/2024 **FIGURE NUMBER** A1
DEPTH 5.5' **SURFACE ELEVATION** N/A
LOCATION South Hills Park
DRILLING FIRM Maritini Drilling
EQUIPMENT CME-75
METHOD Cal-Mod **BORING DIAMETER** 7 in
HAMMER TYPE Auto
HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
0			Fill	**ARTIFICIAL FILL** (**AC: 3" BASE: 12"**) **Silty SAND**, medium dense, dry, brown to dark brown, fine-grained, some medium-grained						
2						■	B1@2.5'	12 10 9	106.4	7.7
4			SM	**RESIDUAL SOIL** **Silty SAND**, loose to medium dense, slightly moist, reddish brown, fine-grained, some medium-grained		■	B1@5'	3 5 8	98.0	10.3
6				Total depth of boring: 5.5 feet Fill to 3 feet. No groundwater encountered. Percolation testing performed. Backfilled with soil cuttings and tamped.						
8										
10										
12										
14										
16										
18										

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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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

SOIL BORING NUMBER: B - 2

PROJECT NAME Glendora South Hills Bike Park
LOGGED BY RP
PROJECT NUMBER W1983-06-01
LATITUDE / LONGITUDE 34.120067, -117.86396
BORING DATE 10/09/2024
FIGURE NUMBER A2
DEPTH 5.5'
SURFACE ELEVATION N/A
LOCATION South Hills Park
DRILLING FIRM Maritini Drilling
EQUIPMENT CME-75
METHOD Cal-Mod
BORING DIAMETER 7 in
HAMMER TYPE Auto
HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
0			Fill	**ARTIFICIAL FILL** **Silty SAND**, medium dense, slightly moist, brown, fine-grained, some medium-grained						
2				increase in gravel		■	B2@2.5'	12 20 28	104.8	4.0
4										
5						■	B2@5'	18 50/4"	114.7	9.9
6				Total depth of boring: 5.5 feet All fill. No groundwater encountered. Backfilled with soil cuttings and tamped.						
8										
10										
12										
14										
16										
18										

DRAFT

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

SOIL BORING NUMBER: B - 3

PROJECT NAME Glendora South Hills Bike Park
LOGGED BY RP
PROJECT NUMBER W1983-06-01
LATITUDE / LONGITUDE 34.120308, -117.86364
BORING DATE 10/09/2024
FIGURE NUMBER A3
DEPTH 5.5'
SURFACE ELEVATION N/A
LOCATION South Hills Park
DRILLING FIRM Maritini Drilling
EQUIPMENT CME-75
METHOD Cal-Mod
BORING DIAMETER 7 in
HAMMER TYPE Auto
HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
0			Fill	**ARTIFICIAL FILL** **Silty SAND**, medium dense, dry to slightly moist, brown, fine-grained, some fine gravel			BULK 0-5'			
2			SM	**RESIDUAL SOIL** **Silty SAND**, medium dense, slightly moist, yellowish brown with oxidation mottles, fine- to medium-grained		■	B3@2.5'	13 15 20	114.5	6.4
4						■	B3@5'	13 17 20	112.5	10.6
6				Total depth of boring: 5.5 feet Fill to 2.5 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.						
8										
10										
12										
14										
16										
18										

DRAFT

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

SOIL BORING NUMBER: B - 4

PROJECT NAME Glendora South Hills Bike Park
LOGGED BY RP
PROJECT NUMBER W1983-06-01
LATITUDE / LONGITUDE 34.120708, -117.863095
BORING DATE 10/09/2024
FIGURE NUMBER A4
DEPTH 14.5'
SURFACE ELEVATION N/A
LOCATION South Hills Park
DRILLING FIRM Maritini Drilling
EQUIPMENT CME-75
METHOD Cal-Mod
BORING DIAMETER -
HAMMER TYPE Auto
HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
			SM	**ARTIFICIAL FILL** **Silty SAND**, medium dense, dry, brown, fine-grained			BULK 0-5'			
2						■	B4@2'	50/6"	90.8	8.4
				TOPANGA FORMATION **Silty SANDSTONE**, yellowish brown with gray mottling, fine- to medium-grained, poorly bedded, thick to massive, medium hard to hard, moderately to highly weathered						
4										
6				completely weathered, increase in silt		■	B4@6'	10 18 26	111.8	12.1
8				hard, localized laminated bedding, some oxidation staining		■	B4@8.5'	1 2 50/1"	102.0	13.3
10				decrease in oxidation staining		■	B4@11'	50/5"	102.5	6.2
12										
14				decrease in silt, grades to Sandstone		■	B4@14'	50/5"	92.4	9.0
				Total depth of boring: 14.5 feet Fill to 3 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.						
16										
18										

DRAFT

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

SOIL BORING NUMBER: B - 5

PROJECT NAME Glendora South Hills Bike Park
LOGGED BY RP
PROJECT NUMBER W1983-06-01
LATITUDE / LONGITUDE 34.120683, -117.863527
BORING DATE 10/09/2024 **FIGURE NUMBER** A5
DEPTH 14.5' **SURFACE ELEVATION** N/A
LOCATION South Hills Park
DRILLING FIRM Maritini Drilling
EQUIPMENT CME-75
METHOD Cal-Mod **BORING DIAMETER** -
HAMMER TYPE Auto
HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
2			Fill	**ARTIFICIAL FILL** **Silty SAND**, medium dense, dry to slightly moist, dark brown, fine-grained, some medium-grained, some clay						
4										
6			SM	**RESIDUAL SOIL** **Silty SAND**, medium dense, slightly moist, brown to reddish brown, fine- to medium-grained, trace Sandstone fragments		■	B5@5'	5 14 17	114.3	12.5
8				**TOPANGA FORMATION** **Silty SANDSTONE**, yellowish brown to reddish brown, fine- to medium-grained, poorly bedded, thick to massive, medium hard to hard, completely weathered, friable						
10						■	B5@9'	50/6"	103.4	7.3
12										
14				oxidized decrease in silt, some coarse-grained fragments		■	B5@14'	50/6"	107.6	13.0
16				Total depth of boring: 14.5 feet Fill to 5 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.						
18										

DRAFT

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. THE STRATIFICATION LINES PRESENTED HEREIN REPRESENT THE APPROXIMATE BOUNDARY BETWEEN EARTH TYPES; THE TRANSITIONS MAY BE GRADUAL.

GEOCON

SOIL BORING NUMBER: B - 6

PROJECT NAME Glendora South Hills Bike Park
LOGGED BY RP
PROJECT NUMBER W1983-06-01
LATITUDE / LONGITUDE 34.120405, -117.863374
BORING DATE 10/09/2024 **FIGURE NUMBER** A6
DEPTH 37' **SURFACE ELEVATION** N/A
LOCATION South Hills Park
DRILLING FIRM Maritini Drilling
EQUIPMENT CME-75
METHOD Cal-Mod **BORING DIAMETER** -
HAMMER TYPE Auto
HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
0			Fill	**ARTIFICIAL FILL** **Silty SAND**, medium dense, dry to slightly moist, brown, fine- to medium-grained, some fine gravel						
2				trace asphalt fragments						
4				**TOPANGA FORMATION**		■	B6@4.5'	15 50/6"	106.3	8.7
6				**Silty SANDSTONE**, yellowish brown, fine- to medium-grained, poorly bedded, massive, hard, completely weathered, friable						
8										
10				reddish brown, highly to completely weathered, oxidized, localized Siltstone interbeds		■	B6@9.5'	44 50/3"	110.9	17.6
12										
14				yellowish brown, increase in silt, increase in moisture		■	B6@14'	50/6"	97.5	10.0
16										
18										
						■	B6@19'	50/6"	98.7	9.2

DRAFT

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SOIL BORING: B-6

Depth (ft)	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
22							
24	olive to brown		■	B6@24.5'	41 50/5"	96.8	21.3
26							
28							
29	medium hard to very hard, moderately to highly weathered, localized laminated bedding		■	B6@29'	50/6"	102.8	10.9
30							
32							
34	some fresh fractured bedrock with granitic fragments		■	B6@34.5'	21 50/2"	93.2	15.4
36	olive brown to gray		■	B6@36'	24 40		
	refusal				50/3"		
38	Total depth of boring: 37 feet Fill to 4.5 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.						

DRAFT

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SOIL BORING NUMBER: B - 7

PROJECT NAME Glendora South Hills Bike Park
LOGGED BY RP
PROJECT NUMBER W1983-06-01
LATITUDE / LONGITUDE 34.120516, -117.863782
BORING DATE 10/09/2024
FIGURE NUMBER A7
DEPTH 15'
SURFACE ELEVATION N/A
LOCATION South Hills Park
DRILLING FIRM Maritini Drilling
EQUIPMENT CME-75
METHOD Cal-Mod
BORING DIAMETER -
HAMMER TYPE Auto
HAMMER WEIGHT / DROP 140 / 30

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number	Blow Counts/6"	Dry Density (pcf)	Moisture Content (%)
0			Fill	**ARTIFICIAL FILL** **Silty SAND**, loose to medium dense, dry to slightly moist, brown, fine-grained, some medium-grained						
2				cobble						
4				cobble						
6				brown with dark brown mottling		■	B7@5'	10 10 10	108.7	13.6
8				**TOPANGA FORMATION** **Silty SANDSTONE**, yellowish brown, fine- to medium-grained, poorly bedded, thick to massive, medium hard to hard, completely weathered						
10						■	B7@9.5'	18 50/4"	99.7	9.6
12										
14						■	B7@14.5'	36 50/5"	100.7	10.0
16				Total depth of boring: 15 feet Fill to 7 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.						
18										

DRAFT

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SOIL BORING NUMBER: B - 8

PROJECT NAME Glendora South Hills Bike Park
PROJECT NUMBER W1983-06-01
BORING DATE 11/22/2024 **FIGURE NUMBER** A8
LOCATION South Hills Park
DRILLING FIRM Dave's Drilling
METHOD Bucket Auger **BORING DIAMETER** 30 in

LOGGED BY RP
LATITUDE / LONGITUDE 34.120467, -117.863415
DEPTH 36' **SURFACE ELEVATION** N/A
EQUIPMENT Bucket Auger
HAMMER TYPE -
HAMMER WEIGHT / DROP - / -

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number
2			Fill	**ARTIFICIAL FILL** **Silty SAND**, medium dense, slightly moist, light brown to dark brown, fine- to coarse-grained, fine to coarse gravel, occasional cobbles, boulder (to 16")			
4				some cobble sized Clayey Sand inclusions **TOPANGA FORMATION** **SANDSTONE**, light orange and gray, fine- to medium-grained, hard, moderately to highly weathered, slightly moist, occasional coarse-grained 1.5" thick Claystone interbed, gray, bedding attitude [N70W, 10N]			
6				massive, moderately oxidized			
8				isolated fracture [N45E, 67W]			
10				medium- to coarse-grained, large mafic clast			
12				some dark brown to dark gray Sandstone inclusions (to 12" in diameter)			
14				steeply dipping fracture			
16				fracture cuts through borehole [N5E, 67W] - no apparent offset			
18				1.5' thick fracture zone with multiple fractures at about [N3E, 60-70W] - moist to very moist in fracture zone, cuts across borehole, no apparent offset 2" thick claystone interbed, bedding attitude [N70W, 9N], some fractures through clay bed, fractures [N45E, 90] [N15E, 74E], appx. 0.5-inch offset on factures (north side up) highly weathered, friable			
20							
22				variable gravel			
24				increase in moisture			
26				10" dark gray cemented inclusion			
28				large oxidation stains, manganese stringers			
				large unweathered clasts			

DRAFT

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GEOCON

SOIL BORING: B-8

Depth (ft)	Water Levels	Graphic Log	USCS	Material Description	Bulk	Driven	Sample Number
32							
34				end downhole log			
36				Total depth of boring: 36 feet Fill to 4 feet. No groundwater encountered. Backfilled with soil cuttings and tamped.			
38							
40							
42							
44							
46							
48							
50							
52							
54							
56							
58							

DRAFT

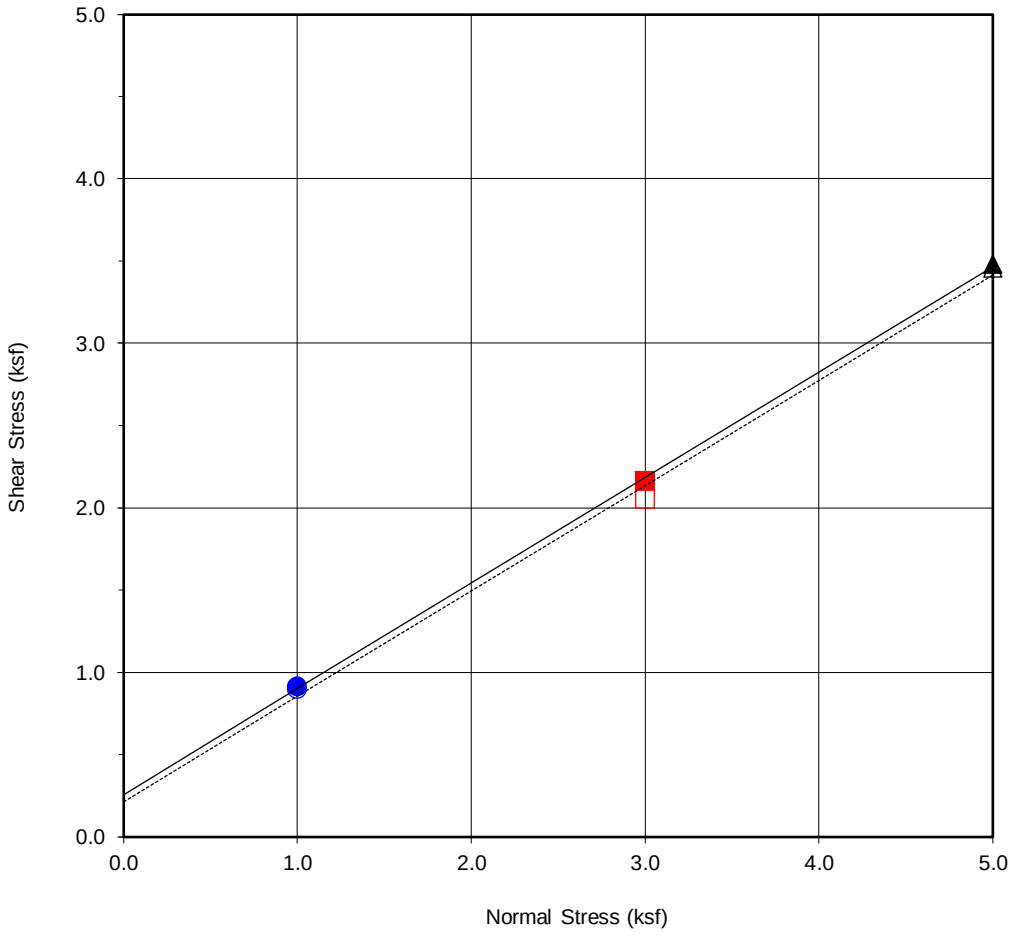
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APPENDIX B

APPENDIX B
LABORATORY TESTING

We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures We tested selected soil samples for in-place dry density/moisture content, expansion index, corrosivity, consolidation, and direct shear strength. The results of the laboratory tests are summarized in Figures B1 through B15.

DRAFT



Boring No.	**B4**
Sample No.	**B4@0-5**
Depth (ft)	**0-5**
Sample Type:	Ring

Soil Identification:		
Silty Sand (SM) - Remold		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	258	33
Ultimate	215	33

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.91	■ 2.16	▲ 3.48
Shear Stress @ End of Test (ksf)	○ 0.90	□ 2.05	△ 3.46
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	10.4	10.5	10.4
Initial Dry Density (pcf)	111.5	111.5	111.5
Initial Degree of Saturation (%)	54.9	55.2	55.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	17.3	16.7	15.9

GEOCON

DIRECT SHEAR TEST RESULTS

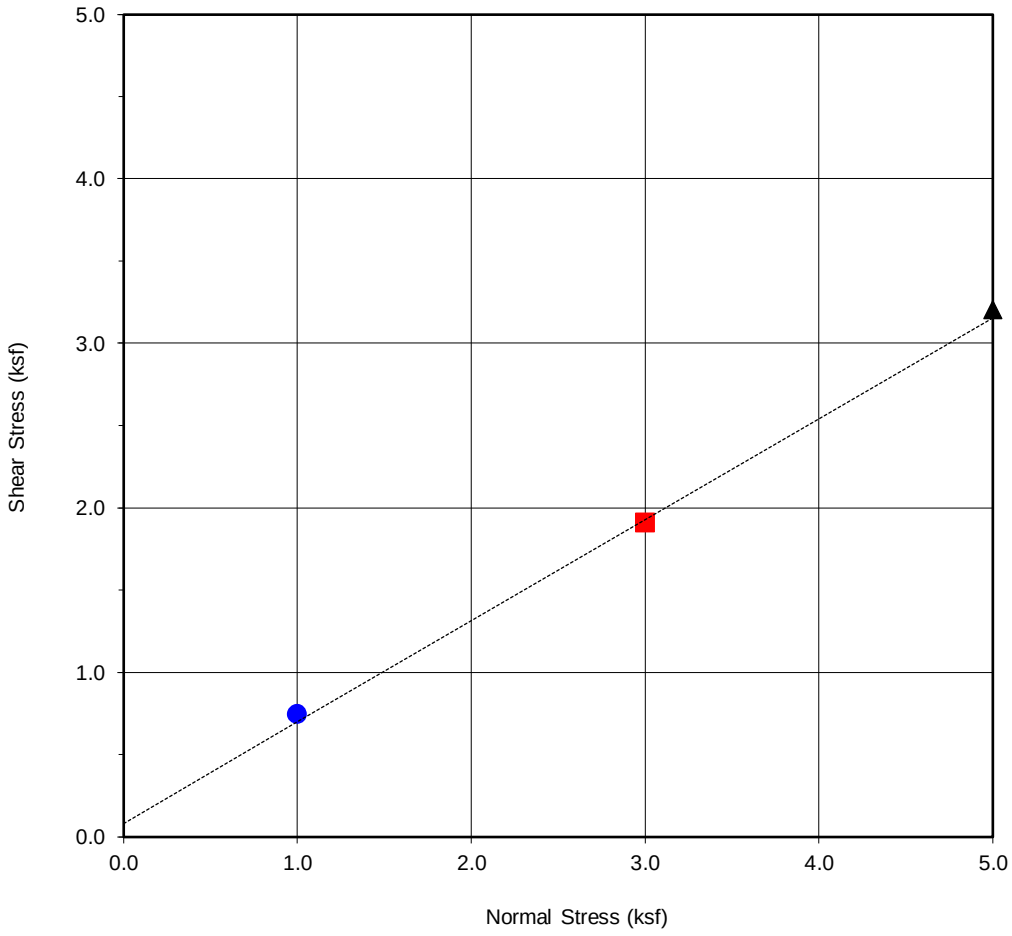
Consolidated Drained ASTM D-3080

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Project No.: W1983-06-01

GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

JAN. 2025 Figure B1



Boring No.	**B6**
Sample No.	**B6@9.5'**
Depth (ft)	**9.5'**
Sample Type:	Ring

Soil Identification:		
Topanga Formation Sandstone		
Strength Parameters		
	C (psf)	ϕ (^{o})
Residual	107	32

Normal Stress (kip/ft²)	1	3	5
*Pass No.	9	11	11
Peak Shear Stress (kip/ft²)	● 0.74	■ 1.91	▲ 3.20
Deformation Rate (in./min.)	0.00	0.00	0.00
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	17.7	19.2	14.8
Initial Dry Density (pcf)	109.8	107.0	114.6
Initial Degree of Saturation (%)	89.2	90.0	85.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	18.6	20.2	16.4

*Direct shear testing was performed on ring samples in consecutive passes in alternating directions. Data selected for residual strength values were taken from the pass number indicated.

GEOCON

DIRECT SHEAR TEST RESULTS

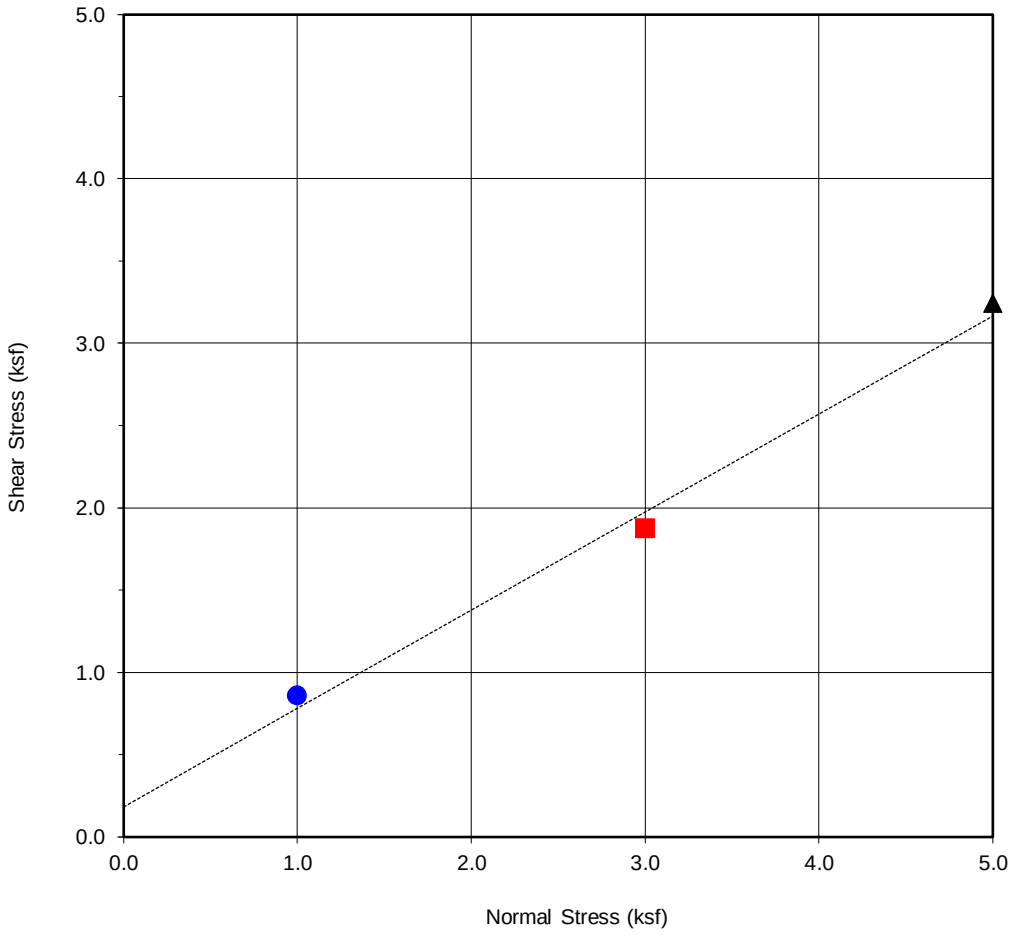
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Project No.: W1983-06-01

GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

JAN. 2025 Figure B2



Boring No.	B6
Sample No.	B6@24.5'
Depth (ft)	24.5'
Sample Type:	Ring

Soil Identification:		
Topanga Formation Sandstone		
Strength Parameters		
	C (psf)	ϕ (^{o})
Residual	203	31

Normal Stress (kip/ft²)	1	3	5
*Pass No.	11	9	11
Peak Shear Stress (kip/ft²)	● 0.86	■ 1.88	▲ 3.24
Deformation Rate (in./min.)	0.00	0.00	0.00
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	21.3	22.6	24.7
Initial Dry Density (pcf)	96.7	94.4	94.4
Initial Degree of Saturation (%)	77.5	77.7	85.1
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	23.1	23.0	23.0

*Direct shear testing was performed on ring samples in consecutive passes in alternating directions. Data selected for residual strength values were taken from the pass number indicated.

GEOCON

DIRECT SHEAR TEST RESULTS

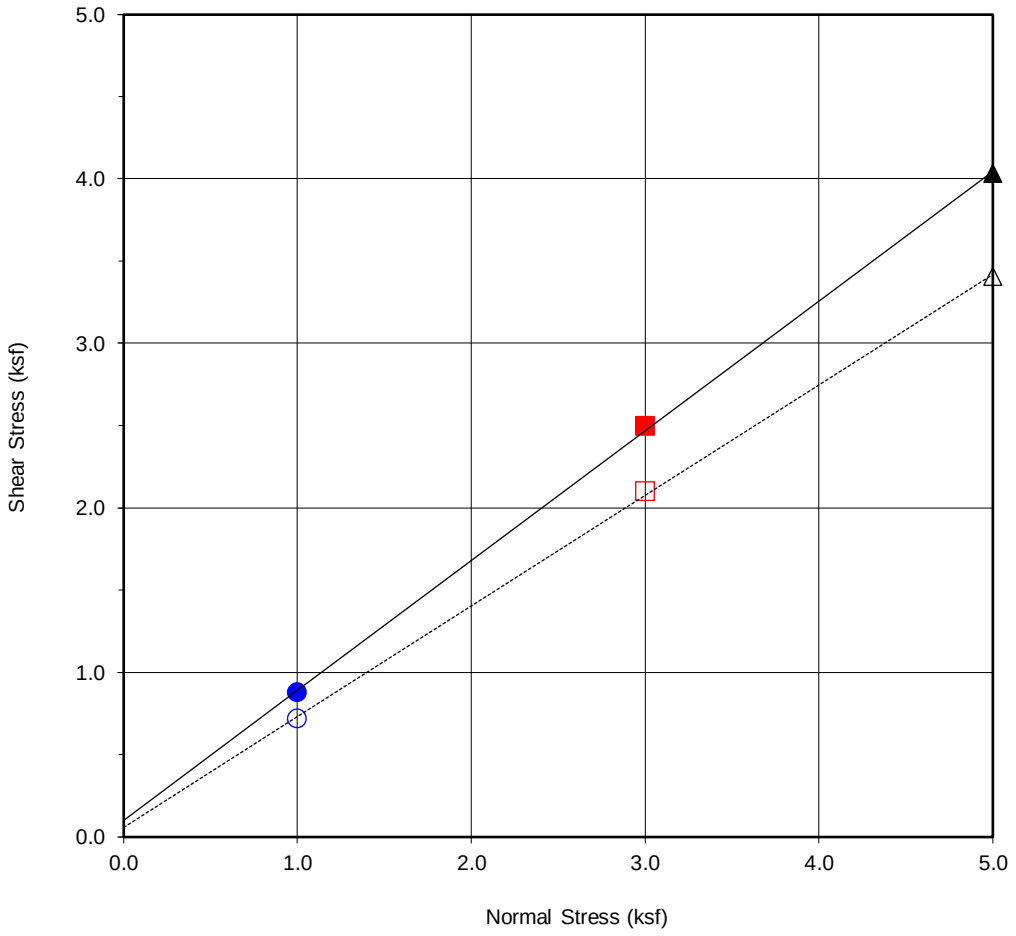
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GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

JAN. 2025 Figure B2



Boring No.	**B1**
Sample No.	**B1@2.5**
Depth (ft)	**2.5**
Sample Type:	Ring

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	101	38
Ultimate	60	34

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.88	■ 2.50	▲ 4.03
Shear Stress @ End of Test (ksf)	○ 0.72	□ 2.10	△ 3.41
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	7.6	7.4	7.6
Initial Dry Density (pcf)	105.2	105.8	108.4
Initial Degree of Saturation (%)	34.3	33.6	37.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	16.9	15.9	15.3

GEOCON

DIRECT SHEAR TEST RESULTS

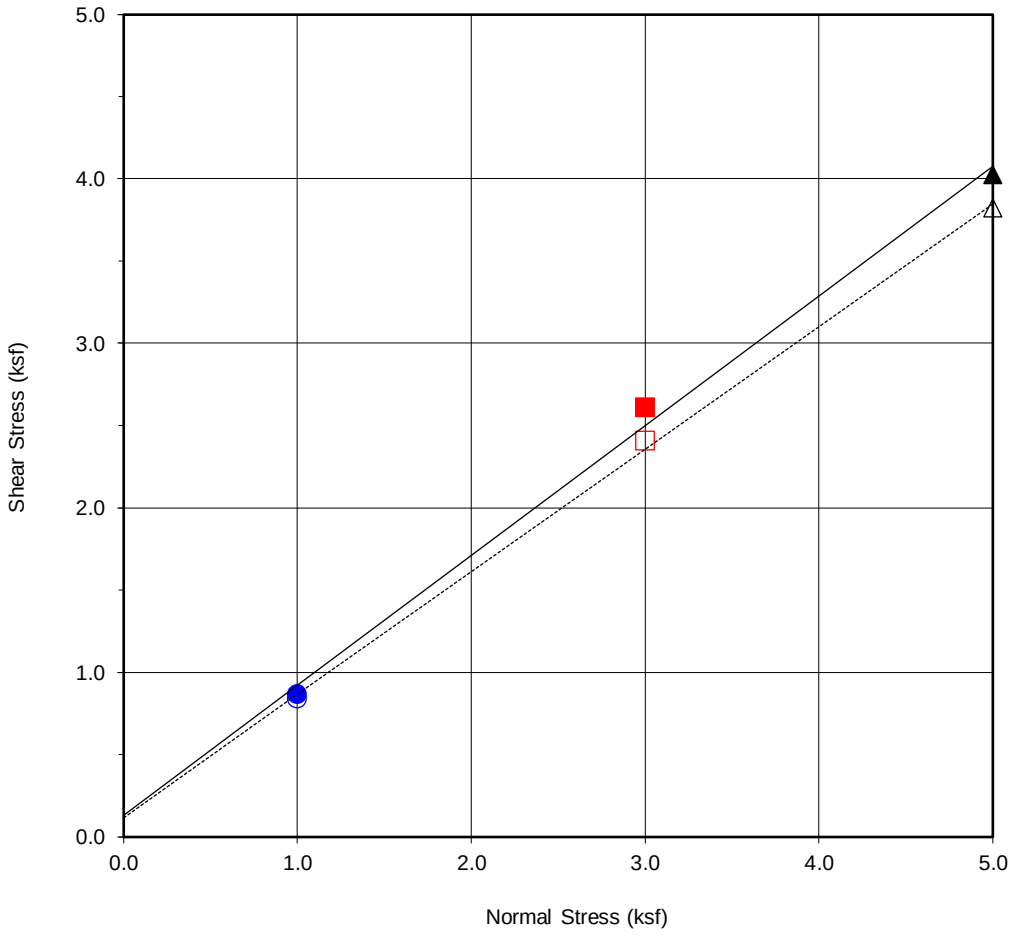
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GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

JAN. 2025 Figure B4



Boring No.	**B1**
Sample No.	**B1@5**
Depth (ft)	**5**
Sample Type:	Ring

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	133	38
Ultimate	119	37

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.87	■ 2.61	▲ 4.02
Shear Stress @ End of Test (ksf)	○ 0.84	□ 2.41	△ 3.82
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	10.3	10.3	9.6
Initial Dry Density (pcf)	93.5	95.1	101.2
Initial Degree of Saturation (%)	34.6	35.8	39.0
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.3	17.0	15.4

GEOCON

DIRECT SHEAR TEST RESULTS

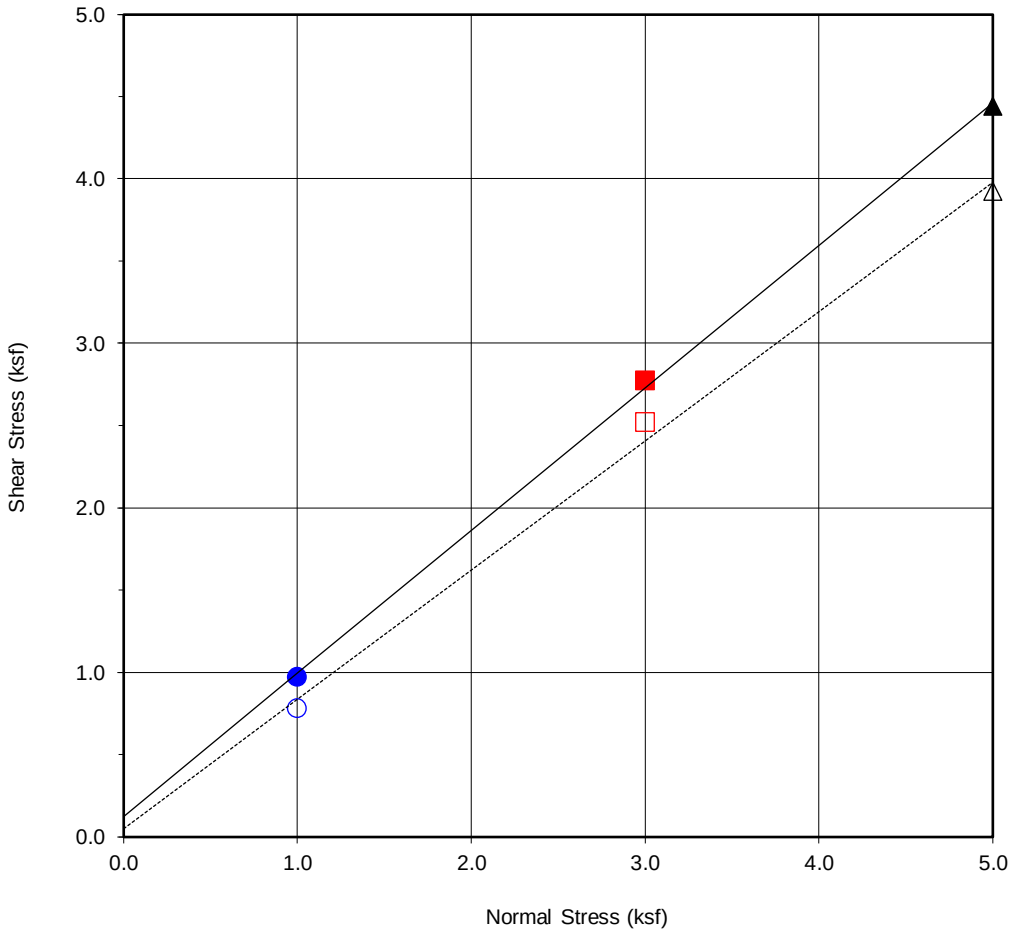
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GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

JAN. 2025 Figure B5



Boring No.	**B4**
Sample No.	**B4@2**
Depth (ft)	**2**
Sample Type:	Ring

Soil Identification:		
Silty Sand (SM)		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	127	41
Ultimate	50	38

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 0.97	■ 2.77	▲ 4.44
Shear Stress @ End of Test (ksf)	○ 0.78	□ 2.52	△ 3.92
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	8.9	8.7	9.0
Initial Dry Density (pcf)	101.6	104.0	102.8
Initial Degree of Saturation (%)	36.4	38.0	37.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	19.8	18.2	18.2

GEOCON

DIRECT SHEAR TEST RESULTS

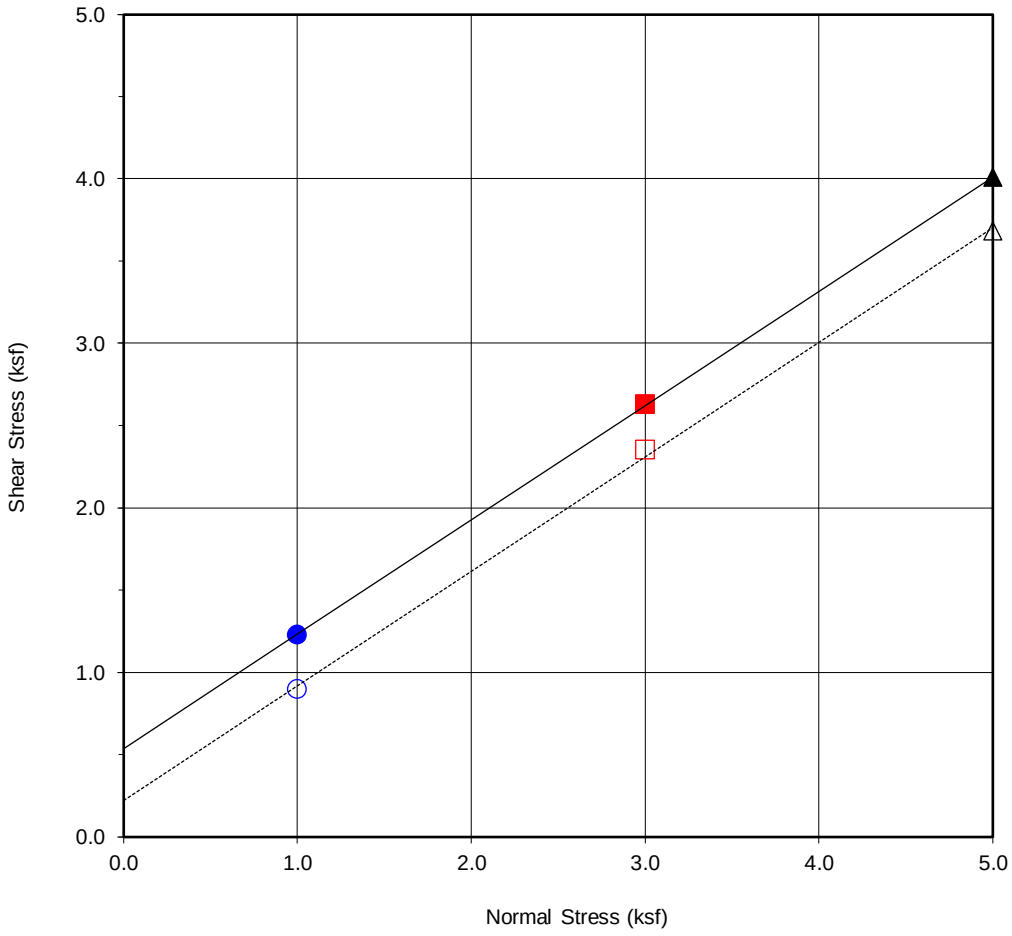
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Project No.: W1983-06-01

GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

JAN. 2025 Figure B6



Boring No.	B6
Sample No.	B6@14
Depth (ft)	14
Sample Type:	Ring

Soil Identification:
Topanga Formation Sandstone

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	537	35
Ultimate	221	35

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.23	■ 2.63	▲ 4.01
Shear Stress @ End of Test (ksf)	○ 0.90	□ 2.35	△ 3.68
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	10.0	13.3	9.1
Initial Dry Density (pcf)	97.7	94.0	98.2
Initial Degree of Saturation (%)	37.3	45.2	34.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	23.6	23.3	20.5

GEOCON

DIRECT SHEAR TEST RESULTS

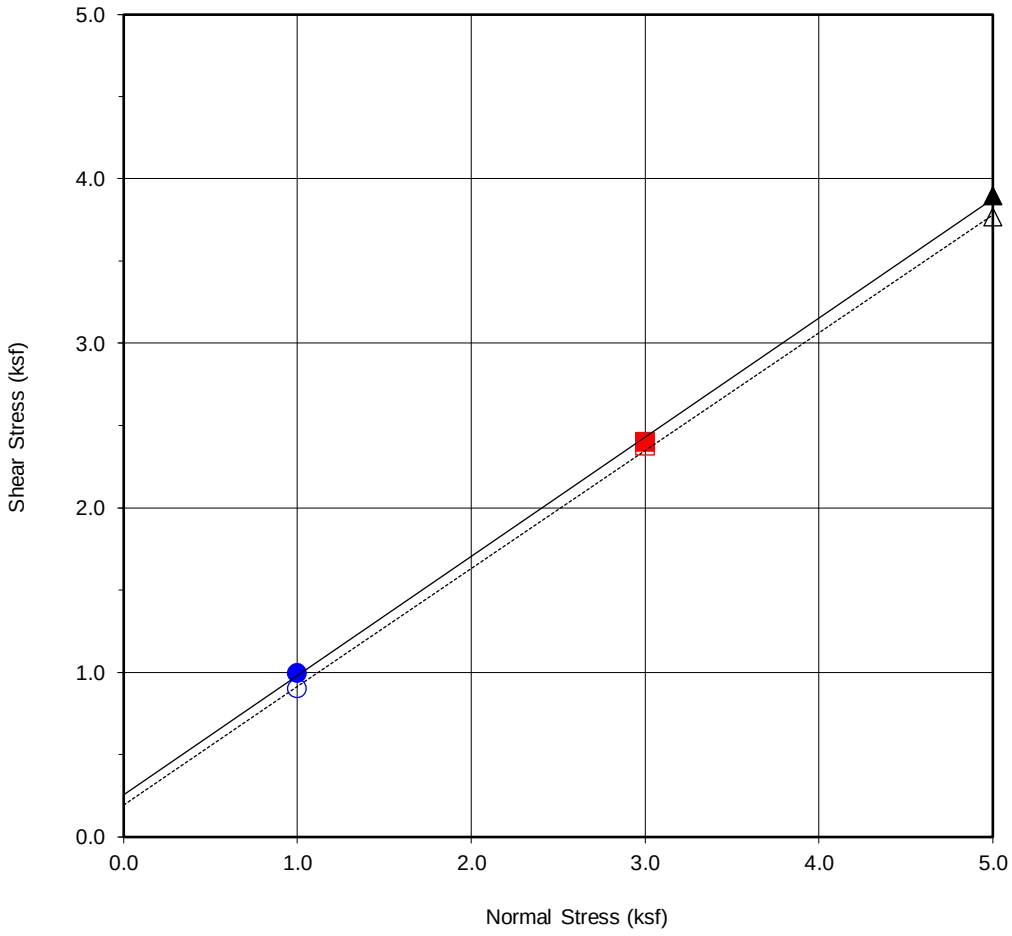
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Project No.: W1983-06-01

GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

JAN. 2025 Figure B7



Boring No.	B7
Sample No.	B7@5
Depth (ft)	5
Sample Type:	Ring

Soil Identification:
Silty Sand (SM)

Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	256	36
Ultimate	197	36

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.00	■ 2.40	▲ 3.90
Shear Stress @ End of Test (ksf)	○ 0.90	□ 2.38	△ 3.77
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	13.5	14.2	14.7
Initial Dry Density (pcf)	106.0	111.0	108.9
Initial Degree of Saturation (%)	62.0	73.7	72.4
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	21.1	19.0	19.1

GEOCON

DIRECT SHEAR TEST RESULTS

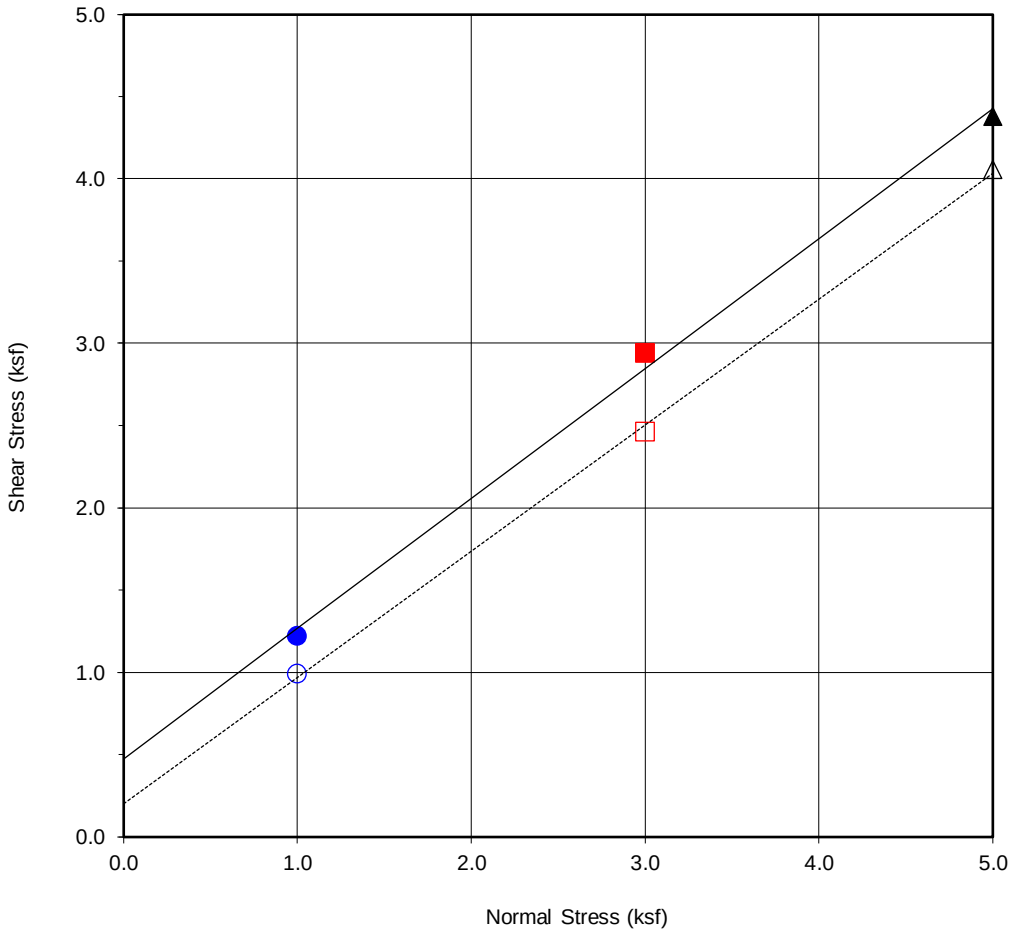
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Project No.: W1983-06-01

GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

JAN. 2025 Figure B8



Boring No.	**B7**
Sample No.	**B7@9.5**
Depth (ft)	**9.5**
Sample Type:	Ring

Soil Identification:		
Topanga Formation Sandstone		
Strength Parameters		
	C (psf)	ϕ (^{o})
Peak	477	38
Ultimate	202	37

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.22	■ 2.94	▲ 4.38
Shear Stress @ End of Test (ksf)	○ 0.99	□ 2.46	△ 4.06
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	9.6	9.0	8.5
Initial Dry Density (pcf)	101.0	101.4	100.2
Initial Degree of Saturation (%)	38.6	36.8	33.5
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	21.6	19.6	19.3

GEOCON

DIRECT SHEAR TEST RESULTS

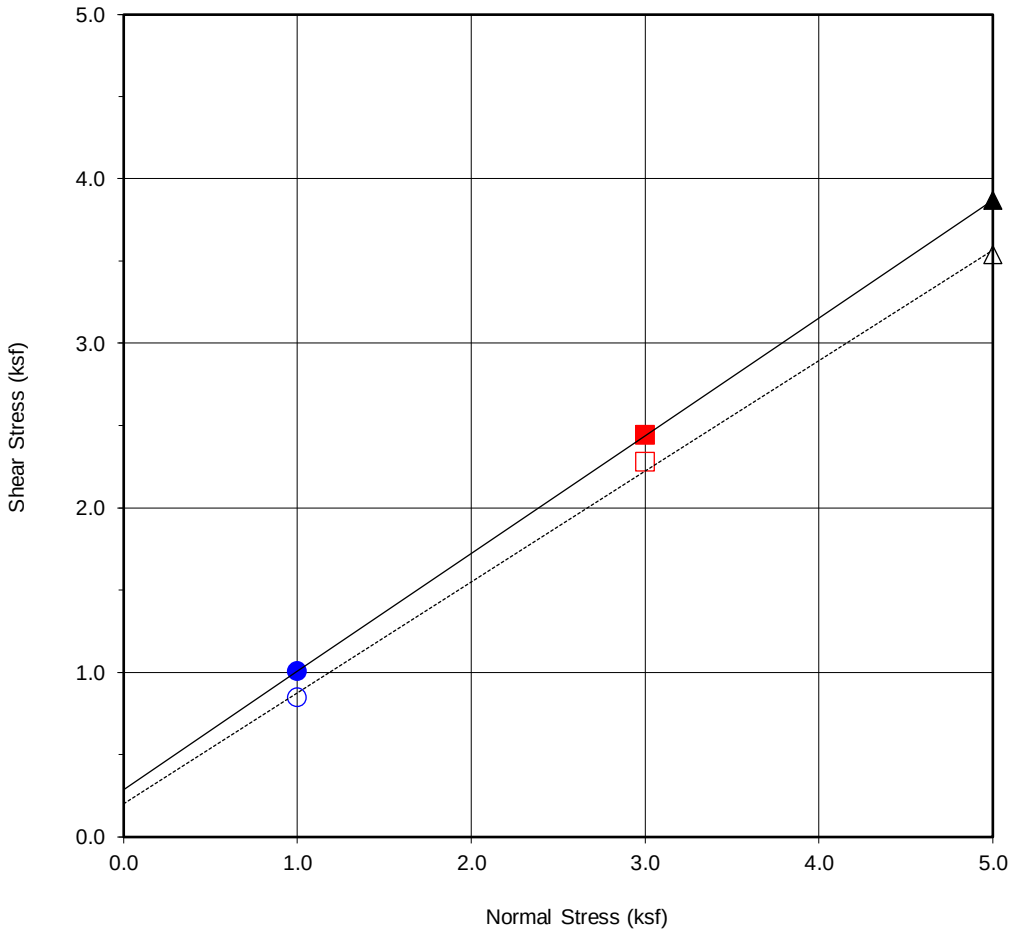
Consolidated Drained ASTM D-3080

Checked by: SY

Project No.: W1983-06-01

GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

JAN. 2025 Figure B9



Boring No.	B7
Sample No.	B7@14.5
Depth (ft)	14.5
Sample Type:	Ring

Soil Identification:

Topanga Formation Sandstone

Strength Parameters

	C (psf)	ϕ (^{o})
Peak	290	36
Ultimate	203	34

Normal Stress (kip/ft²)	1	3	5
Peak Shear Stress (kip/ft²)	● 1.01	■ 2.44	▲ 3.87
Shear Stress @ End of Test (ksf)	○ 0.85	□ 2.28	△ 3.54
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	10.0	7.8	8.0
Initial Dry Density (pcf)	101.2	101.2	104.7
Initial Degree of Saturation (%)	40.7	31.7	35.3
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	21.3	19.8	19.4

GEOCON

DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

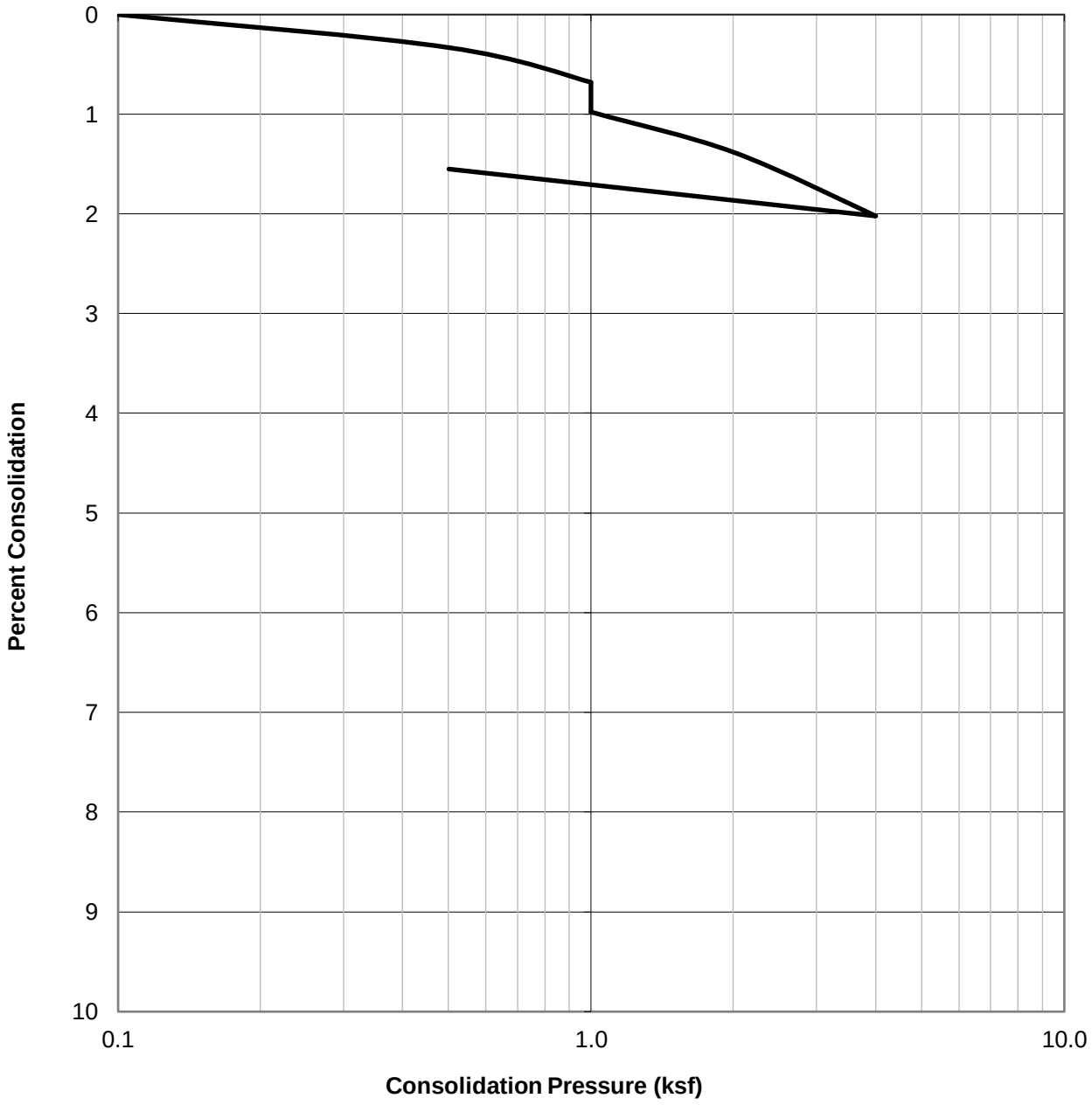
Checked by: SY

Project No.: W1983-06-01

GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

JAN. 2025 Figure B10

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B4@2	Silty Sand (SM)	100.9	8.5	18.0

GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: SY

Project No.: W1983-06-01

GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

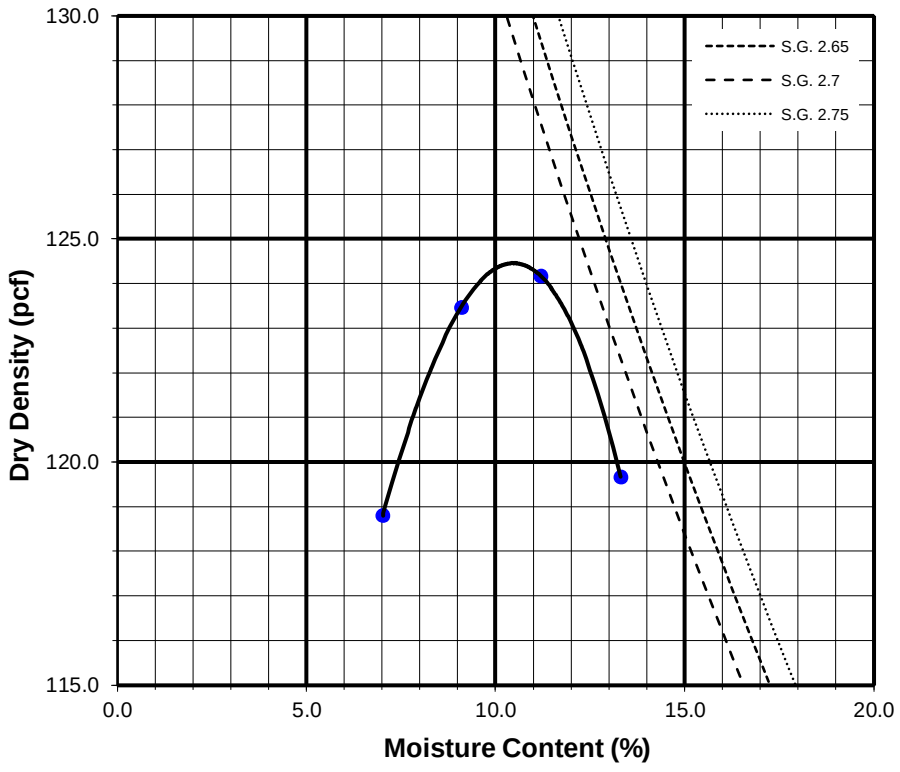
JAN. 2025 Figure B11

Sample No:

B3@ 0-5	Silty Sand (SM)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6120	6234	6285	6247		
Weight of Mold	(g)	4199	4199	4199	4199		
Net Weight of Soil	(g)	1921	2034	2086	2048		
Wet Weight of Soil + Cont.	(g)	716.1	769.3	778.1	693.1		
Dry Weight of Soil + Cont.	(g)	678.8	715.7	714.6	627.7		
Weight of Container	(g)	147.3	125.9	147.0	136.0		
Moisture Content	(%)	7.0	9.1	11.2	13.3		
Wet Density	(pcf)	127.1	134.7	138.1	135.6		
Dry Density	(pcf)	118.8	123.5	124.2	119.7		

Maximum Dry Density (pcf)	**125.0**	**Optimum Moisture Content (%)**	**10.5**



Preparation Method: B

GEOCON

COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS

ASTM D-1557

Checked by: SY

Project No.: W1983-06-01

GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

JAN. 2025 Figure B12

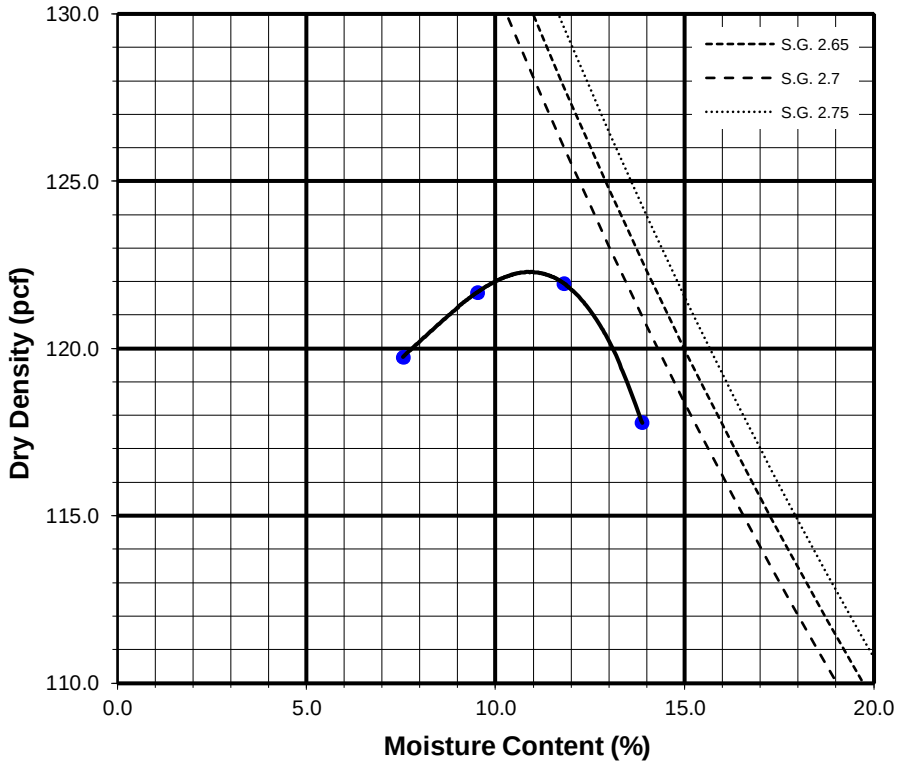
Sample No:

B4@ 0-5	Silty Sand (SM)

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6145	6212	6259	6225		
Weight of Mold	(g)	4199	4199	4199	4199		
Net Weight of Soil	(g)	1945	2013	2059	2026		
Wet Weight of Soil + Cont.	(g)	670.4	670.3	752.4	773.4		
Dry Weight of Soil + Cont.	(g)	632.7	624.6	688.5	697.1		
Weight of Container	(g)	133.2	144.7	147.1	146.9		
Moisture Content	(%)	7.5	9.5	11.8	13.9		
Wet Density	(pcf)	128.8	133.3	136.3	134.1		
Dry Density	(pcf)	119.7	121.7	121.9	117.8		

Maximum Dry Density (pcf)	**123.5**

Optimum Moisture Content (%)	**11.0**



Preparation Method: A

GEOCON

COMPACTION CHARACTERISTICS USING MODIFIED EFFORT TEST RESULTS

ASTM D-1557

Checked by: SY

Project No.: W1983-06-01

GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

JAN. 2025 Figure B13

B4 @ 0-5'

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	761.8	797.8
Wt. of Mold	(gm)	367.5	367.5
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	491.8	797.8
Dry Wt. of Soil + Cont.	(gm)	464.8	358.8
Wt. of Container	(gm)	191.8	367.5
Moisture Content	(%)	9.9	19.9
Wet Density	(pcf)	119.0	129.7
Dry Density	(pcf)	108.2	108.1
Void Ratio		0.6	0.6
Total Porosity		0.4	0.4
Pore Volume	(cc)	74.1	76.4
Degree of Saturation	(%) [S_{meas}]	48.3	93.7

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
10/31/2024	10:00	1.0	0	0.3
10/31/2024	10:10	1.0	10	0.299
Add Distilled Water to the Specimen				
11/1/2024	10:00	1.0	1430	0.31
11/1/2024	11:00	1.0	1490	0.31

Expansion Index (EI meas) =	11
Expansion Index (Report) =	**11**

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: 2022 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: SY

Project No.: W1983-06-01

GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

JAN. 2025 Figure B14

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

Sample No.	pH	Resistivity (ohm centimeters)
B4 @ 0-5'	6.6	3100 (Moderately Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
AASHTO T291 ASTM C1218

Sample No.	Chloride Ion Content (%)
B4 @ 0-5'	0.004

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

Sample No.	Water Soluble Sulfate (% SO_4)	Sulfate Exposure
B4 @ 0-5'	0.002	S0

GEOCON

CORROSIVITY TEST RESULTS

Checked by: SY

Project No.: W1983-06-01

GLENDORA FAMILY BIKE PARK
SOUTH HILLS PARK
GLENDORA, CALIFORNIA

JAN. 2025 Figure B15

APPENDIX C



W1983-06-01 Slide2 Model 2025-01-09
Glendora Bike Park
Geocon West, Inc.
Date Created: 1/7/2025, 9:37:28 AM
Software Version: 9.037

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Slide2 Analysis Information

W1983-06-01 Slide2 Model 2025-01-09

Project Summary

File Name:	W1983-06-01 Slide2 Model 2025-01-09.slmd
Last saved with Slide2 version:	9.037
Project Title:	Glendora Bike Park
Analysis:	Section A-A' - Sencario 1 (Non-Circular)
Author:	JMH
Company:	Geocon West, Inc.
Date Created:	1/7/2025, 9:37:28 AM

Currently Open Scenarios

Group Name	Scenario Name	Compute Time
Section A-A'	Master Scenario	
	Non-Circular Cuckoo - Static	
	Non-Circular Cuckoo - Seismic	
Section B-B'	Master Scenario	
	Non-Circular Cuckoo - Static	
	Non-Circular Cuckoo - Seismic	
Section A-A' Remedial Grading	Master Scenario	
	Non-Circular Cuckoo - Static	
	Non-Circular Cuckoo - Seismic	

General Settings

Units of Measurement:	Imperial Units
Time Units:	days
Permeability Units:	feet/second
Data Output:	Standard
Failure Direction:	Right to Left

Analysis Options

Section A-A' - Master Scenario

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Section A-A' - Non-Circular Cuckoo - Static

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
	Spencer
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Section A-A' - Non-Circular Cuckoo - Seismic

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
	Spencer
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Section B-B'

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
	Spencer
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Section A-A' Remedial Grading - Master Scenario

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Section A-A' Remedial Grading - Non-Circular Cuckoo - Static

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
	Spencer
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Section A-A' Remedial Grading - Non-Circular Cuckoo - Seismic

Slices Type:	Vertical
Analysis Methods Used	
	Bishop simplified
	Spencer
Number of slices:	50
Tolerance:	0.005
Maximum number of iterations:	75
Check malpha < 0.2:	Yes
Create Interslice boundaries at intersections with water tables and piezos:	Yes
Initial trial value of FS:	1
Steffensen Iteration:	Yes

Random Numbers

All Open Scenarios

Pseudo-random Seed:	10116
Random Number Generation Method:	Park and Miller v.3

Surface Options

Section A-A' - Master Scenario

Surface Type:	Circular
Search Method:	Grid Search
Radius Increment:	10
Composite Surfaces:	Disabled
Reverse Curvature:	Invalid Surfaces
Minimum Elevation:	Not Defined
Minimum Depth:	Not Defined
Minimum Area:	Not Defined
Minimum Weight:	Not Defined

Section A-A' - Non-Circular Cuckoo - Static

Search Method:	Cuckoo Search
Initial # of Surface Vertices:	8
Maximum Iterations:	500
Number of Nests:	50
Minimum Elevation:	Not Defined
Minimum Depth:	Not Defined
Minimum Area:	Not Defined
Minimum Weight:	Not Defined
Convex Surfaces Only:	Enabled
Optimize Surfaces:	Enabled

Section A-A' - Non-Circular Cuckoo - Seismic

Search Method:	Cuckoo Search
Initial # of Surface Vertices:	8
Maximum Iterations:	500
Number of Nests:	50
Minimum Elevation:	Not Defined
Minimum Depth:	Not Defined
Minimum Area:	Not Defined
Minimum Weight:	Not Defined
Convex Surfaces Only:	Enabled
Optimize Surfaces:	Enabled

Section B-B' - Master Scenario

Surface Type:	Circular
Search Method:	Grid Search
Radius Increment:	10
Composite Surfaces:	Disabled
Reverse Curvature:	Create Tension Crack
Minimum Elevation:	Not Defined
Minimum Depth:	Not Defined
Minimum Area:	Not Defined
Minimum Weight:	Not Defined

Section B-B' - Non-Circular Cuckoo - Static

Search Method:	Cuckoo Search
Initial # of Surface Vertices:	8
Maximum Iterations:	500
Number of Nests:	50
Minimum Elevation:	Not Defined
Minimum Depth:	Not Defined
Minimum Area:	Not Defined
Minimum Weight:	Not Defined
Convex Surfaces Only:	Enabled
Optimize Surfaces:	Enabled

Section B-B' - Non-Circular Cuckoo - Seismic

Search Method:	Cuckoo Search
Initial # of Surface Vertices:	8
Maximum Iterations:	500
Number of Nests:	50
Minimum Elevation:	Not Defined
Minimum Depth:	Not Defined
Minimum Area:	Not Defined
Minimum Weight:	Not Defined
Convex Surfaces Only:	Enabled
Optimize Surfaces:	Enabled

Section A-A' Remedial Grading - Master Scenario

Surface Type:	Circular
Search Method:	Grid Search
Radius Increment:	10
Composite Surfaces:	Disabled
Reverse Curvature:	Invalid Surfaces
Minimum Elevation:	Not Defined
Minimum Depth:	Not Defined
Minimum Area:	Not Defined
Minimum Weight:	Not Defined

Section A-A' Remedial Grading - Non-Circular Cuckoo - Static

Search Method:	Cuckoo Search
Initial # of Surface Vertices:	8
Maximum Iterations:	500
Number of Nests:	50
Minimum Elevation:	Not Defined
Minimum Depth:	Not Defined
Minimum Area:	Not Defined
Minimum Weight:	Not Defined
Convex Surfaces Only:	Enabled
Optimize Surfaces:	Enabled

Section A-A' Remedial Grading - Non-Circular Cuckoo - Seismic

Search Method:	Cuckoo Search
Initial # of Surface Vertices:	8
Maximum Iterations:	500
Number of Nests:	50
Minimum Elevation:	Not Defined
Minimum Depth:	Not Defined
Minimum Area:	Not Defined
Minimum Weight:	Not Defined
Convex Surfaces Only:	Enabled
Optimize Surfaces:	Enabled

Seismic Loading

Section A-A' - Master Scenario

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No

Section A-A' - Non-Circular Cuckoo - Static

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No

Section A-A' - Non-Circular Cuckoo - Seismic

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No
Seismic Load Coefficient (Horizontal):	0.275

Section B-B' - Master Scenario

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No

Section B-B' - Non-Circular Cuckoo - Static

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No

Section B-B' - Non-Circular Cuckoo - Seismic

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No
Seismic Load Coefficient (Horizontal):	0.275

Section A-A' Remedial Grading - Master Scenario

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No

Section A-A' Remedial Grading - Non-Circular Cuckoo - Static

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No

Section A-A' Remedial Grading - Non-Circular Cuckoo - Seismic

Advanced seismic analysis:	No
Staged pseudostatic analysis:	No
Seismic Load Coefficient (Horizontal):	0.275

Materials

Artificial Fill	
Color	
Strength Type	Mohr-Coulomb
Unit Weight	120 lbs/ft3
Cohesion	60 psf
Phi	34 °
Water Surface	Assigned per scenario
Ru Value	0

Topanga Formation (Anisotropic)	
Color	
Strength Type	Anisotropic Function
Unit Weight	125 lbs/ft3
Anisotropic Function	Topanga
Water Surface	Assigned per scenario
Ru Value	0

Topanga Formation (Isotropic)	
Color	
Strength Type	Mohr-Coulomb
Unit Weight	125 lbs/ft3
Cohesion	200 psf
Phi	36 °
Water Surface	Assigned per scenario
Ru Value	0

Glendora Volcanics	
Color	
Strength Type	Mohr-Coulomb
Unit Weight	130 lbs/ft3
Cohesion	350 psf
Phi	36 °
Water Surface	Assigned per scenario
Ru Value	0

Anisotropic Functions

Name: Topanga

Angle From	Angle To	c	phi
-90	9	200	36
9	15	100	31
15	90	200	36

Materials In Use

Material	Section A-A'	Non-Circular Cuckoo - Static	Non-Circular Cuckoo - Seismic	Section B-B'	Non-Circular Cuckoo - Static	Non-Circular Cuckoo - Seismic	Section A-A' Remedial Grading	Non-Circular Cuckoo - Static	Non-Circular Cuckoo - Seismic
Artificial Fi	✓	✓	✓	✓	✓	✓	✓	✓	✓
Topanga Formation (Anisotrop	✓	✓	✓	✗	✗	✗	✓	✓	✓
Topanga Formation (Isotropic)	✗	✗	✗	✓	✓	✓	✗	✗	✗
Glendora Volcanics	✓	✓	✓	✓	✓	✓	✓	✓	✓

Entity Information

Section A-A'

Shared Entities

Type	Coordinates (x,y)
External Boundary	-0.362579, -24.745 358.252, -24.745 654.379, -24.745 654.379, 171.068 629.365, 165.301 614.238, 154.935 605.12, 154.935 596.791, 155.948 589.53, 151.35 586.885, 149.268 578.949, 141.558 576.248, 137.843 572.186, 132.504 567.035, 128.447 562.339, 125.894 555.364, 124.071 545.462, 121.772 541.128, 120.928 504.387, 109.178 501.224, 108.822 472.852, 102.807 435.881, 96.027 430.593, 96.027 421.328, 89.924 420.143, 87.234 416.04, 82.675 409.384, 75.518 403.909, 71.441 389.876, 65.594 371.977, 64.657 220.988, 61.123 198.854, 56.252 175.555, 53.286 111.327, 45.674 7.329, 0 0, 0
Material Boundary	175.555, 53.286 193.351, 53.286 212.133, 53.811 249.563, 54.252 269.215, 56.418 287.281, 57.994 318.148, 61.056 359.143, 59.875 362.894, 59.875 371.608, 62.835 371.977, 64.657
Material Boundary	358.252, -24.745 401.818, 2.92 443.641, 32.443 459.15, 44.338 487.521, 65.911 512.448, 88.815 531.278, 107.367 545.462, 121.772

Scenario-based Entities

Type	Coordinates (x,y)	Master Scenario	Non-Circular Cuckoo - Static	Non-Circular Cuckoo - Seismic
Focus Search Window	431.598, 75.717 475.49, 84.2488 473.428, 94.8592 429.657, 86.3511			

Section B-B'

Shared Entities

Type	Coordinates (x,y)

External Boundary	-0.302, -0.218 260.678, -0.221117 502.047, -0.224 502.047, 159.681 493.476, 157.164 492.048, 155.939 456.201, 147.491 453.244, 147.32 451.644, 146.81 448.685, 146.81 432.67, 142.58 417.451, 137.595 412.842, 137.291 402.487, 132.961 400.39, 130.819 374.321, 121.799 343.079, 112.817 338.671, 110.889 315.364, 101.246 301.311, 97.477 287.34, 89.408 285.378, 89.408 282.199, 88 269.54, 88 267.334, 88.525 263.365, 88.525 261.213, 89.332 258.869, 88.582 251.396, 88.582 247.374, 88.024 237.5, 88.655 215.604, 87.603 209.2, 87.603 191.348, 89.096 189.589, 88.348 175.576, 88.348 152.363, 86.023 149.759, 85.221 146.416, 85.221 133.356, 82.299 129.113, 82.299 114.677, 78.607 111.04, 76.789 103.035, 74.8 98.866, 73.551 90.898, 69.694 89.806, 69.694 80.315, 64.827 57.136, 52.734 46.974, 46.099 34.125, 37.701 12.329, 23.571 -0.302, 23.571

Material Boundary	98.866, 73.551 128.319, 73.719 135.939, 74.178 145.356, 74.847 150.071, 75.614 162.344, 78.024 166.45, 78.024 172.52, 78.56 177.876, 78.56 180.474, 79.34 185.906, 79.34 189.393, 80.574 201.94, 80.574 208.676, 80.906 213.225, 81.552 219.286, 82.379 231.903, 82.379 246.725, 82.379 267.112, 83.095 274.441, 83.866 280.006, 84.858 284.303, 86.732 287.34, 89.408
Material Boundary	260.678, -0.221117 280.006, 12.921 292.934, 21.113 327.632, 44.769 338.896, 52.577 352.848, 62.177 360.656, 67.809 382.416, 85.217 405.453, 104.679 426.616, 123.407 445.178, 144.941 448.685, 146.81

Section A-A' Remedial Grading

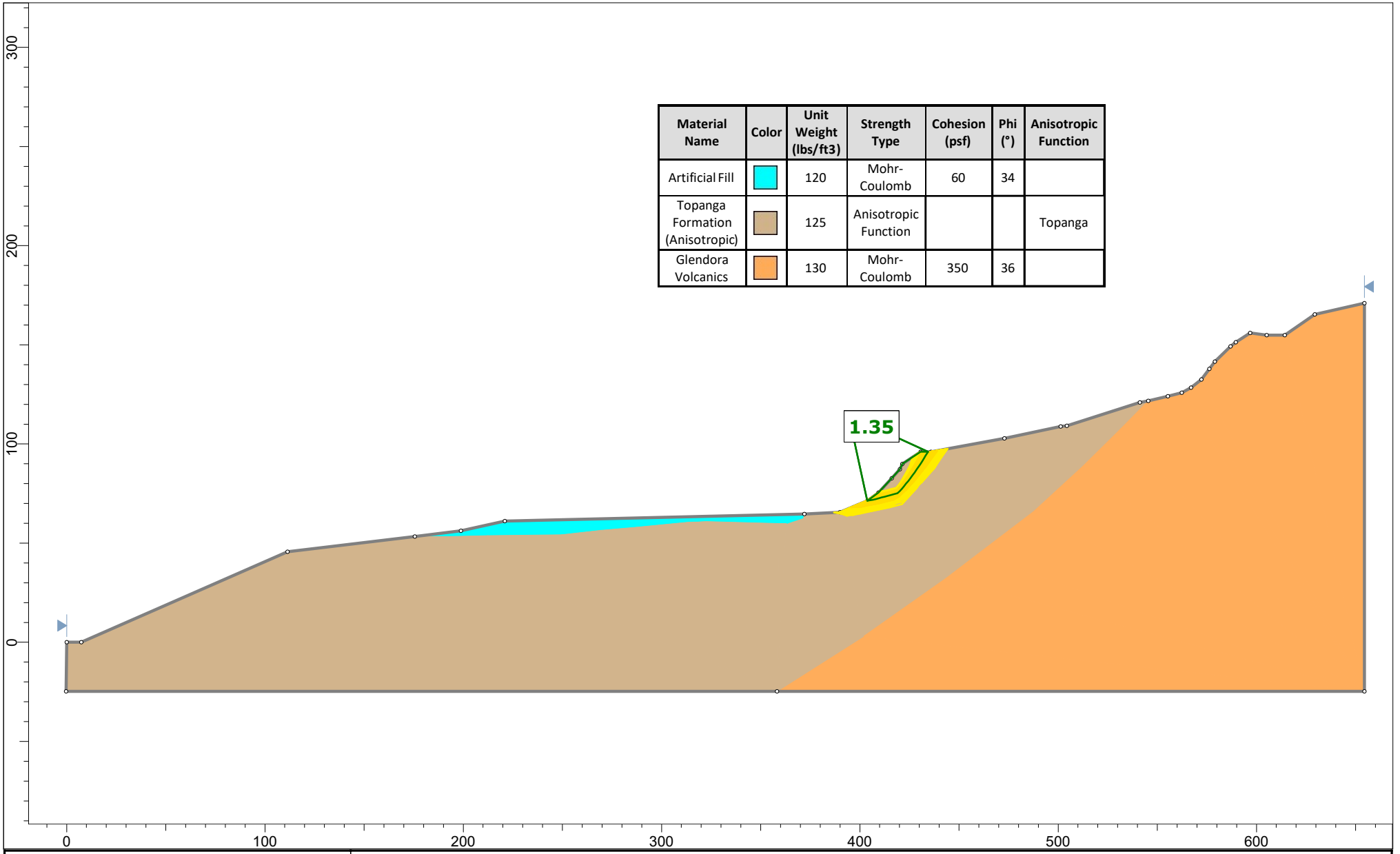
Shared Entities

Type	Coordinates (x,y)
External Boundary	-0.362579, -24.745 358.252, -24.745 654.379, -24.745 654.379, 171.068 629.365, 165.301 614.238, 154.935 605.12, 154.935 596.791, 155.948 589.53, 151.35 586.885, 149.268 578.949, 141.558 576.248, 137.843 572.186, 132.504 567.035, 128.447 562.339, 125.894 555.364, 124.071 545.462, 121.772 541.128, 120.928 504.387, 109.178 501.224, 108.822 472.852, 102.807 459.077, 100.293 389.876, 65.594 371.977, 64.657 220.988, 61.123 198.854, 56.252 175.555, 53.286 111.327, 45.674 7.329, 0 0, 0
Material Boundary	175.555, 53.286 193.351, 53.286 212.133, 53.811 249.563, 54.252 269.215, 56.418 287.281, 57.994 318.148, 61.056 359.143, 59.875 362.894, 59.875 371.608, 62.835 371.977, 64.657
Material Boundary	358.252, -24.745 401.818, 2.92 443.641, 32.443 459.15, 44.338 487.521, 65.911 512.448, 88.815 531.278, 107.367 545.462, 121.772

Scenario-based Entities

Type	Coordinates (x,y)	Master Scenario	Non-Circular Cuckoo - Static	Non-Circular Cuckoo - Seismic
Focus Search Window	431.598, 75.717 475.49, 84.2488 473.428, 94.8592 429.657, 86.3511			

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Anisotropic Function
Artificial Fill		120	Mohr-Coulomb	60	34	
Topanga Formation (Anisotropic)		125	Anisotropic Function			Topanga
Glendora Volcanics		130	Mohr-Coulomb	350	36	

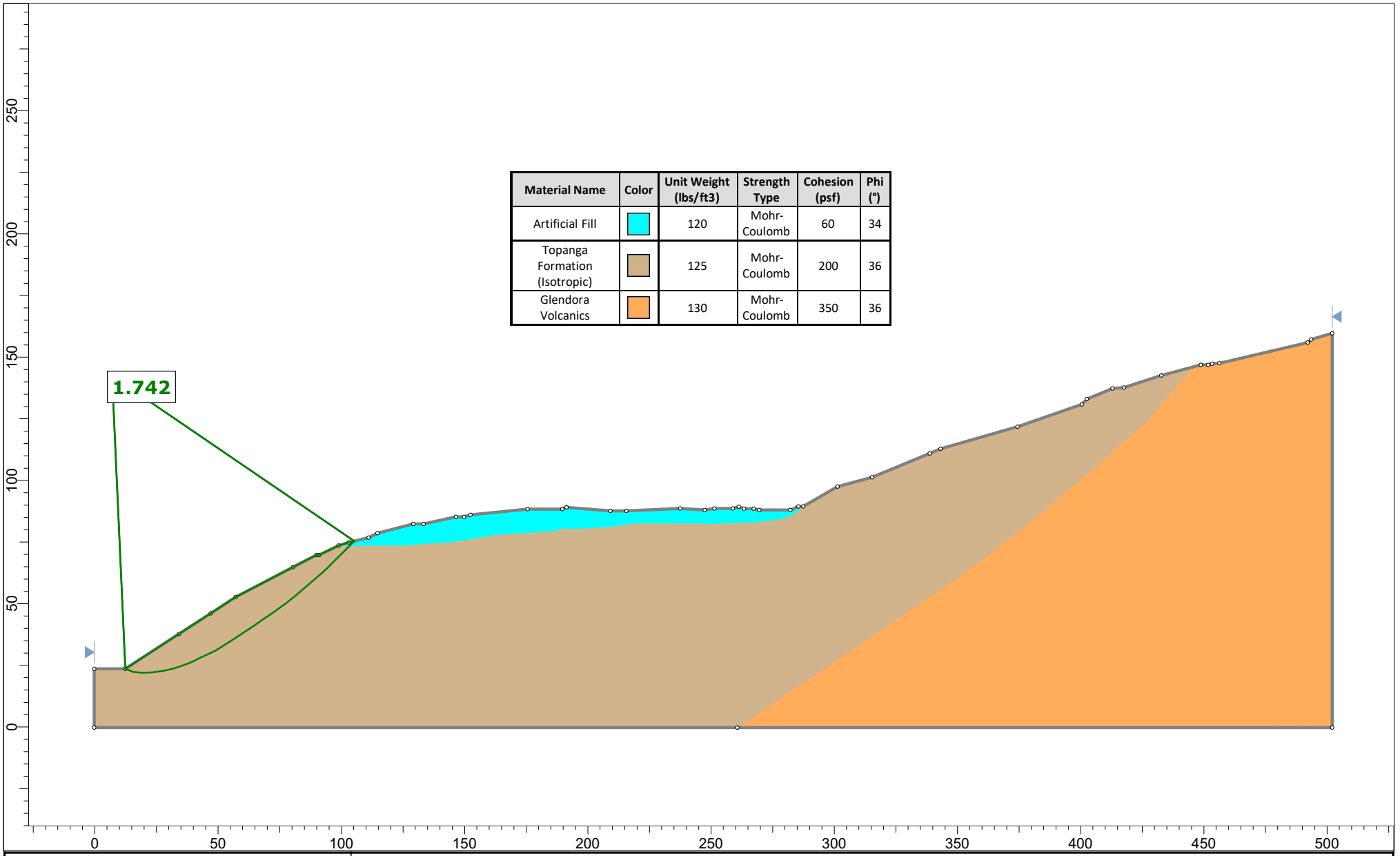


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SLIDEINTERPRET 9.037

Project	Glendora Bike Park		
Group	Section A-A'	*Scenario*	Non-Circular Cuckoo - Static
Drawn By	JMH	*Company*	Geocon West, Inc.
Date	1/7/2025, 9:37:28 AM	*File Name*	W1983-06-01 Slide2 Model 2025-01-09.slmd

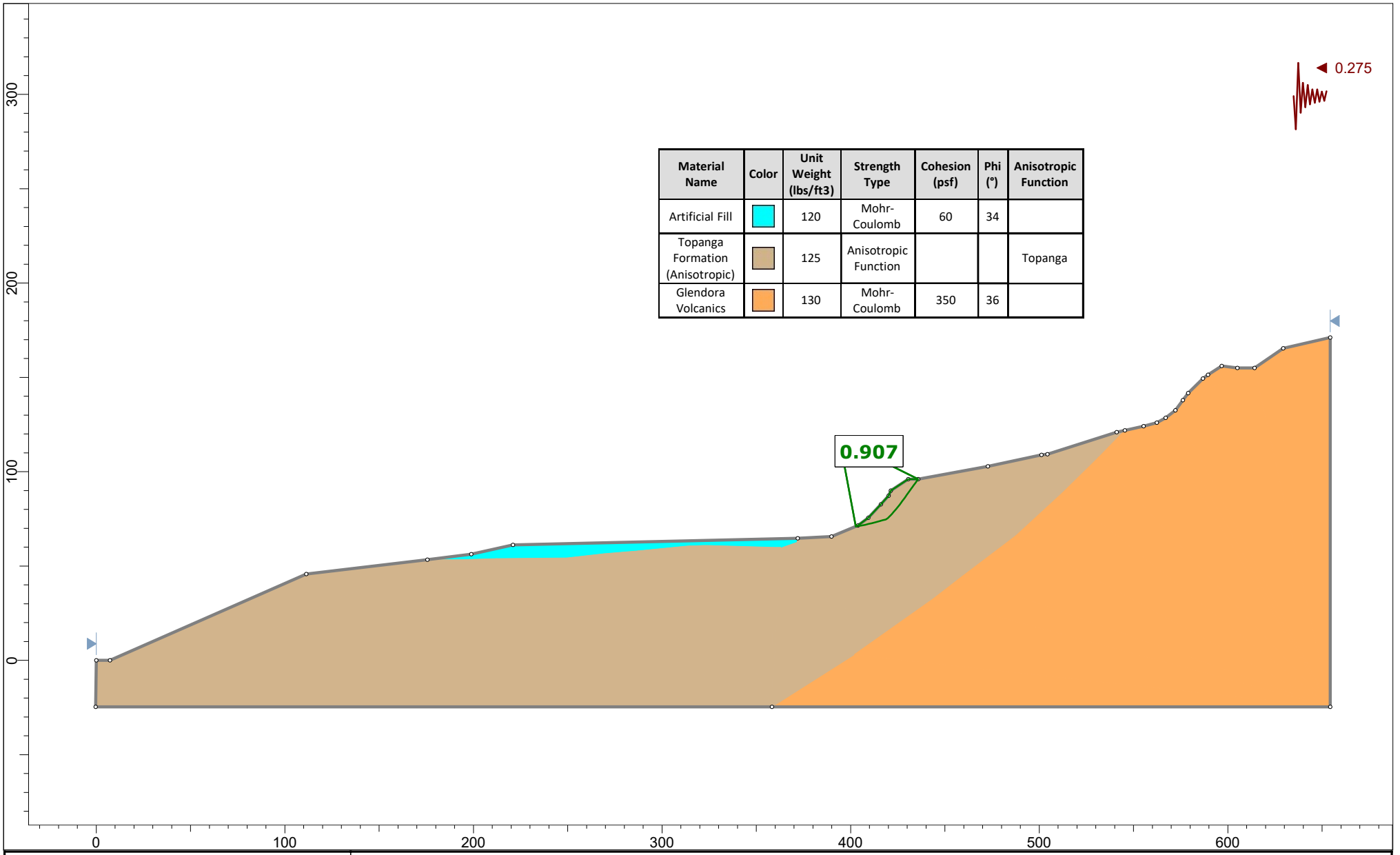
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Artificial Fill		120	Mohr-Coulomb	60	34
Topanga Formation (Isotropic)		125	Mohr-Coulomb	200	36
Glendora Volcanics		130	Mohr-Coulomb	350	36



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SLIDEINTERPRET 9.037

Project	Glendora Bike Park		
Group	Section B-B'	Scenario	Non-Circular Cuckoo - Static
Drawn By	JMH	Company	Geocon West, Inc.
Date	1/7/2025, 9:37:28 AM	File Name	W1983-06-01 Slide2 Model 2025-01-09.slmd

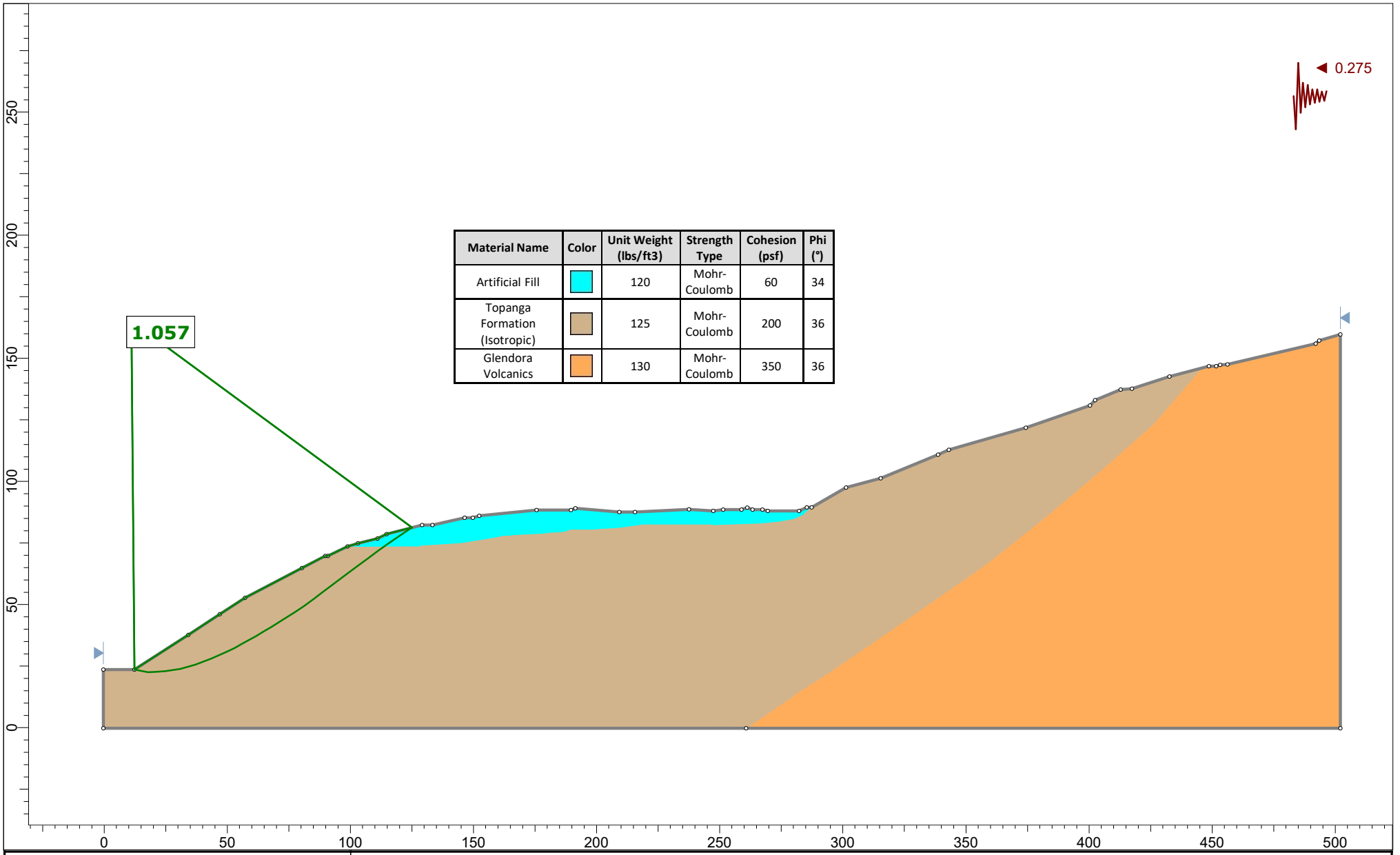


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Anisotropic Function
Artificial Fill		120	Mohr-Coulomb	60	34	
Topanga Formation (Anisotropic)		125	Anisotropic Function			Topanga
Glendora Volcanics		130	Mohr-Coulomb	350	36	

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SLIDEINTERPRET 9.037

Project	Glendora Bike Park		
Group	Section A-A'	*Scenario*	Non-Circular Cuckoo - Seismic
Drawn By	JMH	*Company*	Geocon West, Inc.
Date	1/7/2025, 9:37:28 AM	*File Name*	W1983-06-01 Slide2 Model 2025-01-09.slmd



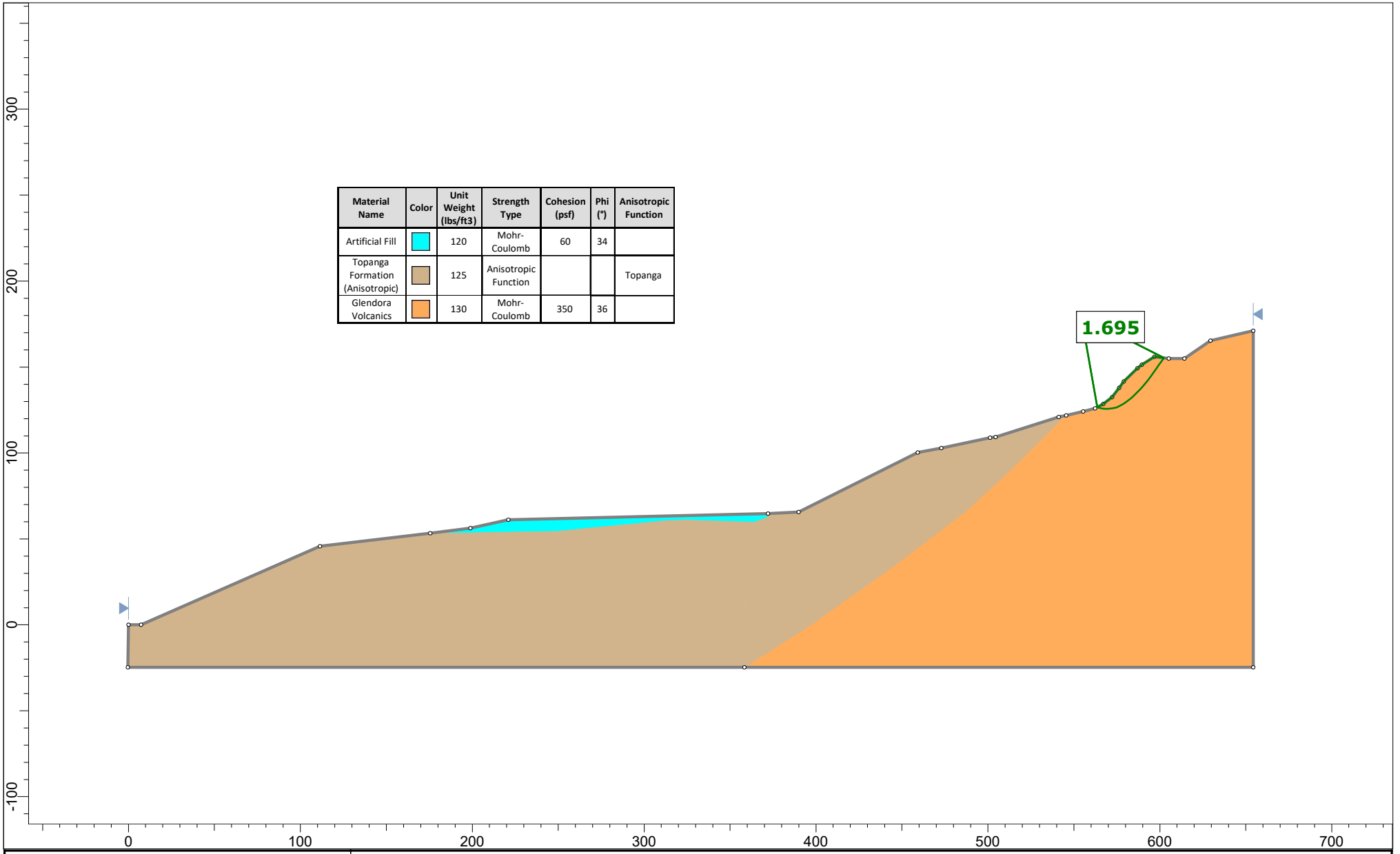
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)
Artificial Fill		120	Mohr-Coulomb	60	34
Topanga Formation (Isotropic)		125	Mohr-Coulomb	200	36
Glendora Volcanics		130	Mohr-Coulomb	350	36

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Project	Glendora Bike Park		
Group	Section B-B'	*Scenario*	Non-Circular Cuckoo - Seismic
Drawn By	JMH	*Company*	Geocon West, Inc.
Date	1/7/2025, 9:37:28 AM	*File Name*	W1983-06-01 Slide2 Model 2025-01-09.slmd

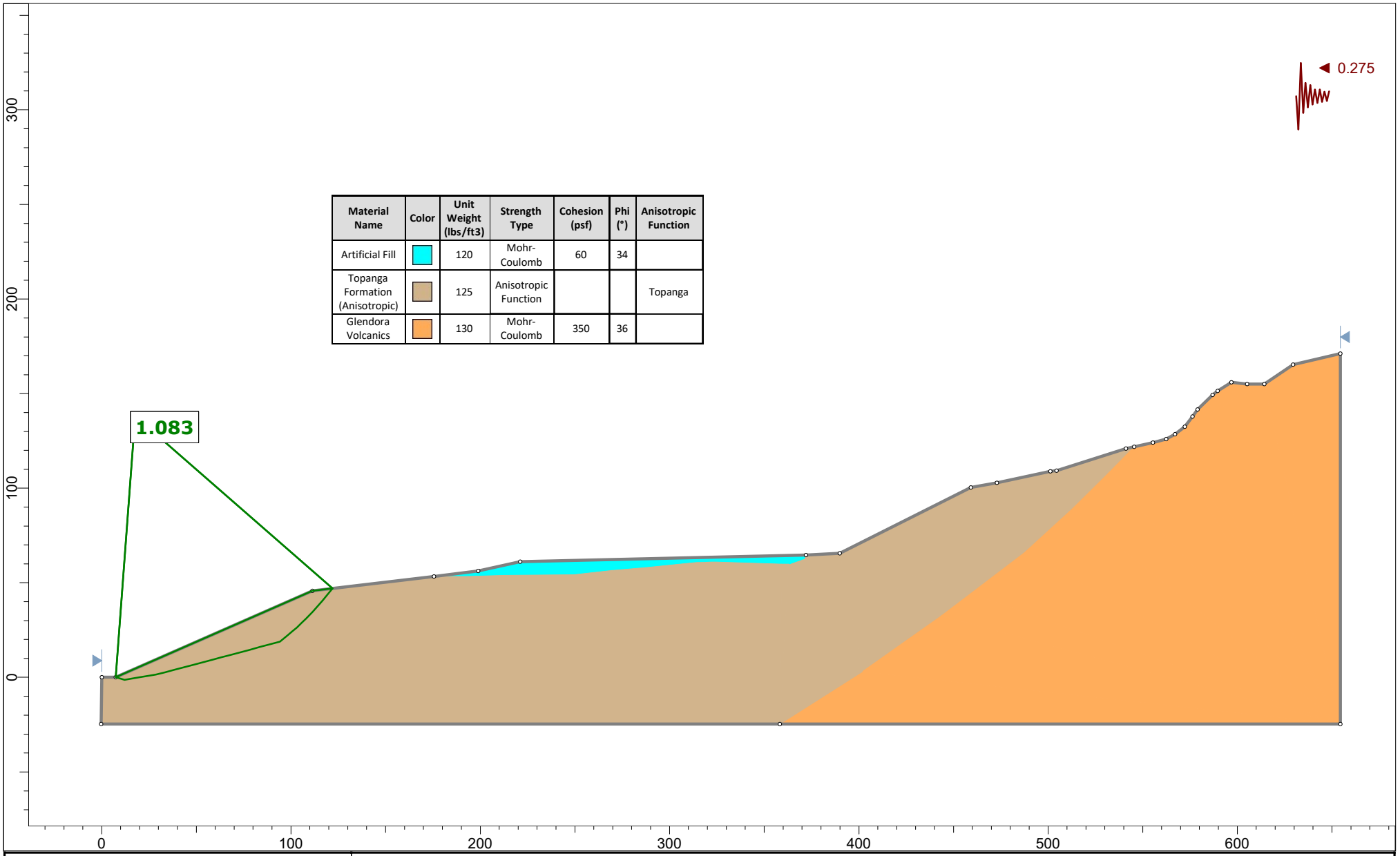
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Anisotropic Function
Artificial Fill		120	Mohr-Coulomb	60	34	
Topanga Formation (Anisotropic)		125	Anisotropic Function			Topanga
Glendora Volcanics		130	Mohr-Coulomb	350	36	



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Project	Glendora Bike Park		
Group	Section A-A' Remedial Grading	*Scenario*	Non-Circular Cuckoo - Static
Drawn By	JMH	*Company*	Geocon West, Inc.
Date	1/7/2025, 9:37:28 AM	*File Name*	W1983-06-01 Slide2 Model 2025-01-09.slmd



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Anisotropic Function
Artificial Fill		120	Mohr-Coulomb	60	34	
Topanga Formation (Anisotropic)		125	Anisotropic Function			Topanga
Glendora Volcanics		130	Mohr-Coulomb	350	36	

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SLIDEINTERPRET 9.037

Project	Glendora Bike Park		
Group	Section A-A' Remedial Grading	*Scenario*	Non-Circular Cuckoo - Seismic
Drawn By	JMH	*Company*	Geocon West, Inc.
Date	1/7/2025, 9:37:28 AM	*File Name*	W1983-06-01 Slide2 Model 2025-01-09.slmd