# REPORT OF LIMITED GEOTECHNICALINVESTIGATION
Cunningham/Shaw Residence ADU
2734 Bordeaux Avenue
La Jolla, California

**JOB NO. 22-14083**
15 March 2023

Prepared for:
***Dr. Sharon Shaw and Ms. Nicole Napper***

**Geotechnical Exploration, Inc.**

SOIL AND FOUNDATION ENGINEERING ● GROUNDWATER ● ENGINEERING GEOLOGY

15 March 2023

Dr. Sharon Shaw and Ms. Nicole Napper
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**Job No. 22-14083**

Subject: **Report of Limited Geotechnical Investigation**
Cunningham/Shaw Residence ADU
2734 Bordeaux Avenue
La Jolla, California

Dear Dr. Shaw and Ms. Napper:

In accordance with your request and our work agreement of September 26, 2022, ***Geotechnical Exploration, Inc.*** has performed a preliminary geotechnical investigation for the subject project in La Jolla, California. The field work and site reconnaissance were performed on December 2, 2022.

If the conclusions and recommendations presented in this report are incorporated into the design and construction of the proposed new ADU and the associated improvements, it is our opinion that the site is suitable for the proposed project improvements.

This opportunity to be of service is sincerely appreciated. Should you have any questions concerning the following report, please do not hesitate to contact us. Reference to our **Job No. 22-14083** will expedite a response to your inquiries.

Respectfully submitted,

**GEOTECHNICAL EXPLORATION, INC.**

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# REPORT OF LIMITED GEOTECHNICAL INVESTIGATION

Cunningham/Shaw Residence ADU
2734 Bordeaux Avenue
La Jolla, California

## JOB NO. 22-14083

The following report presents the findings and recommendations of ***Geotechnical Exploration, Inc.*** for the subject project. Refer to Figure No. I for Vicinity Map.

## I. PROJECT SUMMARY

It is our understanding, based on review of available conceptual plans prepared by Island Architects, the subject property is proposed to receive the addition of a new two-story Accessory Dwelling Unit (ADU). The ADU is proposed at the rear of the existing residence bridging over an existing northeastern to southwestern trending canyon without blocking the natural flow of the drainage canyon.

The new two-story ADU is to be constructed with standard-type building materials supported on a new foundation excavated into adequate bearing soils. Foundation loads are expected to be typical for this type of relatively light construction, and based on the proposed design of spanning over the canyon as well as the off-the-ground elevated first floor of the ADU, we are recommending that the building footprint loads be carried to both sides of the canyon by a combination of pier (caisson) and grade-beam foundations. Deepened conventional continuous footings can be utilized, if feasible, on the buried gray water tank/mechanical and powder location.

When final plans are completed, they should be made available for our review. Additional or modified recommendations may be provided at that time if warranted.

A review of the City of San Diego Seismic Safety Study Geologic Hazards Map Sheet No. 30 indicates that the area of study is located within Geologic Hazards Category (GHC) 53. Category 53 is identified as "Level or sloping terrain, unfavorable geologic structure, Low to Moderate Risk." The *Geologic Map of San Diego, 30'x60' Quadrangle, CA* by Kennedy and Tan, 2008, indicates that the subject site is located in an area near a contact between Quaternary-age Very Old Paralic Deposits Unit 11 ($Qvop_{11}$) and Tertiary-age Scripps Formation (Tsc). The Quaternary Very Old Paralic Deposits Unit 11 are described as very old terrace deposits consisting of*: "poorly sorted, moderately permeable, reddish-brown, interfingered strandline, beach, estuarine and colluvial deposits composed of siltstone, sandstone and conglomerate*." The Tertiary Scripps Formation is described as "*mostly pale-yellowish-brown, medium-grained sandstone containing occasional cobble conglomerate interbeds."*

Our field work revealed that the area of study is underlain by up to 4.5 feet of artificial fill and up to 2.5 feet of topsoil overlying dense Scripps Formation deposits.

Based on our current understanding of the planned construction, it is our opinion that the proposed ADU and associated site improvements would not destabilize neighboring properties or induce the settlement of adjacent structures or right-of-way improvements if designed and constructed in accordance with our recommendations. It is also our opinion that neither an active nor a potentially active fault underlies the subject site.

Please be aware that the importance of thorough observation and testing during construction should be recognized by the client and the contractor(s) to provide appropriate documentation for any necessary as-graded reports. Recommendations for observation and testing are provided under the "Conclusions and Recommendations" section of this report.

## II. SCOPE OF WORK

The scope of work performed for this investigation was based on the conceptual plans prepared by Islands Architects (undated), and included a site reconnaissance, review of available published information pertaining to the site geology, evaluation of the bearing characteristics of the encountered surficial fill, topsoil and formational deposits, geotechnical engineering analysis of the field data, and the preparation of this report. The data obtained and the analyses performed were for the purpose of providing geotechnical design and construction criteria for the proposed construction of the new two-story ADU as well as the associated improvements.

## III. SITE DESCRIPTION

The property is known as Assessor's Parcel No. 344-100-03-00, Lot 58 of Tract 3361, in the La Jolla area of the City of San Diego, State of California. For the purpose of this report, the front of the property is described as facing east (in actuality it faces slightly southeast). For site location, refer to the Vicinity Map, Figure No. I.

The irregular-shaped, 0.880-acre lot is currently developed with a single-family residence, with a swimming pool and associated improvements. The rear of the residence has a drainage canyon that descends generally from northeast to southwest. The proposed two-story ADU is to be located at the top portion of the drainage canyon below the northwestern corner of the existing residence. Presently, the canyon is covered with landscape and natural vegetation.

The property is bordered on the north and south by similar residential properties; on the east by Bordeaux Avenue; and on the west by the aforementioned southwestern draining canyon.

Elevations across the area of study range from approximately +322 feet above Mean Sea Level (AMSL) at the southern portion of the proposed ADU footprint to approximately +336 and +340 feet AMSL along the northwestern and northeastern corners of the proposed ADU, respectively. Approximate elevations were obtained from the topographic survey map prepared by Pasco Laret Suiter and Associates and signed on February 21, 2023. The preliminary plans prepared by Island Architects dated February 15 2023, indicate an ADU first level of +331.5 feet AMSL. Refer to Figure No. II, Plot Plan with Site-specific Geologic Map.

## IV. FIELD INVESTIGATION, OBSERVATIONS & SAMPLING

The field investigation consisted of a surface reconnaissance and a subsurface exploration program utilizing hand tools and a thin-walled hand driven sampler to investigate and sample the subsurface soils on December 2, 2022. Four exploratory handpits (HP-1 to HP-4) were excavated through the existing fill and topsoil, into the dense formational deposits in the area of the proposed ADU and associated improvements. The exploratory handpits were excavated to depths ranging from 3 to 5 feet in order to define the soil profile across the proposed ADU site and to obtain representative soil samples. The soils encountered in the handpits were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (refer to Appendix A). The approximate locations of the exploratory handpits and site-specific geology are shown on Figure No. II, Plot Plan with Site-Specific Geology Map.

Representative soil samples were obtained from the exploratory hand pits at selected depths appropriate to the investigation. Soil sampling included in-place samples and bulk samples collected from the exploratory handpits to aid in classification and for appropriate laboratory testing. All samples were returned to our laboratory for evaluation and testing. Exploratory handpit logs (HP-1 to HP-4) were prepared on the basis of our observations and laboratory test results and are attached as Figure Nos. IIIa-d.

The exploratory handpit logs and related information reveal subsurface conditions only at the specific locations shown on the plot plan and on the particular date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at the explored locations. Also, the passage of time may result in changes in the subsurface conditions due to environmental changes.

## V. LABORATORY TESTING & SOIL INFORMATION

Laboratory tests were performed on the retrieved soil samples in order to evaluate their physical and mechanical properties and their ability to support the proposed two-story ADU and associated improvements. Test results are presented at their respective depths on Figure Nos. IIIa-d and IVa-b. The following tests were conducted on representative soil samples:

1. *Moisture Content (ASTM D2216-19)*
2. *Density Measurements (ASTM D1188-07 and* D2937-17e2*)*
3. *Laboratory Compaction Characteristics (ASTM D1557-21)*
4. *Determination of Percentage of Particles Passing No. 200 Sieve (ASTM D1140-17)*
5. *Expansion Index (ASTM D4829-19)*
6. *Direct Shear Test of Soils (ASTM D3080-11)*

Moisture content and density measurements were performed by ASTM methods D2216-19 and D2937-17e2 respectively to establish the in-situ moisture and density of samples retrieved from the exploratory trenches. The test results are presented on the handpit logs at the appropriate sample depths and laboratory test results.

Laboratory compaction values (ASTM D1557-12e1) establish the optimum moisture content and the laboratory maximum dry density of the tested soils. The relationship between the moisture and density of remolded soil samples helps to establish the relative compaction of the existing fill and soil compaction conditions to be anticipated during any future grading operation. The test results are presented on the excavation logs at the appropriate sample depths and in the laboratory, test results.

The particle size smaller than a No. 200 sieve analysis (ASTM D1140-17) aids in classifying the tested soils in accordance with the Unified Soil Classification System and provides qualitative information related to engineering characteristics such as expansion potential, permeability, and shear strength. The test results are presented on Figure Nos. IIIa-d at the appropriate sample depths.

The expansion potential of soils is determined, when necessary, utilizing the Standard Test Method for Expansion Index of Soils (ASTM D4829-19). In accordance with the Standard (Table 5.3), potentially expansive soils are classified as follows:

| ***EXPANSION INDEX (EI)*** | ***POTENTIAL EXPANSION*** |
|---|---|
| 0 to 20 | Very low |
| 21 to 50 | Low |
| 51 to 90 | Medium |
| 91 to 130 | High |
| Above 130 | Very high |

Based on our laboratory test, a representative sample of the formational material yielded an expansion index of 21 or low expansion potential.

A direct shear test (ASTM D3080-11) was performed on a representative soil sample of the formational soil. The sample was remolded to approximately 90 percent of MDD and sheared under saturated conditions at different normal soil pressures.

Based on the field and laboratory test data, our observations of the primary soil types, and our previous experience with laboratory testing of similar soils, our Geotechnical Engineer has assigned values for friction angle, coefficient of friction, and cohesion for those soils that will have significant lateral support or load bearing functions on the project. The assumed soil strength values have been utilized in determining the recommended bearing value as well as active and passive earth pressure design criteria for foundations and retaining walls.

## VI. REGIONAL GEOLOGIC DESCRIPTION

San Diego County has been divided into three major geomorphic provinces: The Coastal Plain, the Peninsular Ranges and the Salton Trough. The Coastal Plain exists west of the Peninsular Ranges. The Salton Trough is east of the Peninsular Ranges. These divisions are the result of the basic geologic distinctions between the areas. Mesozoic metavolcanic, metasedimentary and plutonic rocks predominate in the Peninsular Ranges with primarily Cenozoic sedimentary rocks to the west and east of this central mountain range (Demere, 1997). Refer to Appendix B for more detailed regional geologic descriptions.

## VII. SITE-SPECIFIC SOIL & GEOLOGIC DESCRIPTION

Review of the "Geologic Map of San Diego, 30'x60' Quadrangle, CA" by Kennedy and Tan, 2008, indicates that the subject site is located in an area near a contact between Quaternary (middle to early Pleistocene) Very Old Paralic Deposits Unit 11 ($Qvop_{11}$) and Tertiary (Middle Eocene) Scripps Formation (Tsc). The Quaternary Very Old Paralic Deposits Unit 11 are described as very old terrace deposits consisting of "*poorly sorted, moderately permeable, reddish-brown, interfingered strandline, beach, estuarine and colluvial deposits composed of siltstone, sandstone and* *conglomerate."* The Tertiary Scripps Formation is described as "*mostly pale-yellowish-brown, medium-grained sandstone containing occasional cobble conglomerate interbeds*." An excerpt of the geologic map and legend is included as Figure No. V.

## *A. Stratigraphy*

*Fill Soil (Qaf):* Our exploratory handpits indicate approximately 1.5 to 4.5 feet of fill in the general area of the proposed ADU footprint. The encountered fill was observed to generally consist of dark brown, fine to medium-grained silty sand. These fill soils are generally loose to medium dense and contain abundant roots in the upper 1 to 2 feet. In our opinion, due to the generally loose density and poor condition of the fill soils, they are unsuitable in their current condition for support of loads from structural foundations or additional fill. Refer to Figure Nos. IIIa-d.

*Topsoil (Qc):* Exploratory handpits HP-2 and HP-4 encountered topsoil deposits overlying the formational soils. These soils were observed to range from 1 to 2.5 feet below existing elevations. The observed topsoils consisted of a light brown, dry, loose to medium dense, fine grained silty sand, and were observed to contain abundant roots. In our opinion, this material is not suitable for support of loads from structural foundations when placed to our recommended depth. Refer to Figure Nos. IIIb and IIId.

*Scripps Formation (Tsc):* We encountered formational soils underlying the fill and topsoil material in all exploratory handpits. The encountered formational soils consisted of Tertiary Scripps Formation that was generally yellowish-brown sandy clay to sandy silt and was noted to be hard, dense and damp. The formational material is considered adequate to support loads from proposed new structure foundations.

### *B.* ***Structure***

Our exploration in the area of the proposed ADU encountered Tertiary (Middle Eocene) Scripps Formation (Tsc), consisting mostly of massive, well cemented, yellowish-brown clayey sandstone to sandy claystone. Visible geologic structure was not identified during our field investigation. Our review of the *Geologic Map of San Diego, 30'x60' Quadrangle, CA* by Kennedy and Tan, 2008, indicates bedding mapped approximately 530 feet west of the site with a strike of N81°W with a dip of 3°N, and approximately 1,500 feet south-southeast of the site with a strike of N70°E with a dip of 7°N. These bedding attitudes indicate generally favorable geologic conditions.

It is our opinion that the geologic structure of the site does not represent a geologic hazard to the proposed ADU and should be considered stable from a geotechnical perspective.

## VIII. GEOLOGIC HAZARDS CATEGORY

Our review of the City of San Diego Seismic Safety Study Geologic Hazards Map Sheet No. 30 indicates that the area of study is located within Geologic Hazards Category (GHC) 53. Category 53 is identified as "*Level or sloping terrain, unfavorable geologic structure, low to moderate risk.*" Based on our reconnaissance and the data obtained in our field investigation, review of pertinent geological literature and analysis of geological maps and aerial photographs, it is our opinion that the area of study has favorable geologic structure, is low risk from a geologic hazard perspective, and there are no known recent, active landslide deposits underlying the site. Our review of available geologic and fault hazards maps and reports, indicate that neither an active nor potentially active fault underlies the site in the area of the proposed ADU structure and improvements.

The following is a discussion of the geologic conditions and hazards common to this area of San Diego, as well as project-specific geologic information relating to development of the subject site.

### *A. __Local and Regional Faulting__*

The primary seismic considerations for improvements at the subject site are surface rupture of fault traces, damage caused by ground shaking during a seismic event, and seismically-induced ground settlement. The potential for any or all of these hazards depends upon the recency of fault activity and the proximity of nearby faults to the subject site. Our review of the proper literature (CGS 2021a) indicates that the subject site lies outside the present Earthquake Fault Zones described in the Alquist-Priolo Earthquake Fault Zoning Act. The major active faults nearest to the site are all part of the Newport-Inglewood-Rose Canyon Fault Zone. The following local fault zones are mapped in general proximity to the site:

- Mount Soledad Fault Zone: Mapped approximately 1.3 miles southwest of the site and considered part of the Rose Canyon Fault Zone.
- Rose Canyon Fault Zone: The northern portion is mapped approximately 1.48 miles south of the site.
- The Spanish Bight, Coronado Fault, Silver Strand and Downtown Graben Fault: These are formed by several splices of the Rose Canyon portion of the Newport-Inglewood-Rose Canyon Fault Zone. This zone of faulting is located approximately 7 to 15 miles south-southeast of the area of study. Review of the available

references indicate that the Rose Canyon Fault Zone system is considered capable of generating an M6.9 earthquake (EERI, 2021).

Other regional faults include:

- The Coronado Bank Fault Zone, the San Diego Trough Fault Zone and the San Clemente Fault Zone: Mapped approximately 13.5, 24 and 49 miles southwest of the site, respectively.
- The Julian and the Temecula sections of the Elsinore Fault Zone: Mapped approximately 35.5 and 37.5 miles northeast of the site, respectively, and estimated to be capable of a M6.5 to M7.5 (Southern California Earthquake Data Center; SCEDC, 2023).
- The San Jacinto Fault: Mapped approximately 62 to 67 miles northeast of the site. This fault is estimated to be capable of a M6.5 to M7.5 (Southern California Earthquake Data Center, SCEDC, 2023).

## ***B. Other Geologic Hazards***

*Ground Rupture*: Ground rupture is characterized by bedrock slippage along an established fault and may result in displacement of the ground surface. For ground rupture to occur along a fault, an earthquake usually exceeds M5.0. If a M5.0 earthquake were to take place on a local fault, an estimated surface-rupture length 1 mile long could be expected (Greensfelder, 1974). Our investigation indicates that the subject site is not directly on a known active fault trace and, therefore, the risk of ground rupture is remote.

*Ground Shaking*: Structural damage caused by seismically induced ground shaking is a detrimental effect directly related to faulting and earthquake activity. Ground shaking is considered to be the greatest seismic hazard in San Diego County. The intensity of ground shaking is dependent upon the magnitude of the earthquake, the distance from the earthquake, and the seismic response characteristics of underlying soils and geologic units. Earthquakes of M5.0 or greater are generally associated with significant damage. It is our opinion that the most serious damage to the site would be caused by a large earthquake originating on a nearby strand of the Rose Canyon, Coronado Bank or Newport-Inglewood Faults. Although the chance of such an event is remote, it could occur within the useful life of the structures.

*Landslides*: Our review of the *Geologic Map of San Diego, 30'x60' Quadrangle, CA* by Kennedy and Tan (2008) and the *USGS US Landslide Inventory*, indicates that there are no landslides identified at the site. These references indicate evidence of landsliding approximately 1,000 feet west of the site. Our site reconnaissance did not reveal evidence of landsliding underlying the site. As such, it is our opinion that there are no known recent or active landslides underlying the site and the site is considered to be stable.

*Slope Stability:* Our site reconnaissance, subsurface investigation and review of the available documents indicates that the new two-story ADU structure and associated improvements are to be located in areas underlain by dense to very dense materials at relatively shallow depths. We performed slope stability calculations considering the steepness of the canyon area slopes where the ADU is to be located, and also considering the depth of excavations. See Figure Nos. IIb-f for cross sections used in our slope stability calculations. In our opinion, there are no slope stability issues in the proposed building area. Refer to Appendix C for slope stability calculations.

*Liquefaction*: The liquefaction of saturated sands during earthquakes can be a major cause of damage to buildings. Liquefaction is the process by which soils are transformed into a viscous fluid that will flow as a liquid when unconfined. It occurs primarily in loose, saturated sands and silts when they are sufficiently shaken by an earthquake. On this site, the risk of soil liquefaction due to seismic shaking is considered to be very low due to the dense nature of the underlying formational materials and lack of shallow static groundwater. In our opinion, the site does not have a potential for soil strength loss to occur due to a seismic event due to the cemented formational soils.

*Tsunamis and Seiches*: A tsunami is a series of long waves generated in the ocean by a sudden displacement of a large volume of water. Underwater earthquakes, landslides, volcanic eruptions, meteor impacts, or onshore slope failures can cause this displacement. Tsunami waves can travel at speeds averaging 450 to 600 miles per hour. As a tsunami nears the coastline, its speed diminishes, its wave length decreases, and its height increases greatly. After a major earthquake or other tsunami-inducing activity occurs, a tsunami could reach the shore within a few minutes. One coastal community may experience no damaging waves while another may experience very destructive waves. Some low-lying areas could experience severe inland inundation of water and deposition of debris more than 3,000 feet inland.

Review of the available references (Tsunami Inundation Map for Emergency Planning, La Jolla Quadrangle, 2009) indicates that the site is located outside the tsunami inundation line at a distance of approximately 0.5-mile from the exposed coastline and at an elevation of approximately +330 feet AMSL. There appears to be no risk of tsunami inundation at the site.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. There are no significant bodies of water located at higher elevation or in the general vicinity capable of producing a seiche and inundating the subject site.

*Flooding:* Review of FEMA Flood Insurance Rate Map 06073C1601G, effective 05/16/2012, indicates the site is within Zone X, described as *"Areas determined to be outside the 0.2% annual chance floodplain."* It is our opinion that the risk of flooding does not exist at the site.

*Geologic Hazards Summary:* No significant geologic hazards are known to exist on the site that would prohibit the construction of the proposed apartment structure and associated improvements. Ground shaking from earthquakes on active southern California faults and active faults in northwestern Mexico is the greatest geologic hazard at the property. Design of the two-story ADU and associated improvements in accordance with the current building codes would reduce the potential for injury or loss of human life. Structures constructed in accordance with current building codes may suffer significant damage but should not undergo total collapse.

It is our opinion, based upon a review of the available maps, our research and our site investigation, that construction of the proposed ADU and associated improvements would not destabilize neighboring properties or induce the settlement of adjacent structures or right-of-way improvements if designed and constructed in accordance with our recommendations.

## IX. GROUNDWATER

Groundwater was not encountered in any of our exploratory excavations. We do not anticipate significant groundwater problems to develop in the future, *if the property is developed as proposed and proper drainage is implemented and maintained*.

It should be kept in mind that any required construction operations will change surface drainage patterns and/or reduce permeabilities due to the densification of compacted soils. Such changes of surface and subsurface hydrologic conditions, plus irrigation of landscaping or significant increases in rainfall, may result in the appearance of surface or near-surface water at locations where none existed previously. The damage from such water is expected to be localized and cosmetic in nature, if good positive drainage is implemented, as recommended in this report, during and at the completion of construction. Due to the proximity of foundations and walls of the proposed structure to running surface water in the underlying canyon, it is recommended that adequate footing embedment and wall protection waterproofing be provided.

On properties such as the subject site where dense, low permeability soils exist at shallow depths, even normal landscape irrigation practices on the property or neighboring properties, or periods of extended rainfall, can result in shallow "perched" water conditions. The perching (shallow depth) accumulation of water on a low permeability surface can result in areas of persistent wetting and drowning of lawns, plants and trees. Resolution of such conditions, should they occur, may require site-specific design and construction of subdrain and shallow "wick" drain dewatering systems.

Subsurface drainage with a properly designed and constructed subdrain system will be required along with continuous back drainage behind any proposed lower-level basement walls, property line retaining walls, or any perimeter stem walls for raised-wood floors where the outside grades are higher than the crawl space grades. Furthermore, crawl spaces, if used, should be provided with the proper cross-ventilation to help reduce the potential for moisture-related problems. Additional recommendations may be required at the time of construction.

It must be understood that unless discovered during site exploration or encountered during site construction operations, it is extremely difficult to predict if or where perched or true groundwater conditions may appear in the future. When site fill or formational soils are fine-grained and of low permeability, water problems may not become apparent for extended periods of time.

Water conditions, where suspected or encountered during construction, should be evaluated and remedied by the project civil and geotechnical consultants. The project developer and property owner, however, must realize that post-construction appearances of groundwater may have to be dealt with on a site-specific basis. Proper functional surface drainage should be implemented and maintained at the property.

## X. CONCLUSIONS & RECOMMENDATIONS

The following recommendations are based upon the practical field investigations conducted by our firm, and resulting laboratory tests, in conjunction with our knowledge and experience with similar soils in the La Jolla area. The opinions, conclusions, and recommendations presented in this report are contingent upon ***Geotechnical Exploration, Inc.*** being retained to review the final plans and specifications as they are developed and to observe the site earthwork and installation of foundations. Accordingly, we recommend that the following paragraph be included on the grading and foundation plans for the project.

> *If the geotechnical consultant of record is changed for the project, the work shall be stopped until the replacement has agreed in writing to accept responsibility within their area of technical competence for approval upon completion of the work. It shall be the responsibility of the permittee to notify the governing agency in writing of such change prior to the recommencement of grading and/or foundation installation work and comply with the governing agency's requirements for a change to the Geotechnical Consultant of Record for the project.*

From a geotechnical engineering standpoint, it is our opinion, based on our current understanding of the proposed construction, that the site is suitable for the planned ADU and improvementsprovided the recommendations herein are incorporated during design and construction. Further, based on our current understanding of the proposed construction, it is our opinion that the ADU construction would not destabilize neighboring properties or induce the settlement of adjacent structures if designed and constructed in accordance with our recommendations.

Our investigation indicates that the area for the proposed ADU is underlain by fill soils and unsuitable topsoil with depths ranging from 1 to 4.5 feet in the vicinity of the areas explored. The fill and topsoils beneath the proposed ADU area were found to be loose to medium dense and are therefore not considered suitable for supporting conventional shallow foundations.

We therefore provide site preparation recommendations that include removal and proper recompaction of the existing fill and topsoils to 5 feet in depth where encountered in the area of the proposed ADU to produce uniform support for exterior improvements such as patios, walkways, parking areas, etc. In addition, we are providing drilled pier recommendations for the northern foundation area of the ADU, and deepened conventional footings for the southern area of the ADU where a subterranean tank is proposed.

Construction plans have not been provided to us for the preparation of this report, however, when completed they should be made available for our review. Additional or modified recommendations for foundation design and construction may be provided as warranted.

The opinions, conclusions and recommendations presented in this report are contingent upon ***Geotechnical Exploration, Inc.*** being retained to review the final plans and specifications as they are developed and to observe the site earthwork and installation of foundations.

## A. Preparation of Soils for Site Development

1. *General:* Grading should conform to the guidelines presented in the 2022 California Building Code (CBC) as well as the requirements of the City of San Diego.

   During earthwork construction, removals and reprocessing of fill materials, as well as general grading procedures of the contractor should be observed and the fill placed selectively tested by representatives of the geotechnical engineer, ***Geotechnical Exploration Inc***. If any unusual or unexpected conditions are exposed in the field, they should be reviewed by the geotechnical engineer and if warranted, modified and/or additional remedial recommendations will be offered. The contractor should properly plan the excavations and temporary and/or permanent support necessary to maintain the stability of the existing structure and the excavation soil walls. If shoring is planned, the soil design parameters recommended herein should be used

The recommendations presented herein have been completed using the information provided to us regarding site development. If information concerning the proposed development is revised, or any changes in the design and location of the proposed ADU is modified after issuing this report, this office should be notified and the changes should be evaluated to determine if the recommendations presented in this report still apply.

2. *Clearing and Stripping:* In areas to receive the new ADU and improvements, any structures and landscaping improvements should be removed before the loose or soft fill soils and topsoils are removed (anticipated to be less than approximately 5 feet). This is to include the complete removal of all surface and subsurface obstructions (i.e., concrete footings, utility lines and miscellaneous debris) that may exist in those areas. After clearing, the ground surface should be stripped of existing vegetation within the areas of proposed new construction. This includes any roots from existing trees and shrubbery.

   Once the required excavations have been made down to suitable soils, holes resulting from the removal of root systems or other buried obstructions that extend below the planned grades should be cleared and backfilled with suitable compacted material compacted to the requirements provided under Recommendation Nos. 3, 4 and 5 below.

   Prior to any filling operations, the cleared and stripped vegetation and debris should be disposed of off-site. Areas with deeper inadequate soils should be prepared with grading equipment to provide access for construction equipment. This preparation also applies to areas where the site topography dictates and in areas to receive drilled piers (on the north side) or the underground tank (south side).

3. *Excavation:* After the area of proposed construction has been cleared and stripped, all of the existing fill soils and topsoil in the areas to receive new foundations or hardscape improvements should be removed and recompacted where applicable. The depth of soil removal across the site will vary depending upon the thickness of unsuitable soils overlying the dense formational materials. It is anticipated that the depth of unsuitable soil removal will be approximately 2.5 to 5 feet below existing grade in the explored areas of the proposed ADU. Based on our exploratory excavations, the extent of unsuitable fill and topsoils in all areas to receive structural improvements that are proposed to bear on compacted fill soils must be removed to expose suitable medium dense to dense sandy soil and replaced with compacted fill soils.

   Based on our experience with similar materials in the project area, it is our opinion that the existing fill soils and topsoil materials can be excavated utilizing ordinary light to heavy weight earthmoving equipment. Contractors should not, however, be relieved of making their own independent evaluation of excavating the on-site materials prior to submitting their bids. Contractors should also review this report along with the excavation logs to understand the scope and quantity of grading required for this project. Variability in excavating the subsurface materials should be expected across the project area. Undercutting may be recommended at time of grading if shallow fills are encountered in one area and deeper fills are encountered in other areas of the proposed ADU.

The areal extent required to remove the surficial soils and existing fill should be confirmed by our representatives during the excavation work based on their examination of the soils being exposed. If necessary, the lateral extent of the excavation and fill recompaction should be at least 5 feet beyond the edge of the perimeter of the proposed ADU building footprint and any areas to receive exterior improvements, or fill slopes, where feasible, or to the depth of excavation or fill at that location, whichever is greater.

At the time of removal and recompaction work, the bottom of the excavation should be observed by a representative of ***Geotechnical Exploration Inc***., to determine the location of the transition that will be needed to finalize the design of the pier and grade beam portion of the foundation system. After determination of the transition location, the bottom of the excavation should be scarified to a minimum depth of 6 inches below removal grade elevations, brought to near-optimum moisture conditions and recompacted to at least 90 percent relative compaction (based on ASTM Test Method D1557).

4. *Subgrade Preparation:* After the proposed ADU site has been cleared, stripped, and the required excavations made, the exposed approved subgrade soils in areas to receive new fill and/or slab on-grade improvements should be scarified to a depth of 6 inches, moisture conditioned, and compacted to the requirements for structural fill. While not anticipated, in the event that planned cuts expose any medium to highly expansive soil materials in the building areas, they should be scarified and moisture conditioned to at least 5 percent.

5. *Material for Fill:* Existing on-site low expansion potential (Expansion Index of 50 or less per ASTM D4829-19) soils with an organic content of less than 3 percent by volume are, in general, suitable for use as fill or backfill. Imported

fill material, where required, should have a low expansion potential. In addition, both imported and existing on-site materials for use as fill should not contain rocks or lumps more than 6 inches in greatest dimension if the fill soils are compacted with heavy compaction equipment (or 3 inches in greatest dimension if compacted with lightweight equipment). All materials for use as fill should be approved by our representative prior to importing to the site.

If encountered on-site, medium or highly expansive soils cannot be used for retaining wall backfill. Backfill material to be placed behind retaining walls should be low expansive (E.I. less than 50), with rocks no larger than 3 inches in diameter.

6. *Structural Fill Compaction:* All structural fill and backfill in designated areas to receive shallow footings, retaining wall backfill and any associated improvements, should be compacted to a minimum degree of compaction of 90 percent based upon ASTM D1557-12e1. Fill material should be spread and compacted in uniform horizontal lifts not exceeding 8 inches in uncompacted thickness. Before compaction begins, the fill should be brought to a water content that will permit proper compaction by either: (1) aerating and drying the fill if it is too wet, or (2) watering the fill if it is too dry. Each lift should be thoroughly mixed before compaction to ensure a uniform distribution of moisture. Low expansive soils should be moisture conditioned to 3 percent above optimum moisture content.

   Soil compaction testing by nuclear method ASTM D6938-17a or sand cone method ASTM D1556-15e1 should be performed a minimum of every 2 feet of fill placement by a representative of ***Geotechnical Exploration, Inc.*** Furthermore, our representative should perform necessary observation of fill placement during grading operations throughout the project.

Any rigid improvements founded on the existing loose or soft soils can be expected to undergo movement and possible damage. ***Geotechnical Exploration, Inc.*** takes no responsibility for the performance of any improvements built on loose natural soils or inadequately compacted fills. Subgrade soils in any exterior area receiving concrete improvements should be verified for compaction and moisture by a representative of our firm within 48 hours prior to concrete placement.

No uncontrolled fill soils should remain after completion of the site work. In the event that temporary ramps or pads are constructed of uncontrolled fill soils, the loose fill soils should be removed and/or recompacted prior to completion of the grading operation.

7. *Water-Soluble Sulfate and Chloride Testing*: We recommend that the water-soluble sulfate content and chloride content of the near-surface soils be tested at the completion of grading or foundation excavations. The test results should be evaluated by an engineer specializing in corrosivity. Cement type recommendations should be provided by the structural engineer based on the current edition of the CBC (2022) or the American Concrete Institute and the soluble sulfate and chloride test results.

8. *Trench and Retaining Wall Backfill:* All utility trenches and retaining walls such in the area of the underground tank, should be backfilled with properly compacted low expansive on-site soils, or imported fill capped (upper 8 inches) with properly compacted on-site soils. Imported backfill material should be placed in lift thicknesses appropriate to the type of compaction equipment utilized and compacted to a minimum degree of compaction of 90 percent by mechanical means. Any portion of the trench backfill in public street areas

within pavement sections should conform to the material and compaction requirements of the adjacent pavement section.

Our experience has shown that even shallow, narrow trenches (such as for irrigation and electrical lines) that are not properly compacted can result in problems, particularly with respect to shallow groundwater accumulation and migration.

9. *Observations and Testing*: As stated in CBC 2022, *Section 1705.6 Soils:* "*Special inspections and tests of existing site soil conditions, fill placement and load-bearing requirements shall be performed in accordance with this section and Table 1705.6 (see below). The approved geotechnical report and the construction documents prepared by the registered design professionals shall be used to determine compliance. During fill placement, the special inspector shall verify that proper materials and procedures are used in accordance with the provisions of the approved geotechnical report." A summary of* Table 1705.6 "REQUIRED SPECIAL INSPECTIONS AND TESTS OF SOILS" is presented below:

   a) *Verify materials below shallow foundations are adequate to achieve the design bearing capacity;*
   b) *Verify excavations are extended to proper depth and have reached proper material;*
   c) *Perform classification and testing of compacted fill materials;*
   d) *Verify use of proper materials, densities and thicknesses during placement and compaction of compacted fill prior to placement of compacted fill, inspect subgrade and verify that site has been prepared properly*

Section 1705.6 “Soils” statement and Table 1705.6 indicate that it is mandatory that a representative of this firm (responsible engineering firm), perform observations and fill compaction testing during excavation operations to verify that the remedial operations are consistent with the recommendations presented in this report. All grading excavations resulting from the removal of soils should be observed and evaluated by a representative of our firm before they are backfilled.

Quality control grading observation and field density testing for the purpose of documenting adequate compaction has been achieved and acceptable soils have been utilized to properly support a project applies not only to fill soils supporting primary structures, but all site improvements such as stairways, patios, decking, sidewalks, driveways and retaining walls, building additions, ADU’s etc. Observation and testing of utility line trench backfill also reduces the potential for localized settlement of all of the above including all improvements outside of the footprint of primary structures.

The Geotechnical Engineer of Record, in this case ***Geotechnical Exploration, Inc.,*** cannot be held responsible for the costs and time delays associated with the lack of contact and requests for testing services by the client, general contractor, grading contractor or any of the project design team responsible for requesting the required geotechnical services. Request for services is to be made through our office telephone number (858) 549-7222 and the telephone number of the GEI personnel assigned to the project or via email at least 24 hours in advance prior to the needed service visit.

## ***B. Seismic Design Criteria***

10. *Seismic Data Bases:* The estimation of the peak ground acceleration and the repeatable high ground acceleration (RHGA) likely to occur at the site is based on the known significant local and regional faults within 100 miles of the site.

11. *Seismic Design Criteria:* The proposed structure should be designed in accordance with the 2022 CBC, which incorporates by reference the ASCE 7-16 for seismic design. We have determined the mapped spectral acceleration values for the site based on a latitude of 32.8661 degrees and a longitude of 117.24543 degrees, utilizing a program titled "S*eismic Design Map Tool,"* which provides a solution for ASCE 7-16 utilizing digitized files for the Spectral Acceleration maps. See Appendix D for the ASCE Seismic Summary Report.

12. *Structure and Foundation Design*: The design of the new structures and foundations should be based on Seismic Design Category D, Risk Category II for a Site Class D "Stiff Soils," which considers the properly compacted fill or foundations bearing in dense formational soils.

13. *Spectral Acceleration and Design Values*: The structural seismic design, when applicable, should be based on the following seismic soil parameter values, which are based on the site location, soil characteristics, and seismic maps by USGS, as required by the 2022 CBC. Seismic design soil parameters were obtained with the SEAOC Seismic Design Map Tool and they are presented in summarized form below. A full computer printout is presented as Appendix D.

### TABLE I
### *Mapped Spectral Acceleration Values and Design Parameters*

| $S_S$ | $S_1$ | $S_{MS}$ | $S_{M1}$ | $S_{DS}$ | $S_{D1}$ | Fa | Fv | PGA | $PGA_M$ | SDC |
|---|---|---|---|---|---|---|---|---|---|---|
| 1.371 | 0.48 | 1.371 | 0.873 | 0.914 | 0.582 | 1.0 | 1.819 | 0.626 | 0.688 | D |

## C. <u>Design Parameters for Foundations</u>

As discussed previously, based on the proposed design of the planned ADU spanning over the canyon as well as the off-the-ground elevated first floor, we recommend the building footprint loads be carried to both sides of the canyon with the use of drilled piers (caissons) structurally interconnected with the rest of the structure via grade beams on the north side of the ADU, and by deepened strip footings on the south side, where an underground rectangular tank is proposed beneath the first floor of the ADU.

14. *Drilled Pier (Caisson) and Grade Beam*: We recommend that the grade beam foundation system be supported by frictional and end-bearing drilled piers (caissons) that penetrate into the undisturbed formational materials at the recommended depth. This depth considers the steep existing slope in front of the north portion of the ADU and from which a minimum 8 feet of lateral cover must be provided to the lower elevation piers. As explained before, the southern portion foundations of the ADU may use conventional strip footings deepened to provide lateral protection from the gentler descending slope on that side of the project.

15. *Grade Beams:* Any grade beams should be founded at least 18 inches below the lowest adjacent finished grade and should have the final dimensions specified by the structural designer. Since the ground surface slopes toward the center of the drainage canyon as well as toward the southwest, the

proposed structure effective foundation should be provided with a setback of at least 8 feet to daylight to a descending slope. The grade beams should contain top and bottom reinforcement to provide structural continuity and to permit spanning of local irregularities. The final dimensions and reinforcing should be specified by the structural engineer based on the spacing of the caissons as well as load per caissons. A minimum clearance of 3 inches should be maintained between steel reinforcement and the bottom or sides of the footing.

16. *Grade Beam Bearing Capacity*: At the recommended dimensions, the grade beams will be designed to support the corresponding part of the structure and transmit the loads to the drilled piers. Therefore, the northern half of the building loads will be carried by drilled piers. The southern half of the structure will be supported by a mat slab or a conventional slab surrounded by building retaining walls enclosing a gray water underground tank. Perimeter strip foundations deepened to provide at least 8 feet to daylight from descending slope faces should be used where applicable. The allowable soil bearing capacity for strip footings is 3,000 pounds per square foot (psf) for combined dead and live loads for footings into dense formational soils penetrating at least 18 inches into the ground and at least 12 inches wide. An increase in soil allowable static bearing can be used as follows: 1,200 psf for each additional foot over 1 foot in depth into formational soils and 500 psf for each additional foot in width to a total not exceeding 5,000 psf.

*NOTE: The project Civil/Structural Engineer should review all reinforcing schedules. The reinforcing minimums recommended herein are not to be construed as structural designs, but merely as minimum reinforcement to reduce the potential for cracking and separations.*

17. *Vertical Piers (caissons) Loading:* For vertical loading, all end-bearing drilled piers (caissons) should be embedded at least 18 feet (measured from the downhill side of the excavation), into dense undisturbed formational materials (through the existing fill soils and any top soil/or slopewash if encountered).

    The grade beams and piles should be designed for a downhill creep pressure of 600 pounds per lineal foot of drilled pier exposed to creep prone material (if loose fill soils, topsoil or slopewash material is not removed). Based on our field investigation, up to 4.5 feet of fill/topsoil was encountered near the northeastern corner of the proposed ADU building footprint (see Figure Nos. IIIa-d).

18. *Vertical Caisson End-Bearing Capacity*: The recommended allowable end bearing capacity is 18,000 psf for caissons penetrating at least 16 feet into dense formational soils and at least 18 feet below the soil surface. This allowable bearing value may be increased by one-third for transient loads such as wind or seismic. The actual required caisson length and embedment into formational soils should be established by the structural engineer based on the length required to adequately support the total vertical and lateral loads included in the design and will also be determined by the required set-back from the natural face of slope. Caissons on a slope face or within 10 feet from the slope top should have the effective embedment measured from the point where the setback distance from caisson end face to the slope face is a minimum 8 feet to daylight. Proper caisson penetration into dense formational materials should be confirmed by a representative of our firm at the time of drilling/excavation.

The recommended allowable end-bearing vertical capacity already includes the effect of negative friction produced by the existing fills. Any caisson weight (150 pcf) above the soil surface should be considered as dead load and should be deducted from the net end-bearing capacity. The effective load of the buried part of caissons may be calculated as being 30 pcf.

It is important that when drilling caissons, in order to utilize the end-bearing capacity, the amount of loose material at the bottom of the excavation should be limited. For end-bearing capacity caissons, no slough over 1 inch in thickness should remain at the bottom of the excavation before concrete placement. Therefore, we recommend that caissons be designed with a minimum diameter of 24 inches in order to facilitate observation of the excavations and allow ease of material removal at the bottom. The drilling contractor should provide an appropriate cleaning tool to satisfy this requirement. Otherwise, casing installation and hand-tool cleaning (or another acceptable option) will be required. The shaft friction capacity of the caissons may be added to the end-bearing capacity if caissons are drilled more than 8 feet into dense formational materials. The recommended allowable average shaft frictional resistance is 700 psf applicable in the portion of the caisson embedded over 8 feet into dense materials, as measured below the effective embedment depth.

19. *Caisson Spacing*: As noted previously, the minimum center-to-center spacing of caissons, for vertical and lateral load support should be at least three caisson diameters. For caissons located in the same line of direction as the applied lateral load, the shadow effect produces a reducing factor in their combined individual lateral load capacity. For spacing that ranges from 3B, 4B, 5B, 6B, and farther (where B is the caisson diameter), the reduction factors in the

caisson group should be 0.6, 0.75, 0.9 and 1.0 for leading caissons, and 0.41, 0.60, 0.78, 0.9, and 1.0, respectively, for trailing caissons.

20. *Caisson Weight:* The caisson weight to be considered is 30 pcf as the weight of buried caisson. For any portion of the caisson above ground, the full weight should be considered. The required caisson diameter, length and embedment into dense formational materials should be established by the structural engineer based on the length needed to adequately support the total vertical and lateral loads included in the design.

21. *Lateral Resistance*: For lateral earthquake or wind load resistance, the structural engineer may use any allowable method that considers the equilibrium of forces and moments.

22. *Caisson Passive Resistance:* If a balance of forces is calculated based on the applied lateral forces and soil reaction forces, the allowable passive (equivalent fluid) forces recommended are 150 pcf for existing level fill and 300 pcf for dense formational materials. Passive resistance of caissons should be measured beginning at an effective embedment depth where at least 8 feet of horizontal daylight distance to the slope face is provided from the face of leading caissons closer to the slope face. Total passive resistance of the caissons may be considered applicable on a projected surface equal to 2½ times the diameter of the caisson, multiplied by the vertical length being considered below the effective start of passive resistance embedment.

23. Similar soil passive resistance values may be used for portions of the project where shallow or conventional footings are used, in addition to a frictional resistance calculated with an allowable friction coefficient of 0.40.

24. *Caisson Drilling Observations*: Caisson drilling or excavation operations should be performed under the continued observations of a representative of our firm to confirm the penetration into dense landslide materials.

25. *Caisson Design Standards:* The design and construction of the caissons should be in accordance with the recommendations presented above, the current CBC requirements accepted by the City of San Diego, and also in accordance with ACI 336, 3R-14 Design and Construction of Drilled Caissons, of the American Concrete Institute.

26. *Filling of Caisson Excavations*: Caisson excavations should be filled with concrete within 2 days after the excavations are completed to help reduce the risk of soil caving, mud or slough intrusion, etc. Slough material filling the bottom of drilled holes should be removed prior to concrete placement. If caving occurs while drilling caissons (prior to reaching the required depth), shoring will be required. Some caving is anticipated to occur due to the loose soil conditions of the fill soils with little cohesion. Shoring may be removed while placing concrete using the Tremie method. Other options in lieu of drilled excavation shoring may be considered if pre-approved by our office.

27. *Cal-OSHA Guidelines*: All excavations should follow Cal-OSHA guidelines for safety purposes.

28. *Settlement:* Settlement under building loads is expected to be within tolerable limits after underpinning of the existing structure is completed. For foundations designed in accordance with the recommendations presented in the preceding paragraphs, we anticipate that total settlement should not

exceed 1 inch and that post-construction angular rotation should be less than 1/240.

## *D.* ***Concrete Slab On-Grade Criteria***

As noted previously, we anticipate that a concrete slab may be used on the south side of the project where an underground gray water tank is proposed. The slab may be a mat slab or a conventional, properly designed slab on-grade forming part of the adjacent retaining wall foundation. The slab and foundations will need to bear on dense formational or properly compacted fill. Mat slabs may be designed for an allowable soil bearing capacity of 2,500 psf when built on dense formational soils or properly compacted fill. To help reduce the potential for soil moisture intrusion the slab may be underlain by a StegoWrap 15 Mil vapor barrier placed directly underneath the slab and on top of the formational soils or properly compacted fill. Shrinkage control and isolation joints should be specified and detailed by the structural engineer.

29. *Exterior Slab Thickness and Reinforcement:* Exterior slabs should be at least 4 inches thick and reinforced with No.3 bars spaced at 15 inches on centers. Shrinkage control joints should be spaced no farther than 8 feet or the width of the slab and at reentrant corners. All exterior slabs should be built on properly compacted soils or dense natural soils.

    The performance of on-site improvements can be greatly affected by soil base preparation and the quality of construction. It is therefore important that all improvements are properly designed and constructed for the existing soil conditions. The improvements should not be built on loose soils or fills placed without our observation and testing. The subgrade of exterior improvements should be verified during fill or backfill placement and to be as properly prepared within 48 hours prior to concrete placement. A minimum thickness of 4 feet of properly recompacted fill soils, where applicable, should underlie the exterior slabs on-grade or they should be constructed on dense formational soils.

## ***E. Retaining Wall Design Criteria***

30. *Design Parameters – Unrestrained:* The active earth pressure to be utilized in the design of any cantilever site retaining walls, utilizing on-site low expansive [EI less than 50] if encountered during grading operations, or imported very low- to low-expansive soils [EI less than 50] as backfill should be based on an Equivalent Fluid Weight of 38 pcf (for level backfill only). For 2.0:1.0 sloping low expansive backfill, the cantilever site retaining walls should be designed with an equivalent fluid pressure of 52 pcf.

Unrestrained retaining walls should be backfilled with properly compacted very low to low expansive soils. Unrestrained retaining walls should be designed when vertical load surcharged using a conversion load factor of 0.31 to convert vertical surcharge loads to uniform horizontal lateral surcharge loads. Temporary cantilever shoring walls may use 40 pcf active pressure, and a conversion factor of 0.35 to convert vertical uniform surcharge to horizontal uniform pressure. For passive resistance, temporary shoring piles may use the value of 750 pcf times the diameter of the soldier pile, times the depth of embedment below the grade excavation in front of the piles for the portions of the pile embedded in formational or properly compacted soils and when the distance to daylight on slope ground is at least 8 feet.

31. *Design Parameters – Restrained:* Temporary or permanent restrained shoring, building, or site retaining walls supporting low expansive level backfill may utilize a triangular pressure increasing at a rate of 56 pcf for wall design (78 pcf for sloping 2.0:1.0 backfill). The soil pressure produced by any footings, improvements, or any other surcharge placed within a horizontal distance equal to the height of the retaining portion of the wall should be considered in the wall design pressure. A conversion factor of 0.47 pcf may be used to convert vertical uniform surcharge loads to lateral uniform pressure behind a restrained retaining wall with level backfill (0.64 when supporting a 2 to 1 sloping backfill). The recommended lateral soil pressures are based on the assumption that no loose soils or unstable soil wedges will be retained by the retaining wall. Backfill soils should consist of low-expansive soils with EI less than 50, placed from the heel of the foundation to the ground surface within the wedge formed by a plane at 30° from vertical, and passing by the heel of the foundation, and include the space between the back face of the retaining wall and the inclined plane described.

32. *Retaining Wall Seismic Design Pressures:* For seismic design of unrestrained walls over 6 feet in exposed height, we recommend that the seismic pressure increment be taken as a fluid pressure distribution utilizing an equivalent fluid weight of 17 pcf. This seismic increment is waived for restrained basement walls. If the walls are designed as unrestrained walls, then the seismic load should be added to the static soil pressure.

33. *Retaining Wall Drainage:* The preceding design pressures assume that the walls are backfilled with properly compacted low expansion potential materials (Expansion Index less than 50) and that there is sufficient drainage behind the walls to prevent the build-up of hydrostatic pressures from surface water

infiltration. We recommend that drainage be provided by a composite drainage material such as J-Drain 200/220 and J-Drain SWD, or equivalent. No perforated pipes or gravel are utilized with the J-Drain system. The drain material should terminate 12 inches below the exterior finish surface where the surface is covered by slabs or 18 inches below the finish surface in landscape areas (see Figure No. VII for Retaining Wall Subdrain Schematic). Waterproofing should extend from the bottom to the top of the wall.

Backfill placed behind retaining walls should be compacted to a minimum degree of compaction of 90 percent using light compaction equipment. If heavy equipment is used, the walls should be appropriately temporarily braced. Crushed rock gravel may only be used as backfill in areas where access is too narrow to place compacted soils. Behind shoring walls sand slurry backfill may be used behind lagging.

***Geotechnical Exploration, Inc.*** will assume no liability for damage to structures or improvements that is attributable to poor drainage. The architectural plans should clearly indicate that subdrains for any lower-level walls be placed at an elevation at least 1 foot ***below*** the bottom of the lower-level slabs.

## *F.* *Slopes*

Temporary slopes may be required during site temporary excavations and construction. Shoring may be required adjacent to property lines where the depth of the excavation may affect existing structures or can cause injury if slope soil collapse occurred.

34. *Temporary Slopes*: Based on our subsurface investigation work, laboratory test results, and engineering analysis, temporary cut slopes up to 10 feet in height should be stable from mass instability at an inclination of 0.75:1.0 (horizontal to vertical) in properly compacted fill or cohesive formation with a high degree of cementation materials, a fines content greater than 35 percent, and no surcharge loads within 10 feet from the top of excavation. Temporary cut slopes up to 10 feet in height in loose/cohesionless soils should be stable against mass instability at an inclination of 1.5:1.0 and also no surcharge within 10 feet of the slope top. Excavations close to existing improvements should be provided with temporary shoring and/or underpinning.

    Some localized sloughing or raveling of the soils exposed on the slopes may occur. Since the stability of temporary construction slopes will depend largely on the contractor's activities and safety precautions (storage and equipment loadings near the tops of cut slopes, surface drainage provisions, etc.), it should be the contractor's responsibility to establish and maintain all temporary construction slopes at a safe inclination appropriate to the methods of operation. No soil stockpiles or surcharge may be placed within a horizontal distance of 10 feet or the depth of the excavation, whichever is larger, from the excavation top.

    If these recommendations are not feasible due to space constraints, temporary shoring may be required for safety and to protect adjacent property improvements. Similarly, footings near temporary cuts should be underpinned and/or protected with shoring, as indicated before.

35. *Temporary Slope Observations*: A representative of ***Geotechnical Exploration, Inc.*** must observe temporary slopes *during construction*. In the event that soils comprising a slope are not as anticipated, any required slope design changes would be presented at that time.

## *G. Site Drainage Considerations*

36. *Erosion Control*: Appropriate erosion control measures should be taken at all times during and after construction to prevent surface runoff waters from entering footing excavations or ponding on finished building pad areas. Structures or foundations close to running surface water shall be sufficiently deepened and designed to support water pressure if the water level raises above ground surface adjacent to the structures.

37. *Surface Drainage:* Adequate measures should be taken to properly finish-grade the lot after the structures and other improvements are in place. Drainage waters from this site and adjacent properties should be directed away from the footings, floor slabs, and slopes, onto the natural drainage direction for this area or into properly designed and approved drainage facilities by the City of San Diego to be indicated by the project Civil Engineer. Roof gutters and downspouts should be installed on the residence, with the runoff directed away from the foundations via closed drainage lines. Proper subsurface and surface drainage will help minimize the potential for waters to seek the level of the bearing soils under the footings, swimming pool and floor slabs.

Failure to observe this recommendation could result in undermining and possible differential settlement of the structure or other improvements on the site or cause other moisture-related problems. Currently, the CBC requires a minimum 2 percent surface gradient for proper drainage of building pads unless waived by the building official. Concrete pavement may have a minimum gradient of 0.5-percent.

38. *Planter Drainage:* Planter areas, flower beds and planter boxes should be sloped to drain away from the footings and floor slabs at a gradient of at least 5 percent within 5 feet of the perimeter walls. Any planter areas adjacent to the residence or surrounded by concrete improvements should be provided with sufficient area drains to help with rapid runoff disposal. No water should be allowed to pond adjacent to the residence or other improvements or anywhere on the site.

39. *Drainage Quality Control*: It must be understood that it is not within the scope of our services to provide quality control oversight for surface or subsurface drainage construction or retaining wall sealing and base of wall drain construction. It is the responsibility of the contractor to verify and provide proper surface drainage at the site, wall sealing, geofabric installation, protection board (if needed), drain depth below interior floor or yard surface, pipe percent slope to the outlet, etc.

## H. General Recommendations

40. *Project Start Up Notification:* In order to reduce work delays during site development, this firm should be contacted 48 hours prior to any need for observation of footing excavations or field density testing of compacted fill soils. If possible, placement of formwork and steel reinforcement in footing excavations should not occur prior to observing the excavations; in the event that our observations reveal the need for deepening or re-designing foundation

structures at any locations, any formwork or steel reinforcement in the affected footing excavation areas would have to be removed prior to correction of the observed problem (i.e., deepening the footing excavation, recompacting soil in the bottom of the excavation, etc.).

41. *Construction Best Management Practices (BMPs):* Construction BMPs must be implemented in accordance with the requirements of the controlling jurisdiction. Sufficient BMPs must be installed to prevent silt, mud or other construction debris from being tracked into the adjacent street(s) or storm water conveyance systems due to construction vehicles or any other construction activity. The contractor is responsible for cleaning any such debris that may be in the street at the end of each work day or after a storm event that causes breach in the installed construction BMPs.

All stockpiles of uncompacted soil and/or building materials that are intended to be left unprotected for a period greater than 7 days are to be provided with erosion and sediment controls. Such soil must be protected each day when the probability of rain is 40% or greater. A concrete washout should be provided on all projects that propose the construction of any concrete improvements that are to be poured in place. All erosion/sediment control devices should be maintained in working order at all times. All slopes that are created or disturbed by construction activity must be protected against erosion and sediment transport at all times. The storage of all construction materials and equipment must be protected against any potential release of pollutants into the environment.

## XI. GRADING NOTES

***Geotechnical Exploration, Inc.*** recommends that we be retained to verify the actual soil conditions revealed during site grading work and footing excavation to be as anticipated in this "*Report of Limited Geotechnical Investigation*" for the project. In addition, the placement and compaction of any fill or backfill soils during site grading work must be observed and tested by the soil engineer.

It is the responsibility of the grading contractor and general contractor to comply with the requirements on the grading plans as well as the local grading ordinance. All retaining wall and trench backfill should be properly compacted. ***Geotechnical Exploration, Inc.*** will assume no liability for damage occurring due to improperly or uncompacted backfill placed without our observations and testing.

## XII. LIMITATIONS

Our conclusions and recommendations have been based on available data obtained from our field investigation and laboratory analysis, as well as our experience with similar soils and formational materials located in this area of La Jolla. Of necessity, we must assume a certain degree of continuity between exploratory excavations and/or natural exposures. It is, therefore, necessary that all observations, conclusions, and recommendations be verified at the time grading operations begin or when footing excavations are placed. In the event discrepancies are noted, additional recommendations may be issued, if required.

The work performed and recommendations presented herein are the result of an investigation and analysis that meet the contemporary standard of care in our profession within the County of San Diego. No warranty is provided.

As stated previously, it is not within the scope of our services to provide quality control oversight for surface or subsurface drainage construction or retaining wall sealing and base of wall drain construction. It is the responsibility of the contractor to verify proper wall sealing, geofabric installation, protection board installation (if needed), drain depth below interior floor or yard surfaces, pipe percent slope to the outlet, etc.

This report should be considered valid for a period of two (2) years, and is subject to review by our firm following that time. If significant modifications are made to the building plans, especially with respect to the height and location of any proposed structures, this report must be presented to us for immediate review and possible revision.

It is the responsibility of the owner and/or developer to ensure that the recommendations summarized in this report are carried out in the field operations and that our recommendations for design of this project are incorporated in the project plans. We should be retained to review the project plans once they are available, to verify that our recommendations are adequately incorporated in the plans. Additional or modified recommendations may be issued if warranted.

This firm does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and we cannot be responsible for the safety of personnel other than our own on the site; the safety of others is the responsibility of the contractor. The contractor should notify the owner if any of the recommended actions presented herein are considered to be unsafe.

The firm of ***Geotechnical Exploration, Inc.*** shall not be held responsible for changes to the physical condition of the property, such as addition of fill soils or changing drainage patterns, which occur subsequent to issuance of this report and the changes are made without our observations, testing, and approval.

Once again, should any questions arise concerning this report, please feel free to contact the undersigned. Reference to our **Job No. 22-14083** will expedite a reply to your inquiries.

Respectfully submitted,

**GEOTECHNICAL EXPLORATION, INC.**

Leslie D. Reed, President
C.E.G. 999/P.G. 3391

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R.C.E. 34422/G.E. 2007
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March 2023

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# APPENDIX B

## *REGIONAL GEOLOGIC DESCRIPTIONS*

In the Coastal Plain region, the "basement" consists of Mesozoic crystalline rocks. Basement rocks are also exposed as high relief areas (e.g., Black Mountain northeast of the subject property and Cowles Mountain near the San Carlos area of San Diego). Younger Cretaceous and Tertiary sediments lap up against these older features. These sediments form a "*layer cake*" sequence of marine and non-marine sedimentary rock units, with some formations up to 140 million years old. Faulting related to the La Nación and Rose Canyon Fault zones has broken up this sequence into a number of distinct fault blocks in the southwestern part of the county. Northwestern portions of the county are relatively undeformed by faulting (Demere, 1997).

The Peninsular Range forms the granitic spine of San Diego County. These rocks are primarily plutonic, forming at depth beneath the earth's crust 140 to 90 million years ago as the result of the subduction of an oceanic crustal plate beneath the North American continent. These rocks formed the much larger Southern California batholith. Metamorphism associated with the intrusion of these great granitic masses affected the much older sediments that existed near the surface over that period of time. These metasedimentary rocks remain as roof pendants of marble, schist, slate, quartzite and gneiss throughout the Peninsular Ranges. Locally, Miocene-age volcanic rocks and flows have also accumulated within these mountains (e.g., Jacumba Valley). Regional tectonic forces and erosion over time have uplifted and unroofed these granitic rocks to expose them at the surface (Demere, 1997).

The Salton Trough is the northerly extension of the Gulf of California. This zone is undergoing active deformation related to faulting along the Elsinore and San Jacinto Fault Zones, which are part of the major regional tectonic feature in the southwestern portion of California, the San Andreas Fault Zone. Translational movement along these fault zones has resulted in crustal rifting and subsidence. The Salton Trough, also referred to as the Colorado Desert, has been filled with sediments to depth of approximately 5 miles since the movement began in the early Miocene, 24 million years ago. The source of these sediments has been the local mountains as well as the ancestral and modern Colorado River (Demere, 1997).

The San Diego area is part of a seismically active region of California. It is on the eastern boundary of the Southern California Continental Borderland, part of the Peninsular Ranges Geomorphic Province. This region is part of a broad tectonic boundary between the North American and Pacific Plates. The actual plate boundary is characterized by a complex system of active, major, right-lateral strike-slip faults, trending northwest/southeast. This fault system extends eastward to the San Andreas Fault (approximately 70 miles from San Diego) and westward to the San

Clemente Fault (approximately 50 miles off-shore from San Diego) (Berger and Schug, 1991).

In California, major earthquakes can generally be correlated with movement on active faults. As defined by the California Division of Mines and Geology, now the California Geological Survey (CGS), an "active" fault, described by CGS (2018) as a Holocene-Active fault, is one that has had (ground) surface displacement within Holocene time, the last 11,700. In addition, "potentially active fault" has been amended to Pre-Holocene fault: a fault whose recency of past movement is older than 11,700 years, and thus does not meet the criteria of Holocene-Active fault as defined in the State Mining and Geology Board regulations.

A three-tier fault classification is used as follows:

- *Holocene-Active Faults* have surface displacement within Holocene time, where Holocene time is the geological epoch that began 11,700 years before present.
- *Pre-Holocene Faults* have demonstrable displacement older than Holocene time.
- *Age-Undetermined Faults* are faults whose age of most recent movement is not known or is unconstrained by dating methods or by limitations in stratigraphic resolution.

During recent history, prior to April 2010, the San Diego County area has been relatively quiet seismically. The youngest paleoearthquake that cuts the early historical living surface is likely the 1862 San Diego earthquake that had an estimated magnitude of M6 (Legg and Agnew, 1979; Singleton et al., 2019). Paleoseismic trenches at the Presidio Hills Golf Course on the main trace of the Rose Canyon Fault contained evidence for historical ground rupturing earthquakes as recently as 1862 and the mid-1700s. Results of the study also suggest the Rose Canyon Fault has a ~700-800-year recurrence interval (Singleton et al., 2019).

On June 15, 2004, a M5.3 earthquake occurred approximately 45 miles southwest of downtown San Diego (26 miles west of Rosarito, Mexico). Another widely felt earthquake on a distant southern California fault was a M5.4 event that took place on July 29, 2008, west-southwest of the Chino Hills area of Riverside County.

Several earthquakes ranging from M5.0 to M6.0 occurred in northern Baja California, centered in the Gulf of California on August 3, 2009. A M5.8 earthquake followed by a M4.9 aftershock occurred on December 30, 2009, centered about 20 miles south of the Mexican border city of Mexicali.

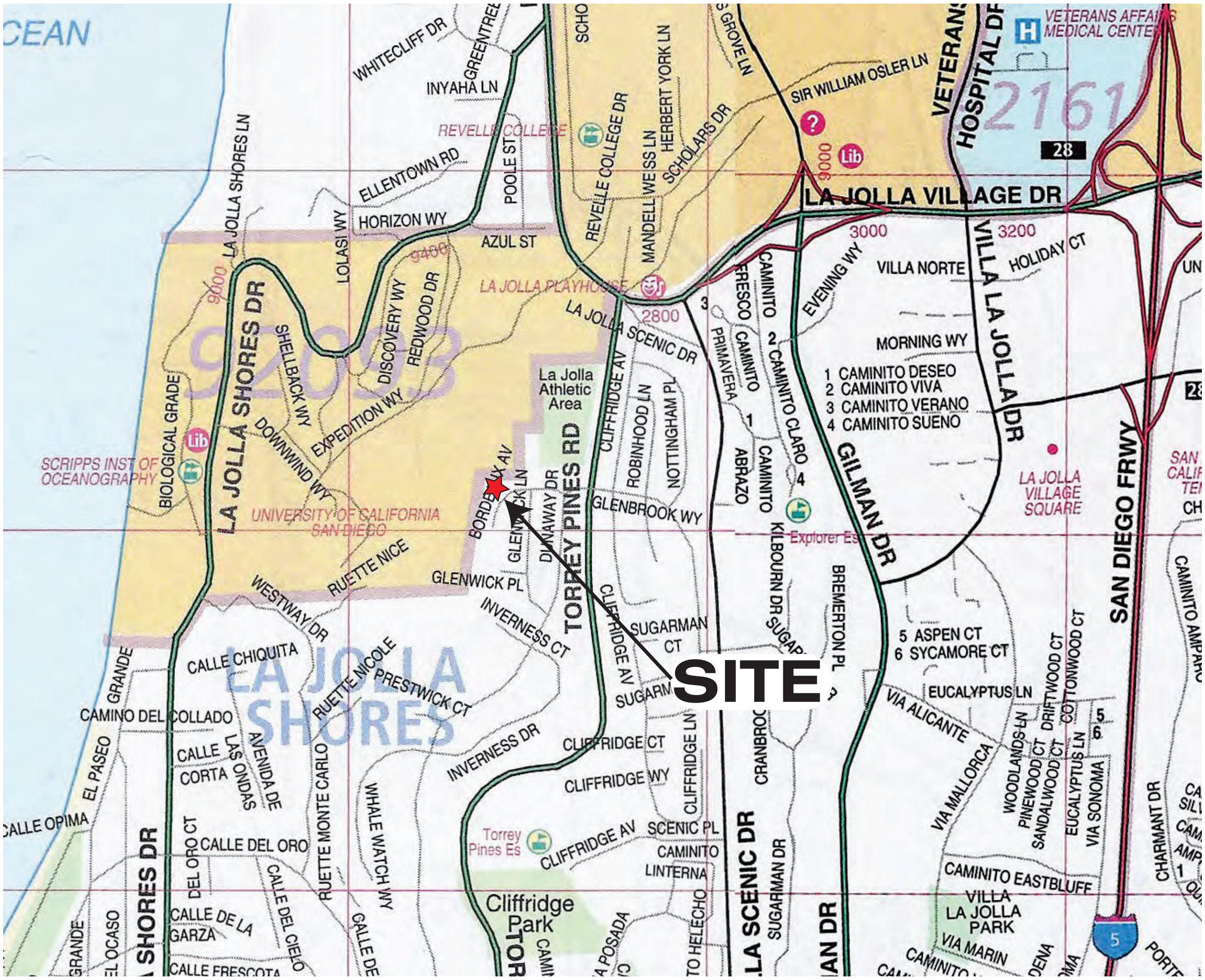
On April 04, 2010, a large earthquake occurred in Baja California, Mexico. It was widely felt throughout the southwest including Phoenix, Arizona and San Diego in California. This M7.2 event, the Sierra El Mayor earthquake, occurred in northern Baja California, approximately 40 miles south of the Mexico-USA border at shallow depth along the principal plate boundary between the North American and Pacific plates. According to the U.S. Geological Survey this is an area with a high level of historical seismicity, and it has recently also been seismically active, although this is the largest event to strike in this area since 1892. The April 04, 2010, earthquake appears to have been larger than the M6.9 earthquake in 1940 or any of the early 20th century events (e.g., 1915 and 1934) in this region of northern Baja California.

This event's aftershock zone extends significantly to the northwest, overlapping with the portion of the fault system that is thought to have ruptured in 1892. Ground motions for the April 04, 2010, main event, recorded at stations in San Diego and reported by the California Strong Motion Instrumentation Program (CSMIP), ranged up to 0.058g.

On July 07, 2010, a M5.4 earthquake occurred in Southern California at 4:53 pm (Pacific Time) about 30 miles south of Palm Springs, 25 miles southwest of Indio, and 13 miles north-northwest of Borrego Springs. The earthquake occurred near the Coyote Creek segment of the San Jacinto Fault. The earthquake exhibited right lateral slip to the northwest, consistent with the direction of movement on the San Jacinto Fault. It was followed by more than 60 aftershocks of M1.3 and greater during the first hour.

In the last 50 years, there have been four other earthquakes in the magnitude M5.0 range within 20 kilometers of the Coyote Creek segment: M5.8 in 1968, M5.3 on 2/25/1980, M5.0 on 10/31/2001, and M5.2 on 6/12/2005. The biggest earthquake near this location was the M6.0 Buck Ridge earthquake on 3/25/1937.

# VICINITY MAP

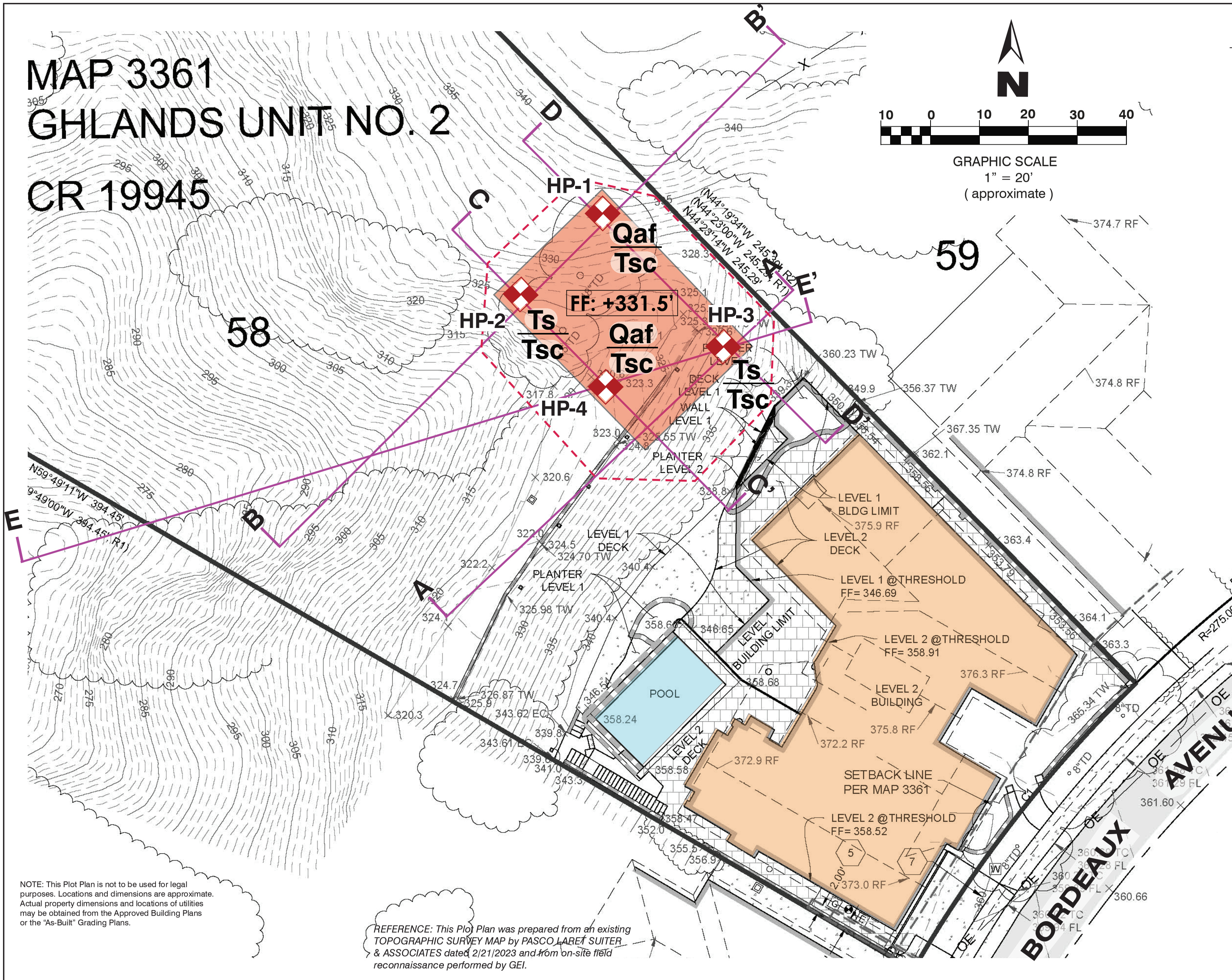


Thomas Bros. Guide San Diego County, pg 1227-J4

Cunningham/Shaw ADU
2734 Bordeaux Avenue
La Jolla, CA.

*Figure No. I*
*Job No. 22-14083*

**Geotechnical Exploration, Inc.**



NOTE: This Plot Plan is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

*REFERENCE: This Plot Plan was prepared from an existing TOPOGRAPHIC SURVEY MAP by PASCO LARET SUITER & ASSOCIATES dated 2/21/2023 and from on-site field reconnaissance performed by GEI.*

*LEGEND*

| | |
|---|---|
| 🟥 HP-4 | Approximate Location of Exploratory Handpit |
| 🟧 | Existing Structure |
| 🟥 | Proposed Structure |
| ▭ | Approximate Limits of Geotechnical Investigation |
| E ⌐ E' | Approximate Location of Cross Section |

*GEOLOGIC UNITS*

| | |
|---|---|
| **Qaf** | Artificial Fill |
| **Ts** | Topsoil |
| **Tsc** | Scripps Formation |

# PLOT PLAN AND SITE SPECIFIC GEOLOGIC MAP

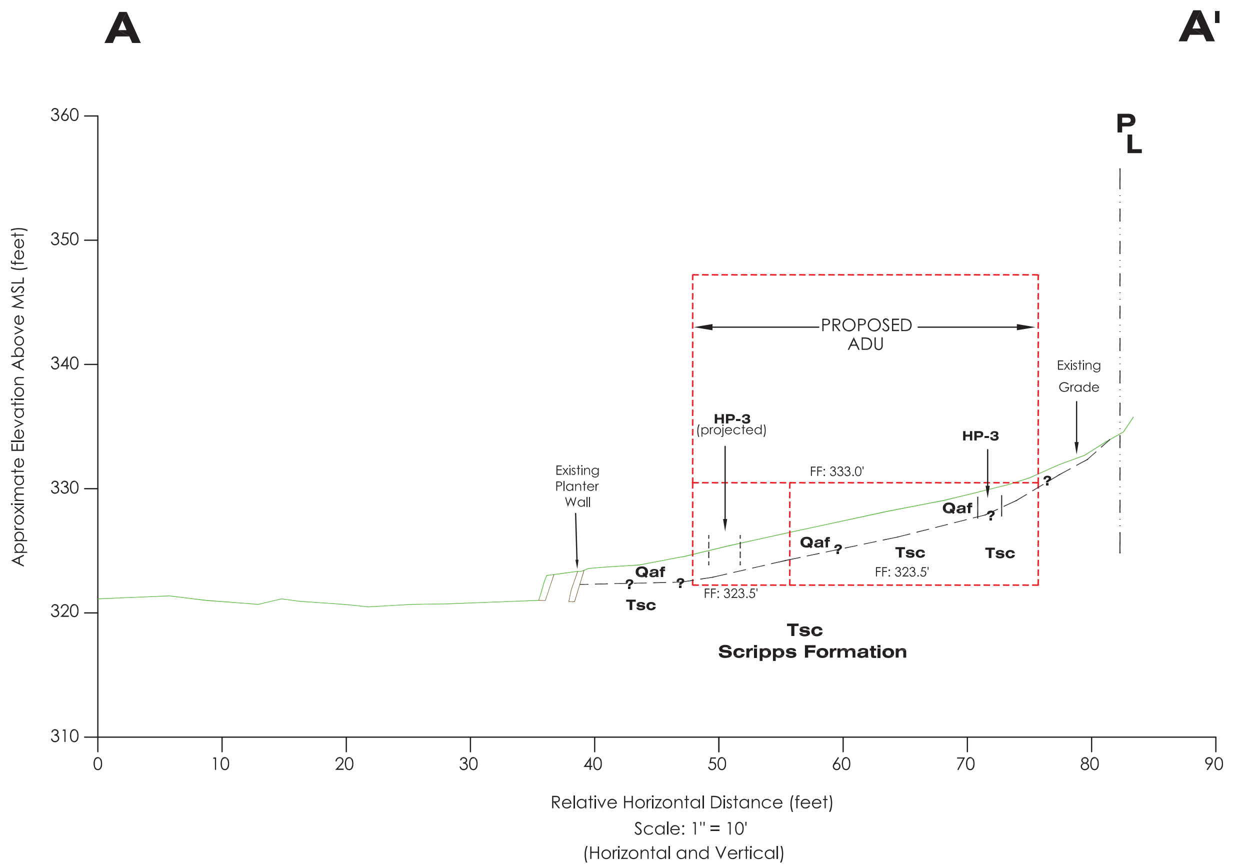
*Cunningham/Shaw ADU*
*2734 Bordeaux Avenue*
*La Jolla, CA.*
*Figure No. IIa*
*Job No. 22-14083*

**Geotechnical Exploration, Inc.**

*March 2023*

# GEOLOGIC CROSS SECTION A-A'

Cunningham/Shaw ADU
2734 Bordeaux Avenue
La Jolla, CA.



| | |
|---|---|
| **Qaf** | Artificial Fill |
| **Tsc** | Scripps Formation |

_ ? _ ? _ Approximate Geologic Contact

NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

22-14083-AA.dwg

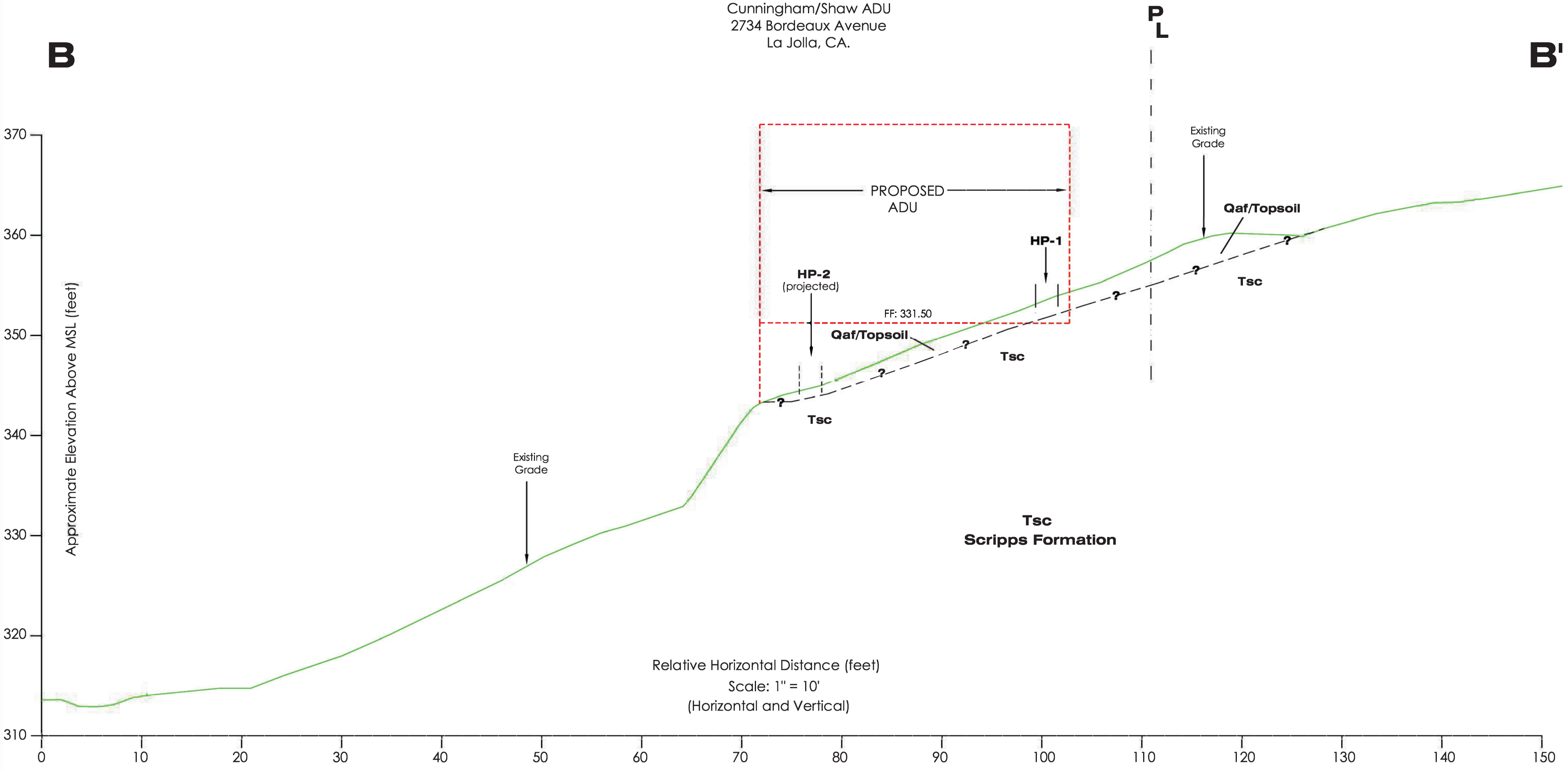
*Figure No.IIb*
*Job No. 22-14083*

**Geotechnical Exploration, Inc.**

*March 2023*

# GEOLOGIC CROSS SECTION B-B'

Cunningham/Shaw ADU
2734 Bordeaux Avenue
La Jolla, CA.



**Qaf** Artificial Fill

**Tsc** Scripps Formation

– ? – ? – Approximate Geologic Contact

NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

22-14083-BB.dwg

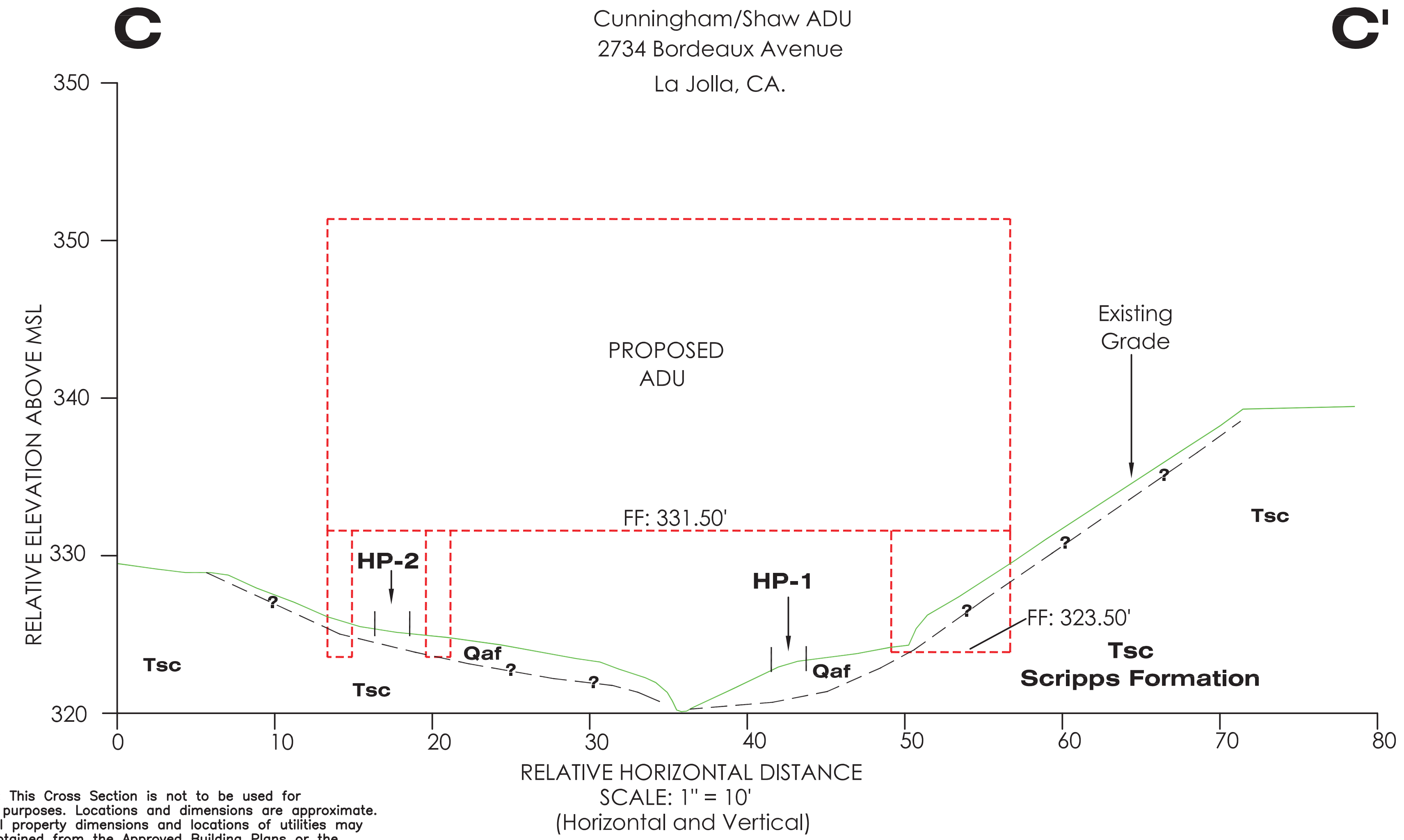
*Figure No. IIc*
*Job No. 22-14083*

**Geotechnical Exploration, Inc.**

*March 2023*

# GEOLOGIC CROSS SECTION C-C'

Cunningham/Shaw ADU
2734 Bordeaux Avenue
La Jolla, CA.



NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As—Built" Grading Plans.

**Qaf** Artificial Fill

**Tsc** Scripps Formation

– ? – ? – Approximate Geologic Contact

*Figure No. IId*
*Job No. 22-14083*

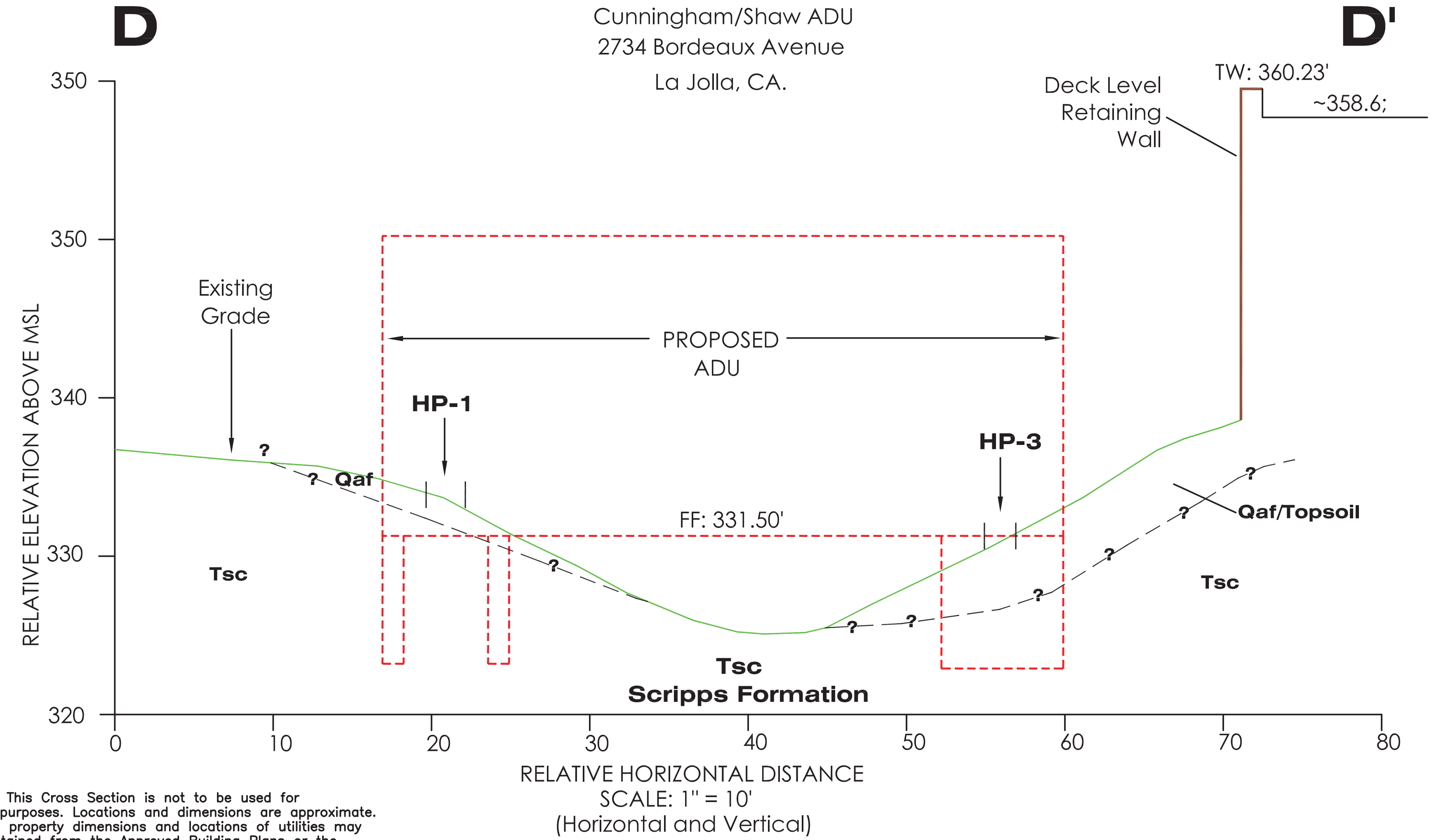
**Geotechnical Exploration, Inc.**

*March 2023*

22-14083-CC.dwg

# GEOLOGIC CROSS SECTION D-D'

Cunningham/Shaw ADU
2734 Bordeaux Avenue
La Jolla, CA.



NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As—Built" Grading Plans.

**Qaf** Artificial Fill

**Tsc** Scripps Formation

– ? – ? – Approximate Geologic Contact

*Figure No. IIe*
*Job No. 22-14083*

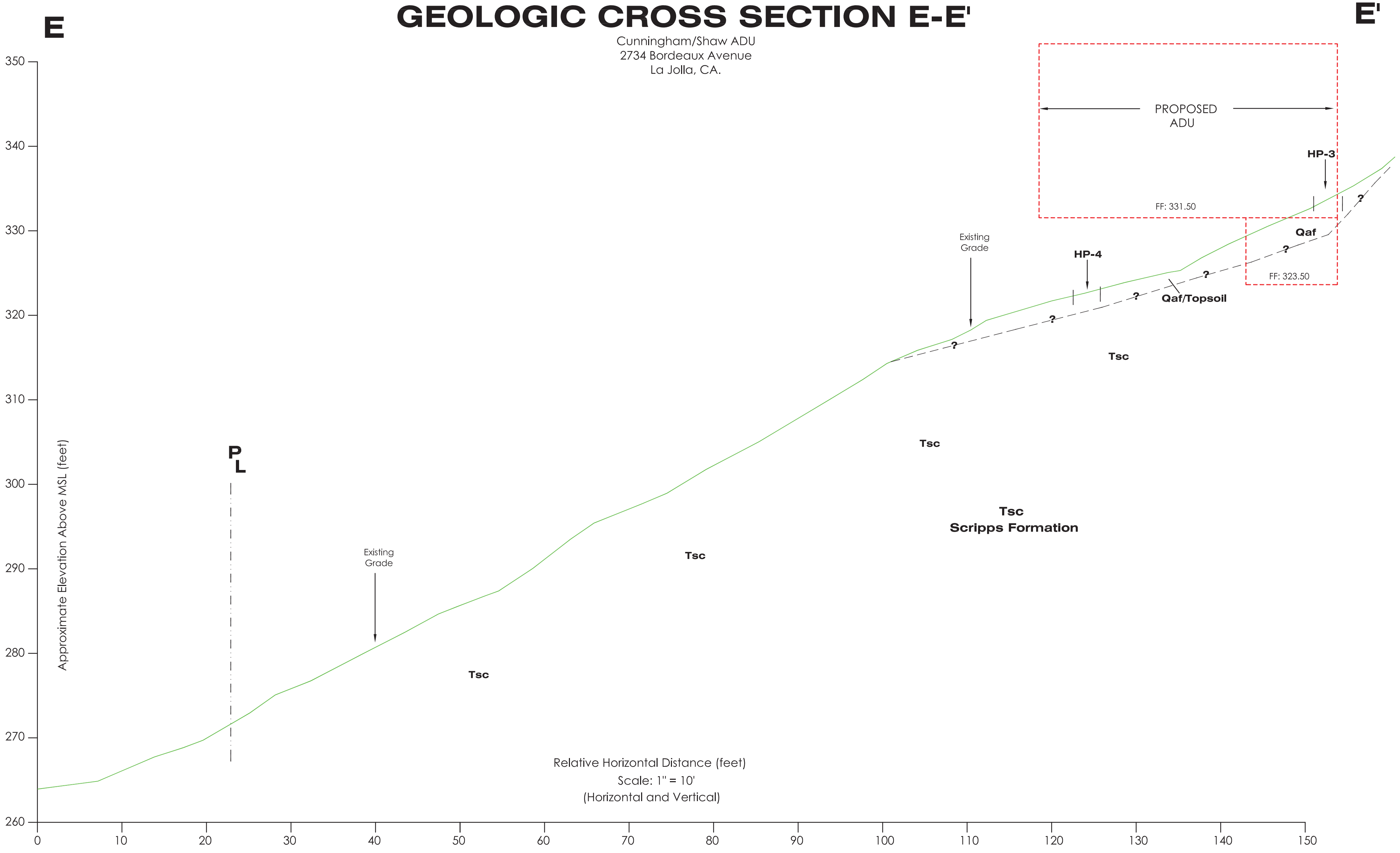
**Geotechnical Exploration, Inc.**

*March 2023*

22-14083-DD.dwg

# GEOLOGIC CROSS SECTION E-E'

Cunningham/Shaw ADU
2734 Bordeaux Avenue
La Jolla, CA.



**Qaf** Artificial Fill

**Tsc** Scripps Formation

– ? – ? – Approximate Geologic Contact

NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

22-14083-EE.dwg

*Figure No.IIf*
*Job No. 22-14083*

**Geotechnical Exploration, Inc.**

*March 2023*

**Geotechnical Exploration, Inc.**

**EQUIPMENT:** Hand Tools

**DIMENSION & TYPE OF EXCAVATION:** 3'x3'x3'

**DATE LOGGED:** December 2, 2022

**LOGGED BY:** HE

**SURFACE ELEVATION:** ± 335' Above Mean Sea Level

**REVIEWED BY:**

**GROUNDWATER/SEEPAGE DEPTH:** Not Encountered

| DEPTH (feet) | SYMBOL | SAMPLE | FTG. DEPTH | BLOWS / 6" | FIELD DESCRIPTION AND CLASSIFICATION<br>DESCRIPTION AND REMARKS (Grain Size, Density, Moisture, Color) | U.S.C.S | IN-PLACE MOISTURE (%) | IN-PLACE DRY DENSITY (pcf) | OPTIMUM MOISTURE (%) | MAXIMUM DRY DENSITY (pcf) | PHI ANGLE (deg.) | COHESION (psf) | EXPANSION INDEX | % PASSING #200 SIEVE | SAMPLE O.D. (in) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | | | | | SILTY SAND, fine grained, loose, dry, light brown, abundant roots and some rock<br>**Artificial Fill (Qaf)** | SM | | | | | | | | | |
| 2 | | 1 | | | SANDY CLAY to CLAYEY SAND, fine grained, hard, dense, damp, yellowish brown well cemented. | CL<br>SC | 6.2 | 88.5 | | | | | | | |
| | | ☒ | | | **Scripps Formation (Tsc)** | | | | 16.1 | 108.6 | 31.0 | 0 | 21 | 70 | |
| 3<br>4<br>5<br>6<br>7<br>8<br>9<br>10 | | | | | Bottom of Excavation at 3.0 ft.<br>No Groundwater, No Caving, Backfilled with Cuttings | | | | | | | | | | |

▒ FOOTING

▼ PERCHED WATER TABLE

☒ BULK BAG SAMPLE

1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

H IN-PLACE HAND-DRIVE SAMPLE

▒ STANDARD PENETRATION TEST

* DISTURBED BLOWCOUNT

**JOB NUMBER:** 22-14083

**JOB NAME:**
Cunningham-Shaw ADU

**SITE LOCATION:**
2734 Bordeaux Ave.
La Jolla, CA

**LOG NO.** **HP-1**

**FIGURE NO.** **IIIa**

**Geotechnical Exploration, Inc.**

**DATE LOGGED:** December 2, 2022
**LOGGED BY:** HE
**REVIEWED BY:**

**EQUIPMENT:** Hand Tools
**DIMENSION & TYPE OF EXCAVATION:** 3'x3'x3'
**SURFACE ELEVATION:** ± 325' Above Mean Sea Level
**GROUNDWATER/SEEPAGE DEPTH:** Not Encountered

| DEPTH (feet) | SYMBOL | SAMPLE | FTG. DEPTH | BLOWS / 6" | FIELD DESCRIPTION AND CLASSIFICATION<br>DESCRIPTION AND REMARKS (Grain Size, Density, Moisture, Color) | U.S.C.S | IN-PLACE MOISTURE (%) | IN-PLACE DRY DENSITY (pcf) | OPTIMUM MOISTURE (%) | MAXIMUM DRY DENSITY (pcf) | DENSITY (% of MDD) | EXPAN (+%) CONSOL (-%) | EXPANSION INDEX | % PASSING #200 SIEVE | SAMPLE O.D. (in) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
|  | ▒ |  |  |  | SILTY SAND, fine grained, loose, dry, light brown, abundant roots.<br>**Topsoil** | SM |  |  |  |  |  |  |  |  |  |
| 1 | ░ |  |  |  | SANDY CLAY to CLAYEY SAND, fine grained, hard, dense, damp, yellowish brown well cemented. | CL<br>SC |  |  |  |  |  |  |  |  |  |
| 2 | ░ |  |  |  | **Scripps Formation (Tsc)** |  |  |  |  |  |  |  |  |  |  |
| 3 |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| 4 |  |  |  |  | Bottom of Excavation at 3.0<br>No Groundwater, No Caving, Backfilled with Cuttings |  |  |  |  |  |  |  |  |  |  |
| 5 |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| 6 |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| 7 |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| 8 |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| 9 |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |
| 10 |  |  |  |  |  |  |  |  |  |  |  |  |  |  |  |

| | |
|---|---|
| ▦ | FOOTING |
| ▼ | PERCHED WATER TABLE |
| ☒ | BULK BAG SAMPLE |
| 1 | IN-PLACE SAMPLE |
| ■ | MODIFIED CALIFORNIA SAMPLE |
| H | IN-PLACE HAND-DRIVE SAMPLE |
| ▨ | STANDARD PENETRATION TEST |

* DISTURBED BLOWCOUNT

**JOB NUMBER:** 22-14083

**JOB NAME:**
Cunningham-Shaw ADU

**SITE LOCATION:**
2734 Bordeaux Ave.
La Jolla, CA

**LOG NO.** **HP-2**

**FIGURE NO.** **IIIb**

**Geotechnical Exploration, Inc.**

**EQUIPMENT: Hand Tools**

**DIMENSION & TYPE OF EXCAVATION:** 3'x3'x3'

**DATE LOGGED: December 2, 2022**

**LOGGED BY: HE**

**REVIEWED BY:**

**SURFACE ELEVATION:** ± 330' Above Mean Sea Level

**GROUNDWATER/SEEPAGE DEPTH:** Not Encountered

| DEPTH (feet) | SYMBOL | SAMPLE | FTG. DEPTH | BLOWS / 6" | FIELD DESCRIPTION AND CLASSIFICATION<br>DESCRIPTION AND REMARKS (Grain Size, Density, Moisture, Color) | U.S.C.S | IN-PLACE MOISTURE (%) | IN-PLACE DRY DENSITY (pcf) | OPTIMUM MOISTURE (%) | MAXIMUM DRY DENSITY (pcf) | DENSITY (% of MDD) | EXPAN (+%) CONSOL (-%) | EXPANSION INDEX | % PASSING #200 SIEVE | SAMPLE O.D. (in) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1<br>2<br>3<br>4 | ▒ | | | | SILTY SAND, fine grained, loose to medium dense, damp, dark brown, abundant roots, (surface erosion concrete block).<br>**Artificial Fill (Qaf)** | SM | | | | | | | | | |
| 5 | ░ | | | | SANDY CLAY to CLAYEY SAND, fine grained, hard, dense, damp, yellowish brown, well cemented.<br>**Scripps Formation (Tsc)** | CL<br>SC | | | | | | | | | |
| 6<br>7<br>8<br>9<br>10 | | | | | Bottom of Excavation at 5.0 ft. (refusal on dense soil)<br>No Groundwater, No Caving, Backfilled with Cuttings | | | | | | | | | | |

▦ FOOTING
▼ PERCHED WATER TABLE
☒ BULK BAG SAMPLE
1 IN-PLACE SAMPLE
■ MODIFIED CALIFORNIA SAMPLE
H IN-PLACE HAND-DRIVE SAMPLE
▒ STANDARD PENETRATION TEST

* DISTURBED BLOWCOUNT

**JOB NUMBER:** 22-14083

**LOG NO. HP-3**

**JOB NAME:**
Cunningham-Shaw ADU

**SITE LOCATION:**
2734 Bordeaux Ave.
La Jolla, CA

**FIGURE NO. IIIc**

**Geotechnical Exploration, Inc.**

**EQUIPMENT: Hand Tools**

**DIMENSION & TYPE OF EXCAVATION:** 3'x3'x3'

**DATE LOGGED: December 2, 2022**

**LOGGED BY: HE**

**SURFACE ELEVATION:** ± 323' Above Mean Sea Level

**REVIEWED BY:**

**GROUNDWATER/SEEPAGE DEPTH:** Not Encountered

| DEPTH (feet) | SYMBOL | SAMPLE | FTG. DEPTH | BLOWS / 6" | FIELD DESCRIPTION AND CLASSIFICATION<br>DESCRIPTION AND REMARKS (Grain Size, Density, Moisture, Color) | U.S.C.S | IN-PLACE MOISTURE (%) | IN-PLACE DRY DENSITY (pcf) | OPTIMUM MOISTURE (%) | MAXIMUM DRY DENSITY (pcf) | DENSITY (% of MDD) | EXPAN (+%) CONSOL (-%) | EXPANSION INDEX | % PASSING #200 SIEVE | SAMPLE O.D. (in) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1<br>2 | | | | | SILTY SAND, fine grained, loose to medium dense, dry, light brown, abundant roots and some rock.<br>**Topsoil** | SM | | | | | | | | | |
| 3<br>4 | | | | | SANDY CLAY to CLAYEY SAND, fine grained, hard, dense, damp, yellowish brown well cemented.<br>**Scripps Formation (Tsc)** | CL<br>SC | | | | | | | | | |
| 5<br>6<br>7<br>8<br>9<br>10 | | | | | Bottom of Excavation at 4.0 ft.<br>No Groundwater, No Caving, Backfilled with Cuttings | | | | | | | | | | |

| | |
|---|---|
| ▨ | FOOTING |
| ▼ | PERCHED WATER TABLE |
| ☒ | BULK BAG SAMPLE |
| 1 | IN-PLACE SAMPLE |
| ■ | MODIFIED CALIFORNIA SAMPLE |
| H | IN-PLACE HAND-DRIVE SAMPLE |
| ▨ | STANDARD PENETRATION TEST |

* **DISTURBED BLOWCOUNT**

**JOB NUMBER:** 22-14083

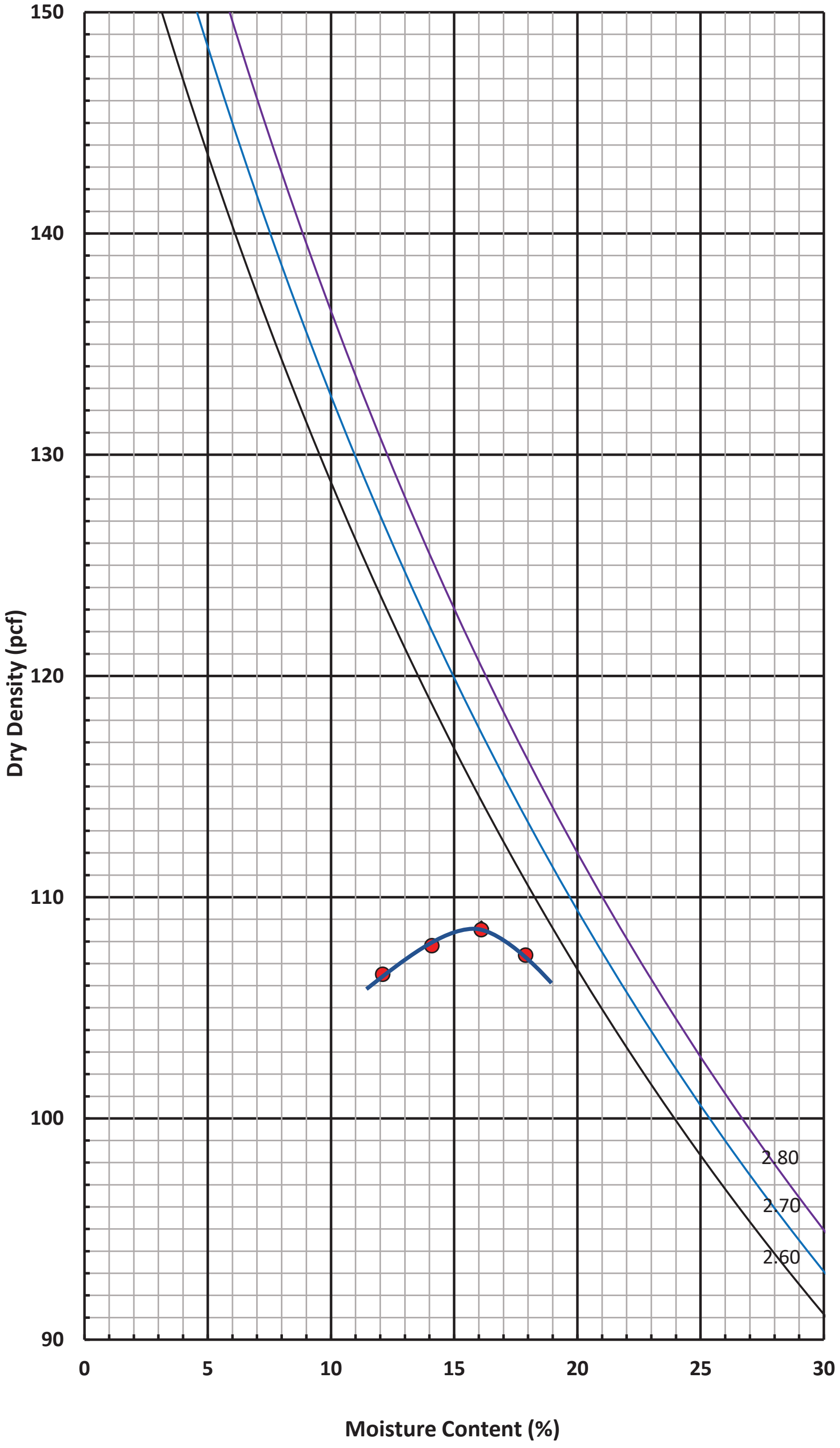
**LOG NO. HP-4**

**JOB NAME:**
Cunningham-Shaw ADU

**SITE LOCATION:**
2734 Bordeaux Ave.
La Jolla, CA

**FIGURE NO. IIId**

# Compaction Curve



| | |
|---|---|
| Source of Material: | **HP-1** |
| Depth: | **1 - 2 ft.** |
| Description of Material: | **Clayey Sand to Sandy Clay (SC-CL)** |
| | **Yellow Brown** |
| Test Method: | **ASTM D1557 Method A** |

**TEST RESULTS**

| | |
|---|---|
| Maximum Dry Density (PCF) | **108.6** |
| Optimum Water Content (%) | **16.1** |
| | |
| Expansion Index (EI) | **21** |
| % Passing #200 | **70** |

**ROCK CORRECTION**

| | |
|---|---|
| Coarse Material (%) | **0.0** |
| Corrected Maximum Dry Density (PCF) | **0.0** |
| Corrected Optimum Water Content (%) | **0.0** |

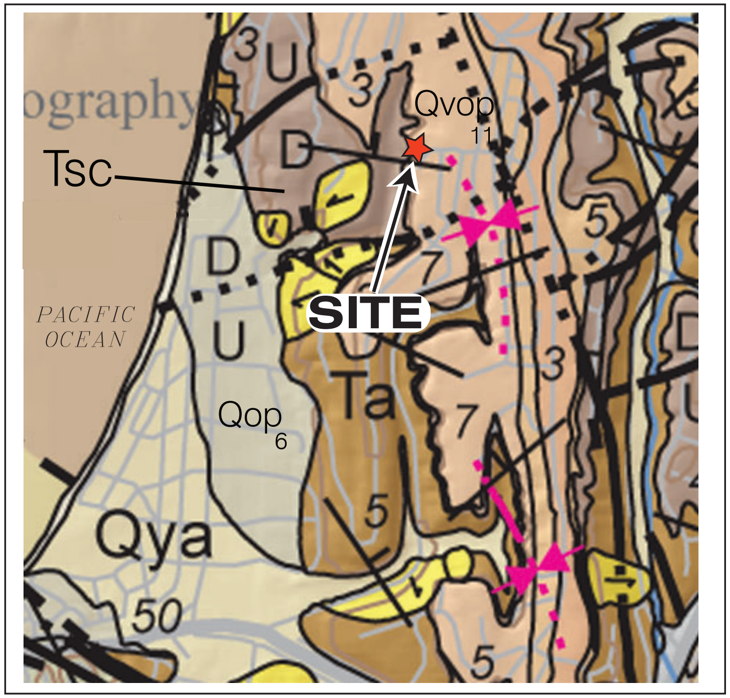
Curves of 100% Saturation
for Specific Gravity Equal to:

2.80
2.70
2.60

**Geotechnical Exploration, Inc.**

## MOISTURE-DENSITY RELATIONSHIP

Figure Number: IV
Cunningham-Shaw ADU
2734 Bordeaux Ave.La Jolla, CA
Job No. 22-14083



Cunningham/Shaw ADU
2734 Bordeaux Avenue
La Jolla, CA.

Base Map

Onshore base (hypsography, hydrography, and transportation) from U.S.G.S. digital line graph (DLG) data, San Diego 30' x 60' metric quadrangle. Shaded topographic base from U.S.G.S. digital elevation models (DEM's). Offshore bathymetric contours and shaded bathymetry from N.O.A.A. single and multibeam data. Projection is UTM, zone 11, North American Datum 1927.

This map was funded in part by the U.S. Geological Survey National Cooperative Geologic Mapping Program, STATEMAP Award no. 98HQAG2049.

Prepared in cooperation with the U.S. Geological Survey, Southern California Areal Mapping Project.

# EXCERPT FROM
# GEOLOGIC MAP OF THE SAN DIEGO 30' x 60' QUADRANGLE, CALIFORNIA

By

Michael P. Kennedy[1] and Siang S. Tan[1]

2008

Digital preparation by

Kelly R. Bovard[2], Anne G. Garcia[2], Diane Burns[2], and Carlos I. Gutierrez[1]

1. Department of Conservation, California Geological Survey
2. U.S. Geological Survey, Department of Earth Sciences, University of California, Riverside

## ONSHORE MAP SYMBOLS

| | |
|---|---|
| | Contact - Contact between geologic units; dotted where concealed. |
| 70 U D | Fault - Solid where accurately located; dashed where approximately located; dotted where concealed. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane. |
| | Anticline - Solid where accurately located; dashed where approximately located; dotted where concealed. Arrow indicates direction of axial plunge. |
| | Syncline - Solid where accurately located; dotted where concealed. Arrow indicates direction of axial plunge. |
| Qls? | Landslide - Arrows indicate principal direction of movement. Queried where existence is questionable. |

Strike and dip of beds

| | |
|---|---|
| 70 | Inclined |

Strike and dip of igneous joints

| | |
|---|---|
| 60 | Inclined |
| | Vertical |

Strike and dip of metamorphic foliation

| | |
|---|---|
| 55 | Inclined |

## DESCRIPTION OF MAP UNITS

| | |
|---|---|
| Qls | Landslide - Arrows indicate principal direction of movement.<br>Queried where existence is questionable |
| $Qvop_{11}$ | Point Loma Formation |
| Tsc | Scripps Formation |
| Ta | Ardath Shale Formation |

*Figure No. V*
*Job No. 22-14083*

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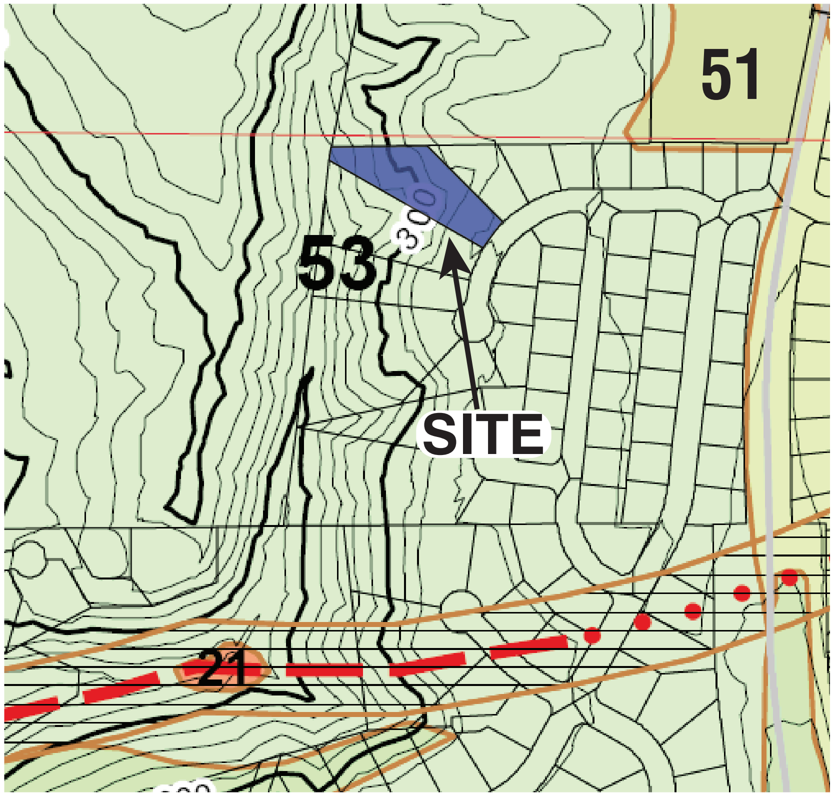
*March 2023*

CUNNINGHAM-GEO.ai

## Geologic Hazards Map Excerpt from City of San Diego Geologic Hazards and Fault Map Sheet 30

Development Services Department

DATE: 4/3/2008



Cunningham/Shaw ADU
2734 Bordeaux Avenue
La Jolla, CA.

## *LEGEND*

## *Geologic Hazard Categories*

***FAULT ZONES***

- ▨ 11 Active, Alquist-Priolo Earthquake Fault Zone
- ▤ 12 Potentially Active, Inactive, Presumed Inactive, or Activity Unknown
- ■ 13 Downtown special fault zone

***LANDSLIDES***

- ■ 21 Confirmed, known, or highly suspected
- ■ 22 Possible or conjectured

***SLIDE-PRONE FORMATIONS***

- ■ 23 Friars: neutral or favorable geologic structure
- ■ 24 Friars: unfavorable geologic structure
- ■ 25 Ardath: neutral or favorable geologic structure
- ■ 26 Ardath: unfavorable geologic structure
- ■ 27 Otay, Sweetwater, and others

***LIQUEFACTION***

- ■ 31 High Potential -- shallow groundwater major drainages, hydraulic fills
- ■ 32 Low Potential -- fluctuating groundwater minor drainages

***COASTAL BLUFFS***

- ■ 41 Generally unstable Numerous landslides, high steep bluffs, severe erosion, unfavorable geologic structure
- ■ 42 Generally unstable Unfavorable bedding plains, high erosion
- ■ 43 Generally unstable Unfavorable jointing, local high erosion
- ■ 44 Moderately stable Mostly stable formations, local high erosion
- ■ 45 Moderately stable Some minor landslides, minor erosion
- ■ 46 Moderately stable Some unfavorable geologic structure, minor or no erosion
- ■ 47 Generally stable Favorable geologic structure, minor or no erosion, no landslides
- ■ 48 Generally stable Broad beach areas, developed harbor

***OTHER TERRAIN***

- ■ 51 Level mesas -- underlain by terrace deposits and bedrock nomimal risk
- ■ 52 Other level areas, gently sloping to steep terrain, favorable geologic structure, Low risk
- ■ 53 Level or sloping terrain, unfavorable geologic structure, Low to moderate risk
- ■ 54 Steeply sloping terrain, unfavorable or fault controlled geologic structure, Moderate risk
- ■ 55 Modified terrain (graded sites) Nominal risk

***Water (Bays and Lakes)***

- ■

***FAULTS***

- 〰 Fault
- ⌃⌃ Inferred Fault
- ∴ Concealed Fault
- ⩘ Shear Zone

*Figure No. VI*
*Job No. 22-14083*

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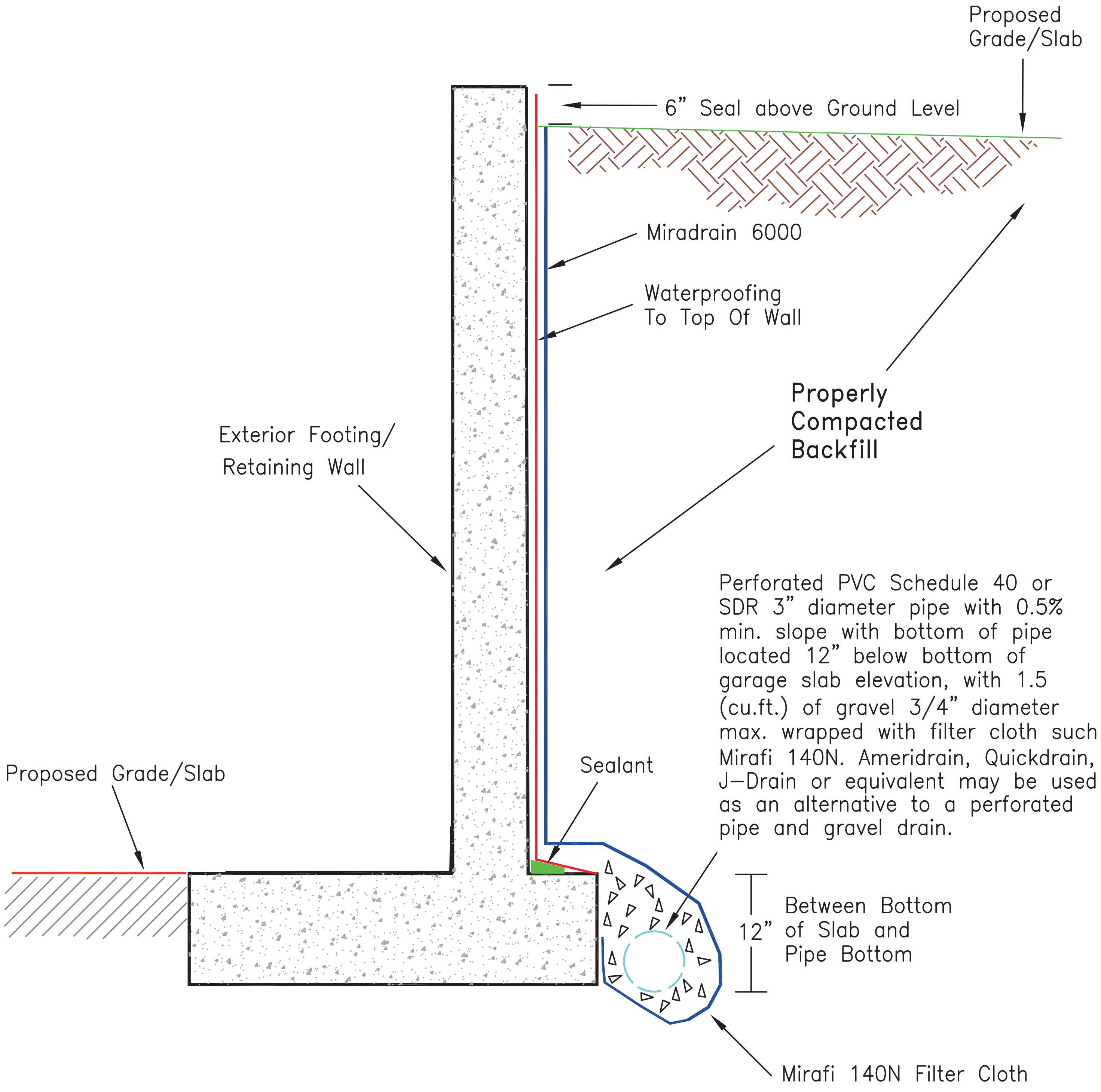
*March 2023*

cunningham-seis.ai

# RETAINING WALL
# DRAINAGE RECOMMENDATIONS



Perforated PVC Schedule 40 or SDR 3" diameter pipe with 0.5% min. slope with bottom of pipe located 12" below bottom of garage slab elevation, with 1.5 (cu.ft.) of gravel 3/4" diameter max. wrapped with filter cloth such Mirafi 140N. Ameridrain, Quickdrain, J–Drain or equivalent may be used as an alternative to a perforated pipe and gravel drain.

**NOT TO SCALE**

*Figure No. VII*
*Job No. 22-14083*

NOTE: As an option to Miradrain 6000, gravel or crushed rock 3/4" maximu diameter may be used with a minimum 12" thickness along the exterior foace of the wall and 2.0 cu/ft of pipe.

**Geotechnical Exploration, Inc.**

*March 2023*

# APPENDIX A

## UNIFIED SOIL CLASSIFICATION CHART
## SOIL DESCRIPTION

**Coarse-grained (More than half of material is larger than a No. 200 sieve)**

| | | |
|---|---|---|
| GRAVELS, CLEAN GRAVELS (More than half of coarse fraction is larger than No. 4 sieve size, but smaller than 3”) | GW | Well-graded gravels, gravel and sand mixtures, little or no fines. |
| | GP | Poorly graded gravels, gravel and sand mixtures, little or no fines. |
| GRAVELS WITH FINES (Appreciable amount) | GC | Clay gravels, poorly graded gravel-sand-silt mixtures |
| SANDS, CLEAN SANDS (More than half of coarse fraction is smaller than a No. 4 sieve) | SW | Well-graded sand, gravelly sands, little or no fines |
| | SP | Poorly graded sands, gravelly sands, little or no fines. |
| SANDS WITH FINES (Appreciable amount) | SM | Silty sands, poorly graded sand and silty mixtures. |
| | SC | Clayey sands, poorly graded sand and clay mixtures. |

**Fine-grained (More than half of material is smaller than a No. 200 sieve)**

SILTS AND CLAYS

| | | |
|---|---|---|
| *Liquid Limit Less than 50* | ML | Inorganic silts and very fine sands, rock flour, sandy silt and clayey-silt sand mixtures with a slight plasticity |
| | CL | Inorganic clays of low to medium plasticity, gravelly clays, silty clays, clean clays. |
| | OL | Organic silts and organic silty clays of low plasticity. |
| *Liquid Limit Greater than 50* | MH | Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts. |
| | CH | Inorganic clays of high plasticity, fat clays. |
| | OH | Organic clays of medium to high plasticity. |
| HIGHLY ORGANIC SOILS | PT | Peat and other highly organic soils |

## APPENDIX B

### *REGIONAL GEOLOGIC DESCRIPTION*

The Peninsular Range forms the granitic spine of San Diego County. These rocks are primarily plutonic, forming at depth beneath the earth's crust 140 to 90 million years ago as the result of the subduction of an oceanic crustal plate beneath the North American continent. These rocks formed the much larger Southern California batholith. Metamorphism associated with the intrusion of these great granitic masses affected the much older sediments that existed near the surface over that period of time. These metasedimentary rocks remain as roof pendants of marble, schist, slate, quartzite and gneiss throughout the Peninsular Ranges. Locally, Miocene-age volcanic rocks and flows have also accumulated within these mountains (e.g., Jacumba Valley). Regional tectonic forces and erosion over time have uplifted and unroofed these granitic rocks to expose them at the surface (Demere, 1997).

The Salton Trough is the northerly extension of the Gulf of California. This zone is undergoing active deformation related to faulting along the Elsinore and San Jacinto Fault Zones, which are part of the major regional tectonic feature in the southwestern portion of California, the San Andreas Fault Zone. Translational movement along these fault zones has resulted in crustal rifting and subsidence. The Salton Trough, also referred to as the Colorado Desert, has been filled with sediments to depth of approximately 5 miles since the movement began in the early Miocene, 24 million years ago. The source of these sediments has been the local mountains as well as the ancestral and modern Colorado River (Demere, 1997).

The San Diego area is part of a seismically active region of California. It is on the eastern boundary of the Southern California Continental Borderland, part of the Peninsular Ranges Geomorphic Province. This region is part of a broad tectonic boundary between the North American and Pacific Plates. The actual plate boundary is characterized by a complex system of active, major, right-lateral strike-slip faults, trending northwest/southeast. This fault system extends eastward to the San Andreas Fault (approximately 70 miles from San Diego) and westward to the San Clemente Fault (approximately 50 miles off-shore from San Diego) (Berger and Schug, 1991).

In California, major earthquakes can generally be correlated with movement on active faults. As defined by the California Division of Mines and Geology, now the California Geological Survey (CGS), an "active" fault, described by CGS (2018) as a Holocene-Active fault, is one that has had (ground) surface displacement within Holocene time, the last 11,700. In addition, "potentially active fault" has been amended to Pre-Holocene fault: a fault whose recency of past movement is older than 11,700 years, and thus does not meet the criteria of Holocene-Active fault as defined in the State Mining and Geology Board regulations.

A three-tier fault classification is used as follows:

- *Holocene-Active Faults* have surface displacement within Holocene time, where Holocene time is the geological epoch that began 11,700 years before present.
- *Pre-Holocene Faults* have demonstrable displacement older than Holocene time.
- *Age-Undetermined Faults* are faults whose age of most recent movement is not known or is unconstrained by dating methods or by limitations in stratigraphic resolution.

During recent history, prior to April 2010, the San Diego County area has been relatively quiet seismically. The youngest paleoearthquake that cuts the early historical living surface is likely the 1862 San Diego earthquake that had an estimated magnitude of M6 (Legg and Agnew, 1979; Singleton et al., 2019). Paleoseismic trenches at the Presidio Hills Golf Course on the main trace of the Rose Canyon Fault contained evidence for historical ground rupturing earthquakes as recently as 1862 and the mid-1700s. Results of the study also suggest the Rose Canyon Fault has a ~700-800-year recurrence interval (Singleton et al., 2019).

On June 15, 2004, a M5.3 earthquake occurred approximately 45 miles southwest of downtown San Diego (26 miles west of Rosarito, Mexico). Another widely felt earthquake on a distant southern California fault was a M5.4 event that took place on July 29, 2008, west-southwest of the Chino Hills area of Riverside County.

Several earthquakes ranging from M5.0 to M6.0 occurred in northern Baja California, centered in the Gulf of California on August 3, 2009. A M5.8 earthquake followed by a M4.9 aftershock occurred on December 30, 2009, centered about 20 miles south of the Mexican border city of Mexicali.

On April 04, 2010, a large earthquake occurred in Baja California, Mexico. It was widely felt throughout the southwest including Phoenix, Arizona and San Diego in California. This M7.2 event, the Sierra El Mayor earthquake, occurred in northern Baja California, approximately 40 miles south of the Mexico-USA border at shallow depth along the principal plate boundary between the North American and Pacific plates. According to the U.S. Geological Survey this is an area with a high level of historical seismicity, and it has recently also been seismically active, although this is the largest event to strike in this area since 1892. The April 04, 2010, earthquake appears to have been larger than the M6.9 earthquake in 1940 or any of the early 20th century events (e.g., 1915 and 1934) in this region of northern Baja California.

This event's aftershock zone extends significantly to the northwest, overlapping with the portion of the fault system that is thought to have ruptured in 1892. Ground motions for the April 04, 2010, main event, recorded at stations in San Diego and reported by the California Strong Motion Instrumentation Program (CSMIP), ranged up to 0.058g.

On July 07, 2010, a M5.4 earthquake occurred in Southern California at 4:53 pm (Pacific Time) about 30 miles south of Palm Springs, 25 miles southwest of Indio, and 13 miles north-northwest of Borrego Springs. The earthquake occurred near the Coyote Creek segment of the San Jacinto Fault. The earthquake exhibited right lateral slip to the northwest, consistent with the direction of movement on the San Jacinto Fault. It was followed by more than 60 aftershocks of M1.3 and greater during the first hour.

In the last 50 years, there have been four other earthquakes in the magnitude M5.0 range within 20 kilometers of the Coyote Creek segment: M5.8 in 1968, M5.3 on 2/25/1980, M5.0 on 10/31/2001, and M5.2 on 6/12/2005. The biggest earthquake near this location was the M6.0 Buck Ridge earthquake on 3/25/1937.

# *APPENDIX C*

## *SLOPE STABILITY CALCULATIONS WITH SLIDE 6 COMPUTER PROGRAM*

*Proposed Cunningham/Shaw ADU*

***Job No. 22-14083***

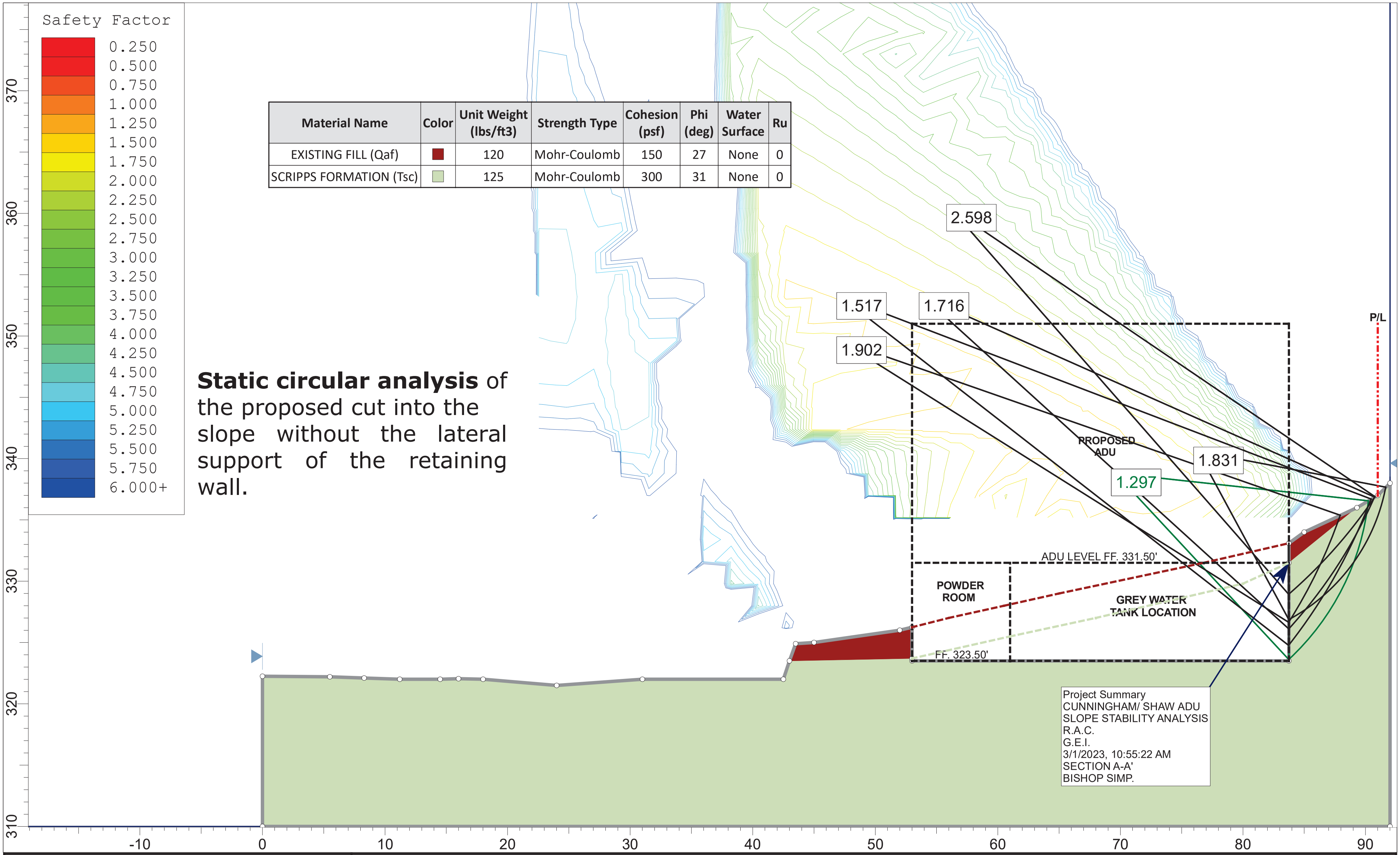
We performed gross slope stability calculations using the *SLIDE 6* program by Roc Science. The program is a limit equilibrium method, slope stability program that allows the use of several slope stability methods to calculate the factors of safety against shear failure. On this project the Bishop Simplified method was used as the basis for calculations when using circular slide surfaces for analysis through the site geologic cross sections.

The program calculates the factor of safety against shear failure for potential slide surfaces over a selected range. We chose the range of slide surfaces where failures are most likely to occur. The printout shows a block with contours of different colors and shades that correspond to the different factors of safety calculated that can be obtained for the analyzed range of slide surfaces for Section A-A′, B-B′ and E-E′, which include the most unfavorable slope conditions at the site (see attached printouts). The green circular surface and value displayed in the printout is the lowest possible factor of safety located within the specified search range of each analysis. Soil strength values, geometry, and water conditions (seepage was not encountered) used in the program were based on geological information at the site, obtained by our project geologist. Direct shear test results from the on-site soils were performed and were used for the gross slope stability analysis. Shear strength values were conservatively adjusted.

The static gross slope stability factors of safety were calculated and yielded a factor of safety value above 1.50 once the lateral earth pressure due the basement retaining walls were included.

Once the static gross stability was determined, a seismic analysis was performed for the same analyzed sections. The seismic analysis yielded a factor of safety value above 1.15 as required by the City of San Diego and the State of California.

The surficial slope stability calculations were performed on the slope face using a geotechnical accepted equation for infinite slopes with a saturated upper layer. The calculations were performed by assuming that depth of soils was saturated down to either 1 meter (3.28 feet) or the depth to the firm bedrock. It is our professional



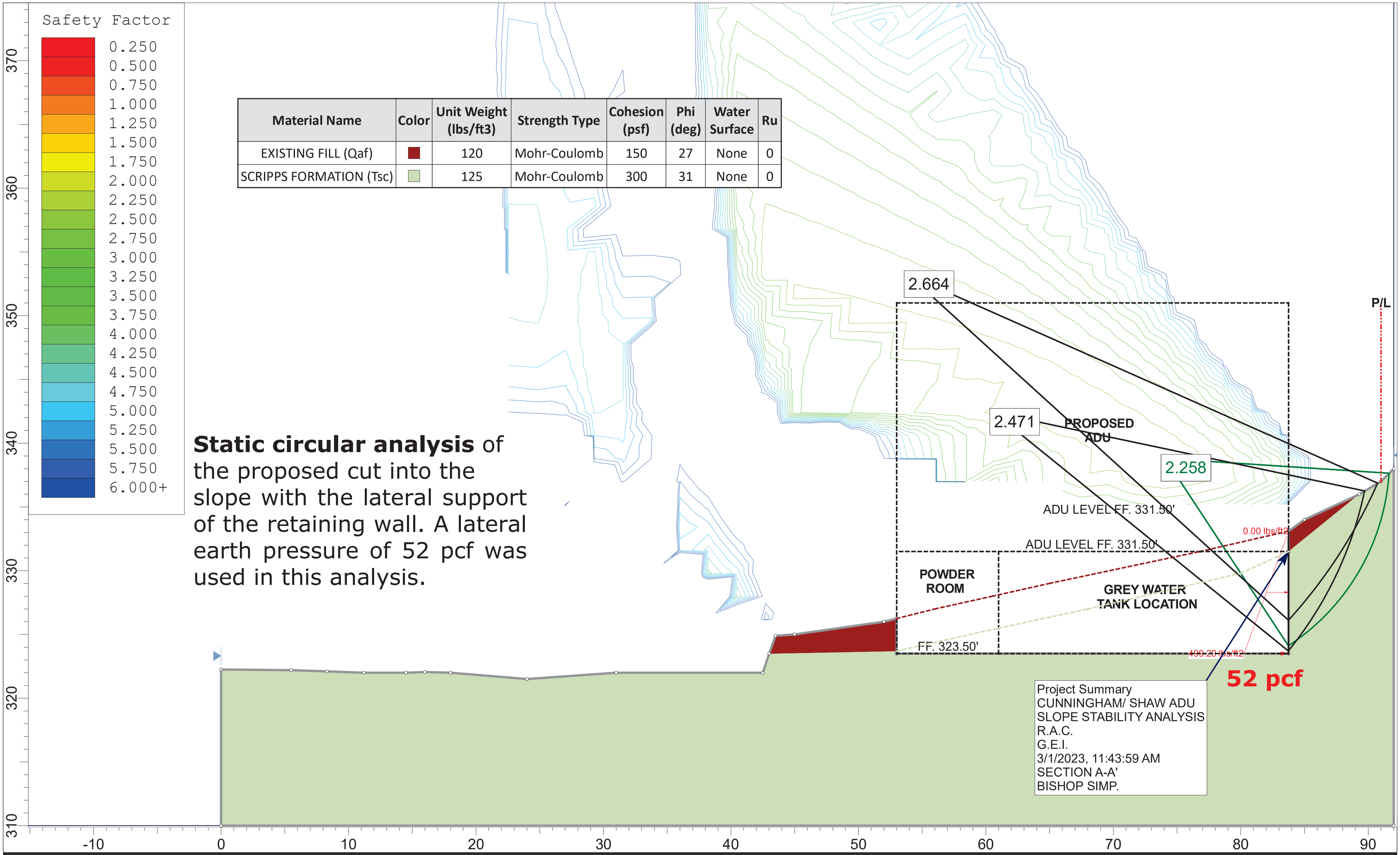
| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| EXISTING FILL (Qaf) | 🟥 | 120 | Mohr-Coulomb | 150 | 27 | None | 0 |
| SCRIPPS FORMATION (Tsc) | 🟩 | 125 | Mohr-Coulomb | 300 | 31 | None | 0 |

**Static circular analysis** of the proposed cut into the slope without the lateral support of the retaining wall.

**Geotechnical Exploration, Inc.**

SLIDEINTERPRET 6.039

| | | | |
|---|---|---|---|
| *Project* | CUNNINGHAM/ SHAW ADU | | **SECTION A-A'** |
| *Analysis Description* | SLOPE STABILITY ANALYSIS | | |
| *Drawn By* R.A.C. | *Scale* 1:130 | *Company* G.E.I. | |
| *Date* 3/1/2023, 10:55:22 AM | | *File Name* JOB NO. 22-14083_S(A)_01.slim | |



| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| EXISTING FILL (Qaf) | 🟥 | 120 | Mohr-Coulomb | 150 | 27 | None | 0 |
| SCRIPPS FORMATION (Tsc) | 🟩 | 125 | Mohr-Coulomb | 300 | 31 | None | 0 |

**Static circular analysis** of the proposed cut into the slope with the lateral support of the retaining wall. A lateral earth pressure of 52 pcf was used in this analysis.

Project Summary
CUNNINGHAM/ SHAW ADU
SLOPE STABILITY ANALYSIS
R.A.C.
G.E.I.
3/1/2023, 11:43:59 AM
SECTION A-A'
BISHOP SIMP.

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| | |
|---|---|
| *Project* | CUNNINGHAM/ SHAW ADU |
| | **SECTION A-A'** |
| *Analysis Description* | SLOPE STABILITY ANALYSIS |
| *Drawn By* | R.A.C. |
| *Scale* | 1:125 |
| *Company* | G.E.I. |
| *Date* | 3/1/2023, 11:43:59 AM |
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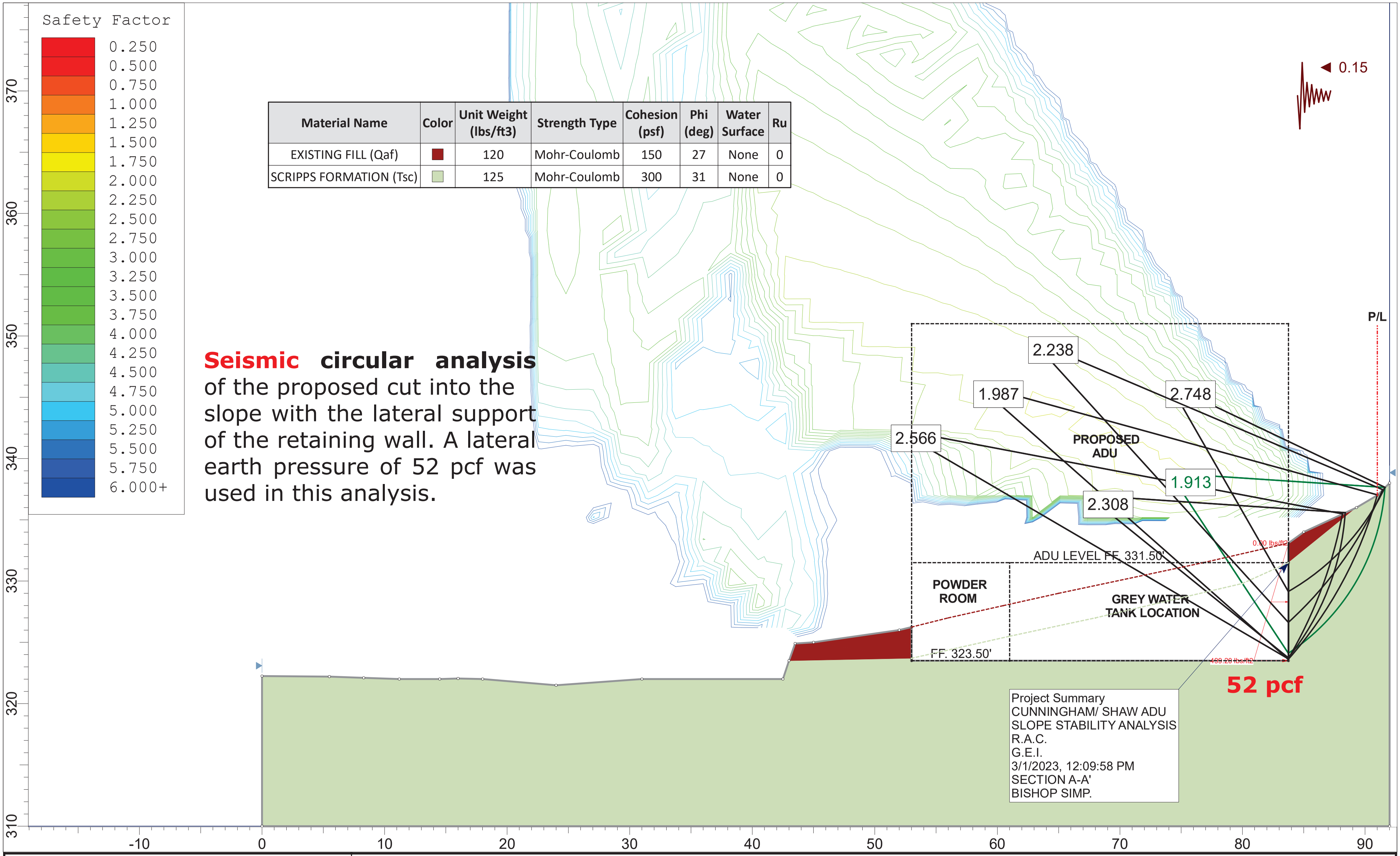

| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| EXISTING FILL (Qaf) | 🟥 | 120 | Mohr-Coulomb | 150 | 27 | None | 0 |
| SCRIPPS FORMATION (Tsc) | 🟩 | 125 | Mohr-Coulomb | 300 | 31 | None | 0 |

**Seismic circular analysis** of the proposed cut into the slope with the lateral support of the retaining wall. A lateral earth pressure of 52 pcf was used in this analysis.

Project Summary
CUNNINGHAM/ SHAW ADU
SLOPE STABILITY ANALYSIS
R.A.C.
G.E.I.
3/1/2023, 12:09:58 PM
SECTION A-A'
BISHOP SIMP.

**Geotechnical Exploration, Inc.**

SLIDEINTERPRET 6.039

| | | | |
|---|---|---|---|
| *Project* CUNNINGHAM/ SHAW ADU | | | **SECTION A-A'** |
| *Analysis Description* SLOPE STABILITY ANALYSIS | | | |
| *Drawn By* R.A.C. | *Scale* 1:130 | *Company* G.E.I. | |
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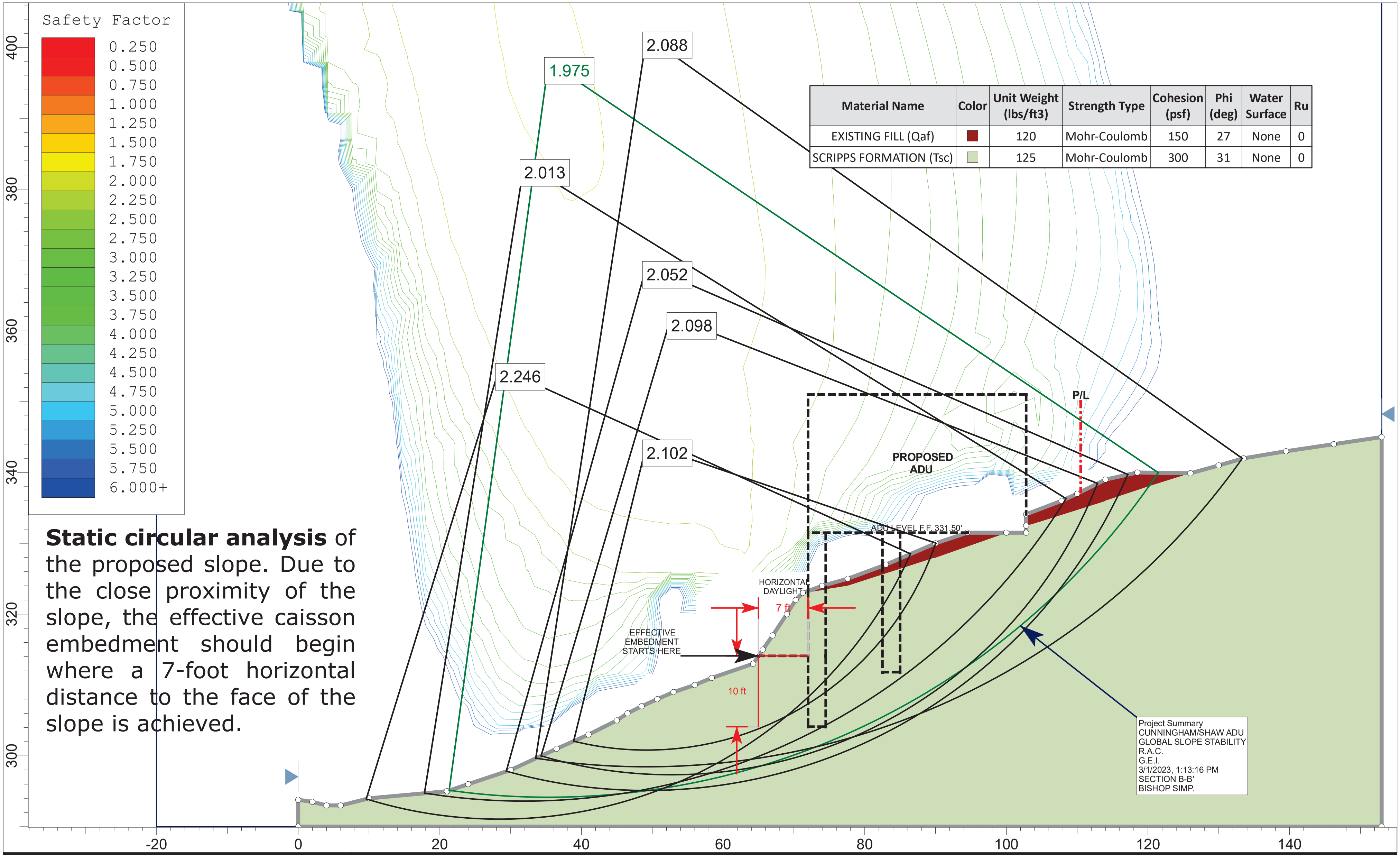

| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| EXISTING FILL (Qaf) | 🟥 | 120 | Mohr-Coulomb | 150 | 27 | None | 0 |
| SCRIPPS FORMATION (Tsc) | 🟩 | 125 | Mohr-Coulomb | 300 | 31 | None | 0 |

**Static circular analysis** of the proposed slope. Due to the close proximity of the slope, the effective caisson embedment should begin where a 7-foot horizontal distance to the face of the slope is achieved.

**Geotechnical Exploration, Inc.**

SLIDEINTERPRET 6.039

| | |
|---|---|
| *Project* | CUNNINGHAM/SHAW ADU **SECTION B-B'** |
| *Analysis Description* | GLOBAL SLOPE STABILITY |
| *Drawn By* | R.A.C. |
| *Scale* | 1:225 |
| *Company* | G.E.I. |
| *Date* | 3/1/2023, 1:13:16 PM |
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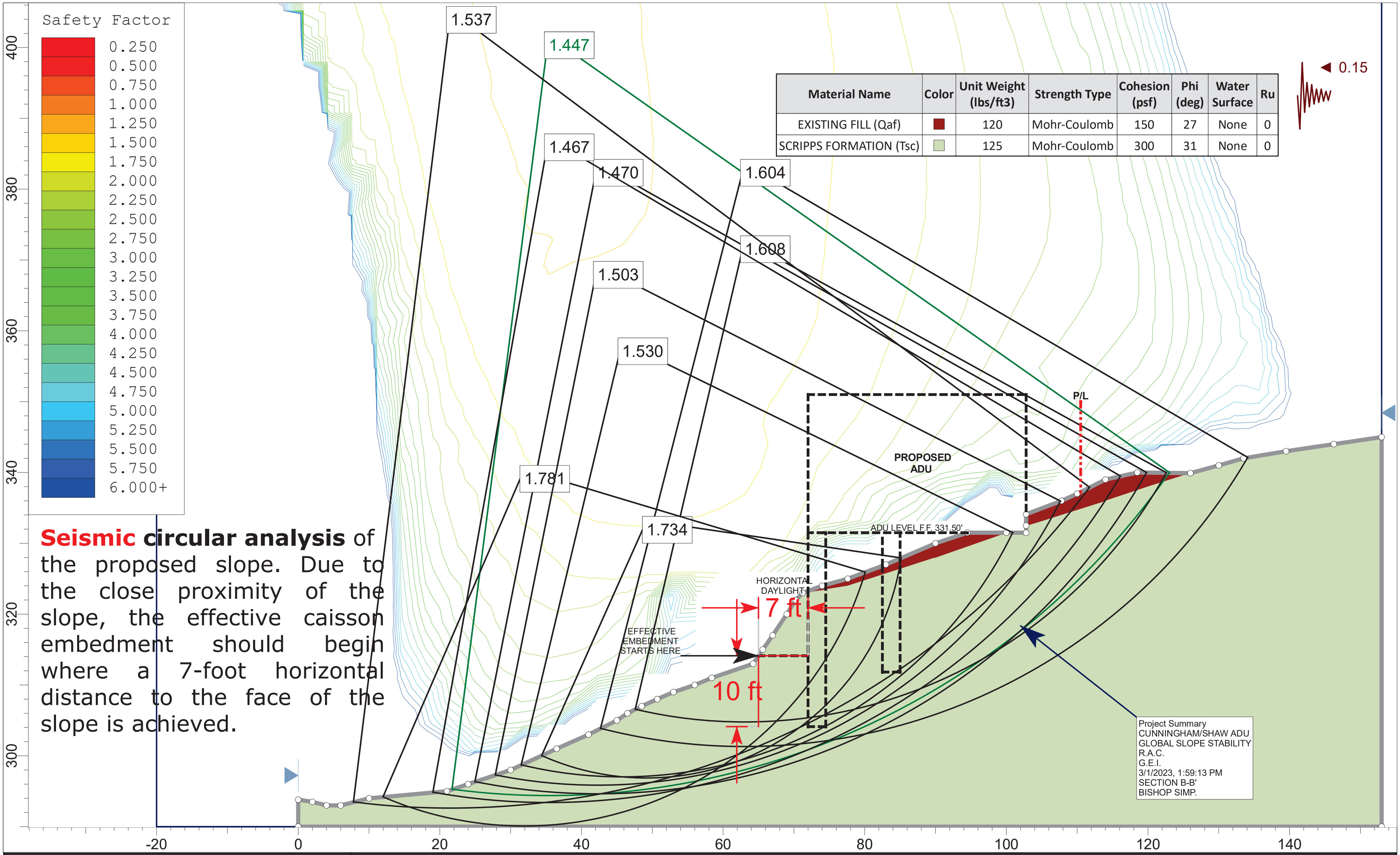


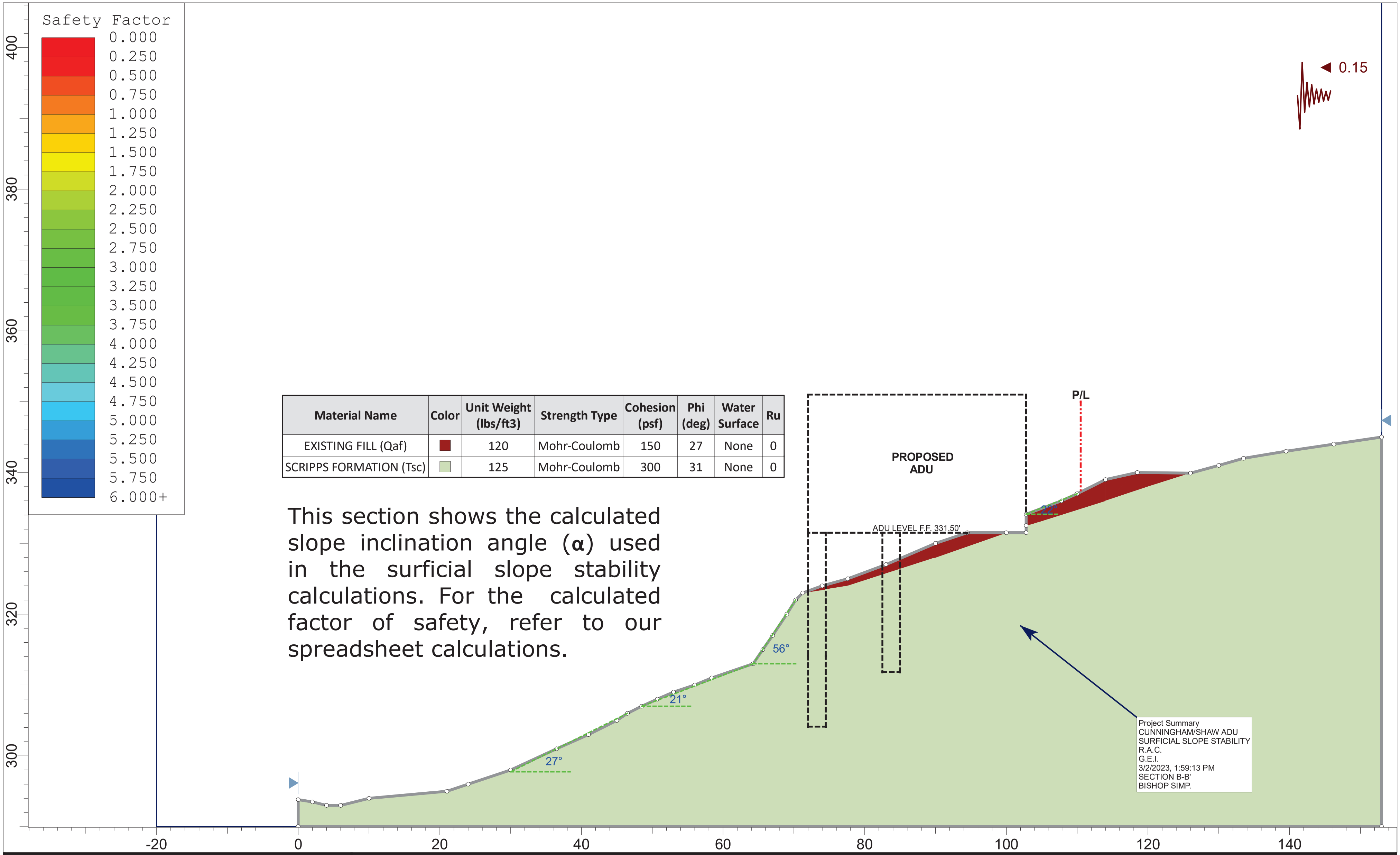
| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| EXISTING FILL (Qaf) | 🟥 | 120 | Mohr-Coulomb | 150 | 27 | None | 0 |
| SCRIPPS FORMATION (Tsc) | 🟩 | 125 | Mohr-Coulomb | 300 | 31 | None | 0 |

**Seismic circular analysis** of the proposed slope. Due to the close proximity of the slope, the effective caisson embedment should begin where a 7-foot horizontal distance to the face of the slope is achieved.

**Geotechnical Exploration, Inc.**

SLIDEINTERPRET 6.039

| | | | |
|---|---|---|---|
| *Project* | CUNNINGHAM/SHAW ADU | | **SECTION B-B'** |
| *Analysis Description* | GLOBAL SLOPE STABILITY | | |
| *Drawn By* | R.A.C. | *Scale* | 1:225 |
| *Company* | G.E.I. | | |
| *Date* | 3/1/2023, 1:59:13 PM | *File Name* | JOB NO. 22-14083_S(B)_02.slim |



| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| EXISTING FILL (Qaf) | 🟥 | 120 | Mohr-Coulomb | 150 | 27 | None | 0 |
| SCRIPPS FORMATION (Tsc) | 🟩 | 125 | Mohr-Coulomb | 300 | 31 | None | 0 |

This section shows the calculated slope inclination angle (α) used in the surficial slope stability calculations. For the calculated factor of safety, refer to our spreadsheet calculations.

**Geotechnical Exploration, Inc.**

SLIDEINTERPRET 6.039

*Project* CUNNINGHAM/SHAW ADU

## SECTION B-B'

*Analysis Description* SURFICIAL SLOPE STABILITY

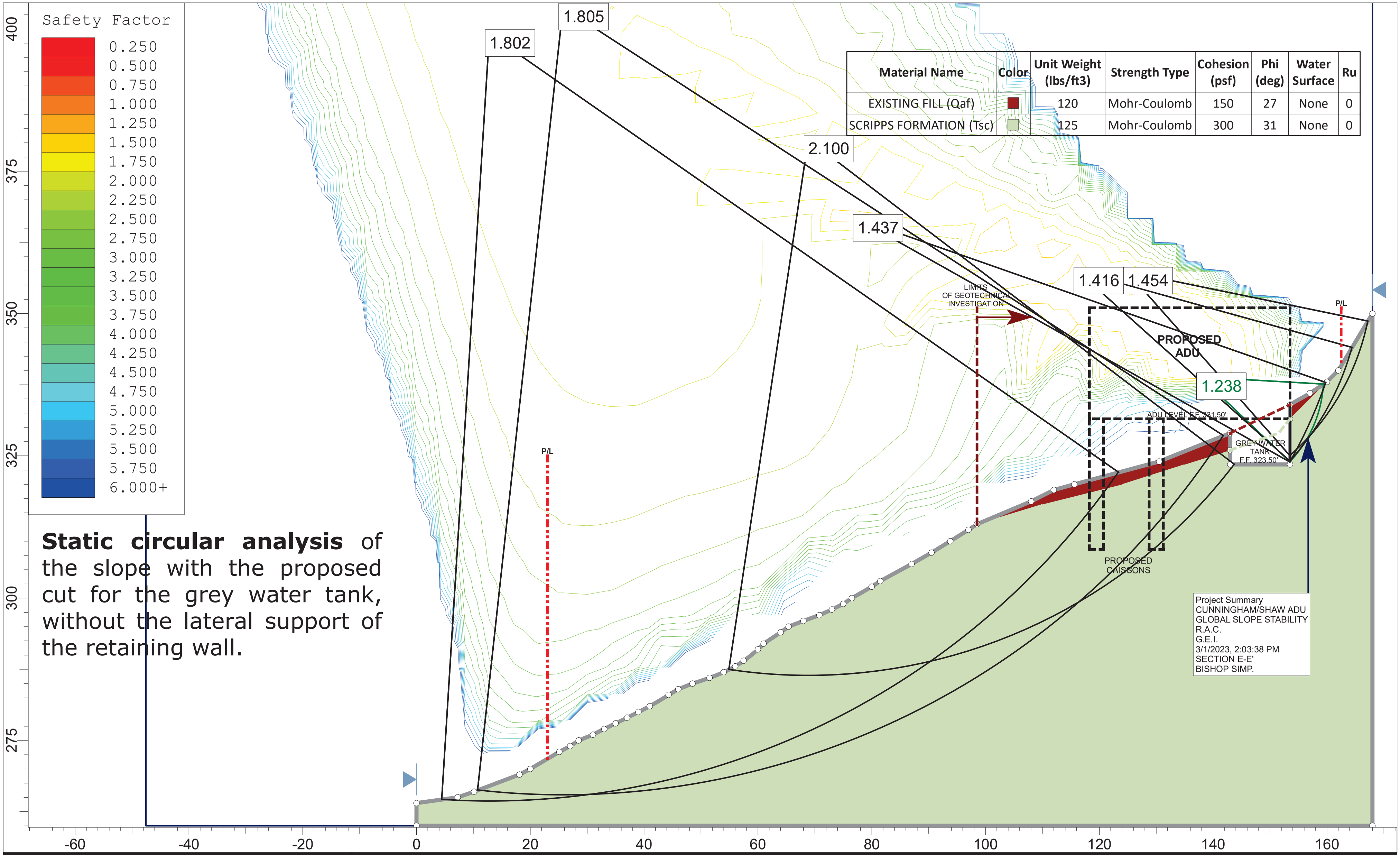
*Drawn By* R.A.C.

*Scale* 1:225

*Company* G.E.I.

*Date* 3/2/2023, 1:59:13 PM

*File Name* JOB NO. 22-14083_S(B)_03.slim



| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| EXISTING FILL (Qaf) | 🟥 | 120 | Mohr-Coulomb | 150 | 27 | None | 0 |
| SCRIPPS FORMATION (Tsc) | 🟩 | 125 | Mohr-Coulomb | 300 | 31 | None | 0 |

**Static circular analysis** of the slope with the proposed cut for the grey water tank, without the lateral support of the retaining wall.

Project Summary
CUNNINGHAM/SHAW ADU
GLOBAL SLOPE STABILITY
R.A.C.
G.E.I.
3/1/2023, 2:03:38 PM
SECTION E-E'
BISHOP SIMP.

**Geotechnical Exploration, Inc.**

SLIDEINTERPRET 6.039

*Project* CUNNINGHAM/SHAW ADU

## SECTION E-E'

*Analysis Description* GLOBAL SLOPE STABILITY

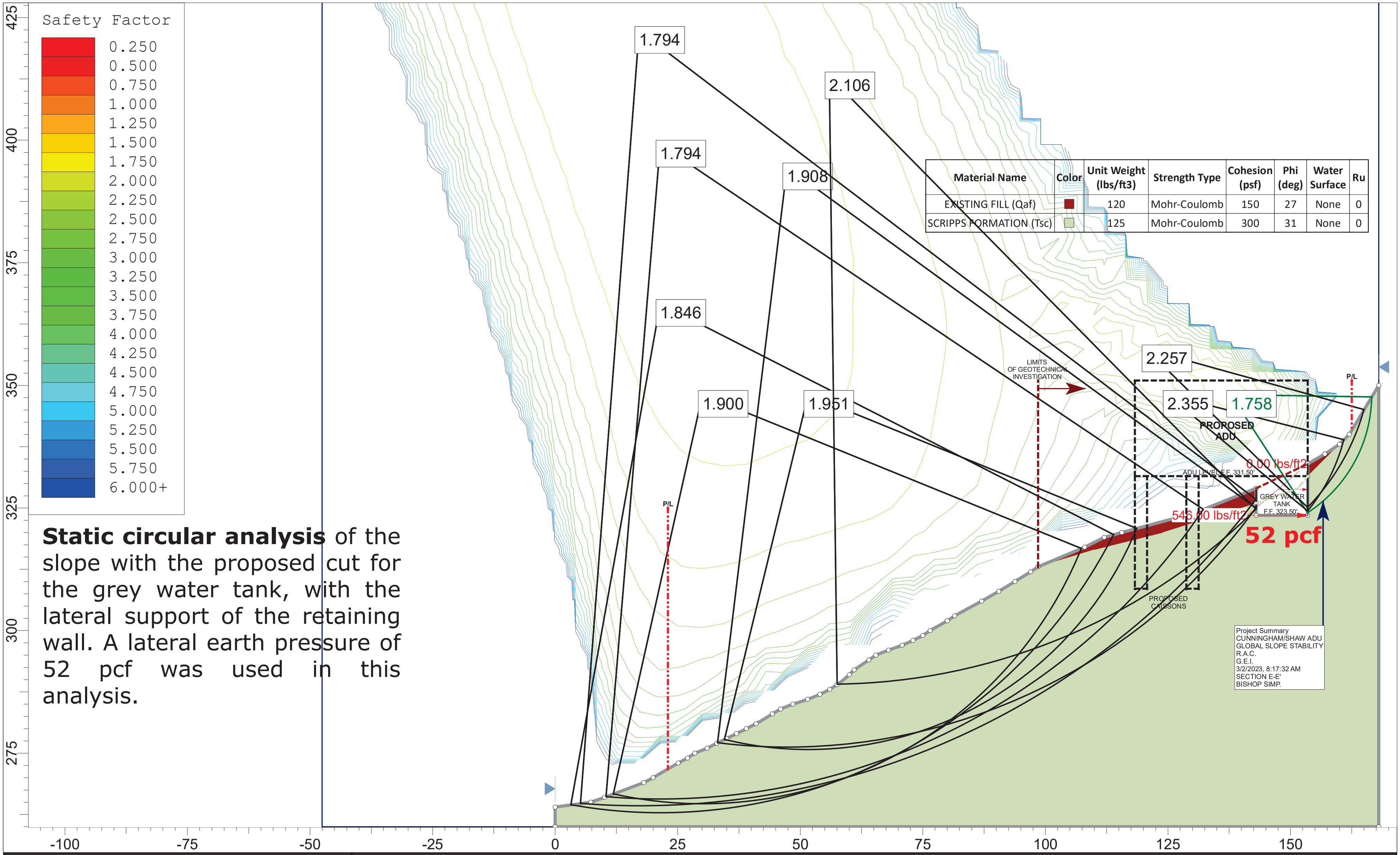
*Drawn By* R.A.C.

*Scale* 1:280

*Company* G.E.I.

*Date* 3/1/2023, 2:03:38 PM

*File Name* JOB NO. 22-14083_S(E)_01.slim



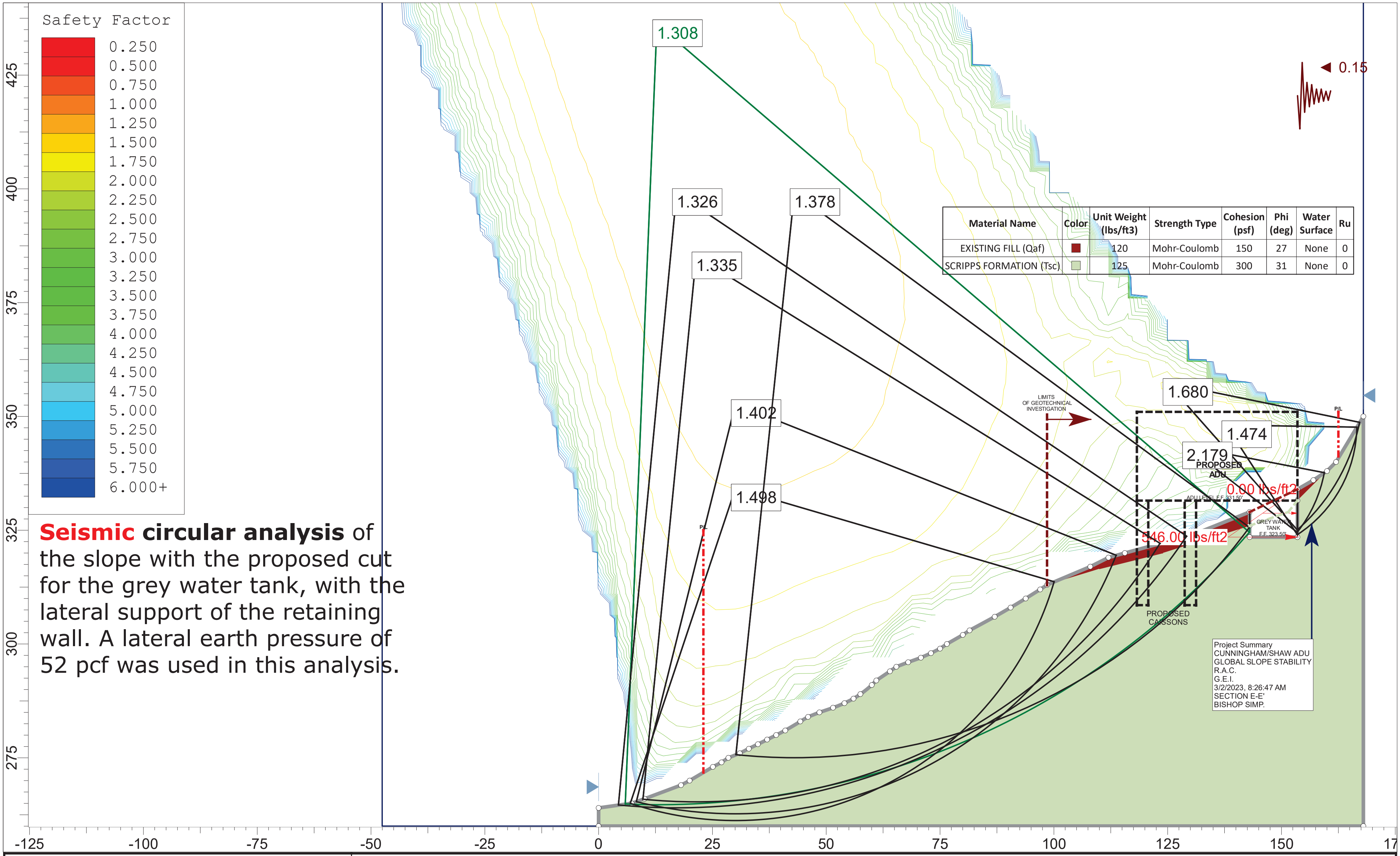
| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| EXISTING FILL (Qaf) | 🟥 | 120 | Mohr-Coulomb | 150 | 27 | None | 0 |
| SCRIPPS FORMATION (Tsc) | 🟩 | 125 | Mohr-Coulomb | 300 | 31 | None | 0 |

**Static circular analysis** of the slope with the proposed cut for the grey water tank, with the lateral support of the retaining wall. A lateral earth pressure of 52 pcf was used in this analysis.

**Geotechnical Exploration, Inc.**

| | |
|---|---|
| *Project* | CUNNINGHAM/SHAW ADU **SECTION E-E'** |
| *Analysis Description* | GLOBAL SLOPE STABILITY |
| *Drawn By* | R.A.C. |
| *Scale* | 1:325 |
| *Company* | G.E.I. |
| *Date* | 3/2/2023, 8:17:32 AM |
| *File Name* | JOB NO. 22-14083_S(E)_02.slim |

SLIDEINTERPRET 6.039



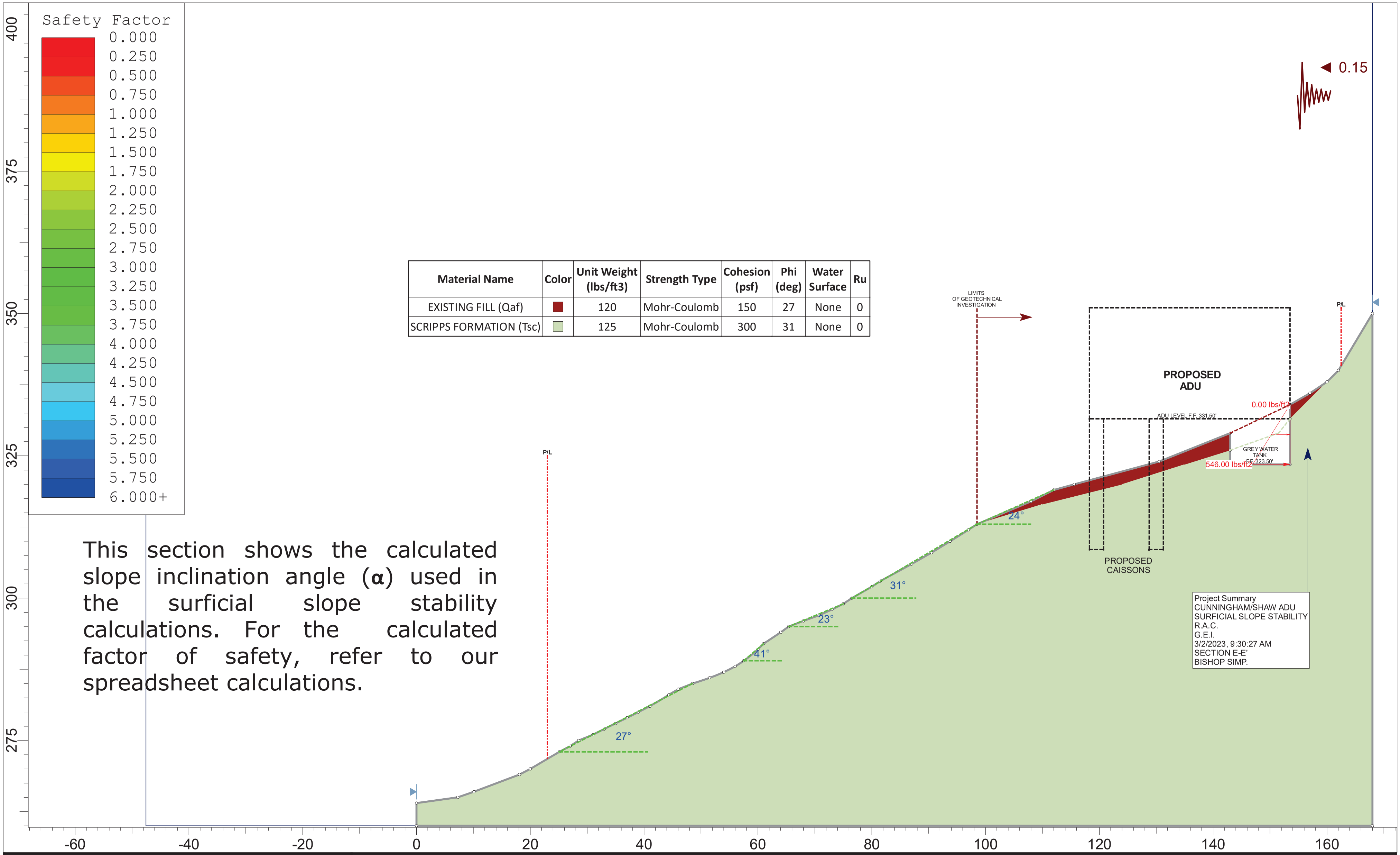
| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| EXISTING FILL (Qaf) | 🟥 | 120 | Mohr-Coulomb | 150 | 27 | None | 0 |
| SCRIPPS FORMATION (Tsc) | 🟩 | 125 | Mohr-Coulomb | 300 | 31 | None | 0 |

**Seismic circular analysis** of the slope with the proposed cut for the grey water tank, with the lateral support of the retaining wall. A lateral earth pressure of 52 pcf was used in this analysis.

**Geotechnical Exploration, Inc.**

SLIDEINTERPRET 6.039

| | | | |
|---|---|---|---|
| *Project* | CUNNINGHAM/SHAW ADU | | **SECTION E-E'** |
| *Analysis Description* | GLOBAL SLOPE STABILITY | | |
| *Drawn By* | R.A.C. | *Scale* | 1:350 |
| *Company* | G.E.I. | | |
| *Date* | 3/2/2023, 8:26:47 AM | *File Name* | JOB NO. 22-14083_S(E)_03.slim |



| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi (deg) | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| EXISTING FILL (Qaf) | 🟥 | 120 | Mohr-Coulomb | 150 | 27 | None | 0 |
| SCRIPPS FORMATION (Tsc) | 🟩 | 125 | Mohr-Coulomb | 300 | 31 | None | 0 |

This section shows the calculated slope inclination angle (α) used in the surficial slope stability calculations. For the calculated factor of safety, refer to our spreadsheet calculations.

**Geotechnical Exploration, Inc.**

SLIDEINTERPRET 6.039

| | | | |
|---|---|---|---|
| *Project* | CUNNINGHAM/SHAW ADU | | **SECTION E-E'** |
| *Analysis Description* | SURFICIAL SLOPE STABILITY | | |
| *Drawn By* | R.A.C. | *Scale* | 1:280 |
| *Company* | G.E.I. | | |
| *Date* | 3/2/2023, 9:30:27 AM | *File Name* | JOB NO. 22-14083_S(E)_04.slim |

## SURFICIAL FAILURE

**EQUATION 1**

$$FOS = \frac{c' + (\gamma_T - \gamma_W) z_W \cos(\alpha)^2 \tan\varphi'}{\gamma_T z_W \sin\alpha \cos\alpha}$$

|  | $\Upsilon_t$ | $\Upsilon_w$ | $\Upsilon'$ | $z_w$ |
|---|---|---|---|---|
|  | pcf | pcf | pcf | ft |
| **FILL ($Q_{af}$)** | 120 | 62.4 | 57.6 | 2 |
| **($T_{sc}$)** | 125 | 62.4 | 62.6 | 3.28 |

SURFICIAL SLOPE STABILITY ANALYSIS IS BASED ON EQUATION (1) FOR THE CALCULATED VALUES. Reference: Abramson L.W., Lee T.S., Sharma S., Boyce G.M., 2002, Slope Stability and Stabilziation Methods, 2nd Edition, John Wiley and Sons, Inc., New York, USA. Pg. 658

**SECTION B-B'**

| SOIL TYPE | c (psf) | ϕ'(°) | α(°) | F.O.S. |
|---|---|---|---|---|
| FILL ($Q_{af}$) | 150 | 27 | 22 | 2.405 |
| SCRIPPS-FORMATION ($T_{sc}$) | 300 | 31 | 56 | 1.781 |
| SCRIPPS-FORMATION ($T_{sc}$) | 300 | 31 | 21 | 2.971 |
| SCRIPPS-FORMATION ($T_{sc}$) | 300 | 31 | 27 | 2.399 |

**SECTION E-E'**

| SOIL TYPE | c (psf) | ϕ'(°) | α(°) | F.O.S. |
|---|---|---|---|---|
| FILL ($Q_{af}$) | 150 | 27 | 24 | 2.231 |
| SCRIPPS-FORMATION ($T_{sc}$) | 300 | 31 | 31 | 2.158 |
| SCRIPPS-FORMATION ($T_{sc}$) | 300 | 31 | 23 | 2.743 |
| SCRIPPS-FORMATION ($T_{sc}$) | 300 | 31 | 41 | 1.824 |
| SCRIPPS-FORMATION ($T_{sc}$) | 300 | 31 | 27 | 2.399 |

|  |  |
|---|---|
| **α** | The slope angle; (inclination angle) with respect to the horizontal plane |
| **ϕ'** | The effective friction angle of the soil |
| **c'** | The effective cohesion of the soil |
| **$\Upsilon_t$** | The total unit weight (Soil with moisture) |
| **$\Upsilon_w$** | The unit weight of the water |
| **$\Upsilon'$** | Submerged unit weight of the soil (Saturated unit weight - unit weight of water) |
| **$z_w$** | Vertical depth of the saturated soil |
| **F.O.S.** | Factor of Safety |

1 meter = 3.28 feet

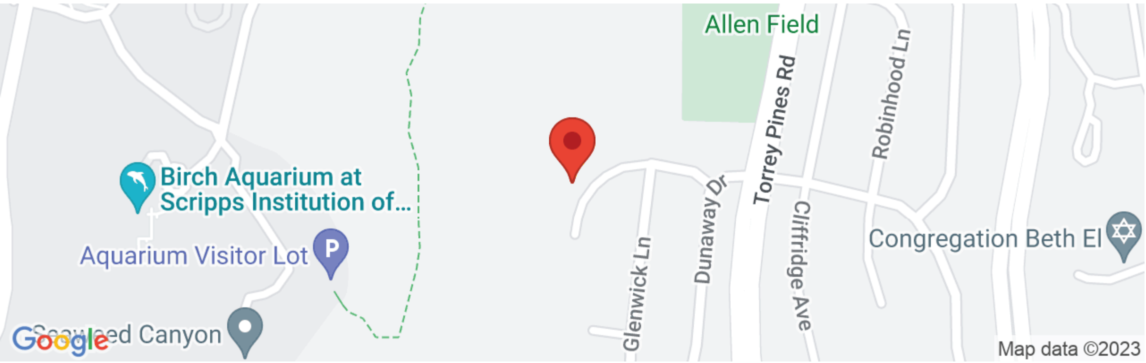
Special Publication 117A (2008, page 27): for infinite slope analysis, the minimum assumed depth of soil saturation is the smaller of either a depth of one meter or depth to firm bedrock.

The Factor of Safety values are **ABOVE** 1.50 and are adequate.

# APPENDIX D

## 2734 Bordeaux Ave, La Jolla, CA 92037, USA

**Latitude, Longitude: 32.8661023, -117.2454305**



| | |
|---|---|
| **Date** | 2/17/2023, 7:58:19 PM |
| **Design Code Reference Document** | ASCE7-16 |
| **Risk Category** | II |
| **Site Class** | D - Stiff Soil |

| **Type** | **Value** | **Description** |
|---|---|---|
| $S_S$ | 1.371 | $MCE_R$ ground motion. (for 0.2 second period) |
| $S_1$ | 0.48 | $MCE_R$ ground motion. (for 1.0s period) |
| $S_{MS}$ | 1.371 | Site-modified spectral acceleration value |
| $S_{M1}$ | null -See Section 11.4.8 = **0.873** | Site-modified spectral acceleration value |
| $S_{DS}$ | 0.914 | Numeric seismic design value at 0.2 second SA |
| $S_{D1}$ | null -See Section 11.4.8 = **0.582** | Numeric seismic design value at 1.0 second SA |

| **Type** | **Value** | **Description** |
|---|---|---|
| SDC | null -See Section 11.4.8 = **D** | Seismic design category |
| $F_a$ | 1 | Site amplification factor at 0.2 second |
| $F_v$ | null -See Section 11.4.8 = **1.819** | Site amplification factor at 1.0 second |
| PGA | 0.626 | $MCE_G$ peak ground acceleration |
| $F_{PGA}$ | 1.1 | Site amplification factor at PGA |
| $PGA_M$ | 0.688 | Site modified peak ground acceleration |
| $T_L$ | 8 | Long-period transition period in seconds |
| SsRT | 1.371 | Probabilistic risk-targeted ground motion. (0.2 second) |
| SsUH | 1.579 | Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration |
| SsD | 2.099 | Factored deterministic acceleration value. (0.2 second) |
| S1RT | 0.48 | Probabilistic risk-targeted ground motion. (1.0 second) |
| S1UH | 0.54 | Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration. |
| S1D | 0.736 | Factored deterministic acceleration value. (1.0 second) |
| PGAd | 0.869 | Factored deterministic acceleration value. (Peak Ground Acceleration) |
| $PGA_{UH}$ | 0.626 | Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration |
| $C_{RS}$ | 0.868 | Mapped value of the risk coefficient at short periods |
| $C_{R1}$ | 0.888 | Mapped value of the risk coefficient at a period of 1 s |
| $C_V$ | 1.374 | Vertical coefficient |

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