REPORT OF PRELIMINARY GEOTECHNICAL AND GEOLOGIC FAULT INVESTIGATION

Proposed Javaheri Residence
2072 Via Casa Alta
La Jolla, California

JOB NO. 21-13556
28 July 2022

Prepared for:

Mr. Kevin Javaheri

Geotechnical Exploration, Inc.

SOIL AND FOUNDATION ENGINEERING • GROUNDWATER • ENGINEERING GEOLOGY

28 July 2022

Mr. Kevin Javaheri
c/o *MARENGO MORTON ARCHITECTS*
Attn: Mr. Claude-Anthony Marengo
Via email: CAMarengo@m2a.io

Job No. 21-13556

Subject: **Report of Preliminary Geotechnical Investigation and Geologic Fault Investigation**
Proposed Javaheri Residence
2072 Via Casa Alta
La Jolla, California

Dear Mr. Javaheri:

In accordance with your request, and our work agreement dated September 14, 2021, ***Geotechnical Exploration, Inc.*** has performed a preliminary geotechnical investigation and geologic fault investigation for the subject project in La Jolla, California. The field work was performed on December 20, 2021, and March 2-3, 2022.

If the conclusions and recommendations presented in this report are incorporated into the design and construction of the proposed site development, it is our opinion that the site is suitable for the proposed project from a geotechnical perspective.

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. 21-13556** will expedite a response to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

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TABLE OF CONTENTS

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The following report presents the findings and recommendations of ***Geotechnical Exploration, Inc.*** for the subject project.

I. PROJECT SUMMARY

It is our understanding, based on communications with your project architect, Mr. Claude-Anthony Marengo of Marengo Morton Architects, and review of the preliminary architectural plans, that the vacant subject site is proposed to receive a new 16,251-square-foot, two-story over basement single-family residential structure, an accessory dwelling unit (ADU), a swimming pool, driveway, landscaping and associated improvements. The proposed new structures and improvements are to be constructed of standard-type building materials utilizing conventional foundation systems with either concrete slab on-grade or raised wood floors. Foundation loads are expected to be typical for this type of relatively light construction.

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 in this report under *Conclusions and Recommendations No. 8.*

Based on our current understanding of the proposed construction, it is our explicit opinion that the proposed site development 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 explicit opinion, based on our field investigation, review of pertinent geologic literature and analysis of geological maps and aerial photographs, that neither an active nor a potentially active fault or landslide underlies the subject site.

II. SCOPE OF WORK

The scope of work performed for this investigation consisted of a field investigation with a site reconnaissance and geotechnical subsurface exploration program under the direction of our geologist, review of available published literature pertaining to the site geology, laboratory testing, geotechnical engineering analysis of the field and laboratory data, and the preparation of this report.

The field investigation consisted of an exploratory large-diameter boring and exploratory trench to gather subsurface data and evaluate geologic hazards at the site. Advancement, logging and sampling of the large diameter boring on December 20, 2021, gathered subsurface data and enabled us to assess potential landslide hazards across the project area. Excavation, logging and sampling of an exploratory trench on March 2 and 3, 2022, extending across the building pad area of the site, gathered subsurface data and enabled us to assess potential faulting hazards across the project area. The data obtained and the analyses performed were for the purpose of evaluating geologic hazards and providing appropriate mitigation, as well as providing geotechnical design and construction criteria for the project earthwork, building foundations, slab on-grade floors, swimming pool, driveway, retaining walls and associated improvements.

At the request of Mr. Marengo, ***Geotechnical Exploration Inc.*** also provided a *"Report of Geologic Reconnaissance"* dated 08 March 2022. The purpose of that report was to provide a research study of potential geologic hazards that should be evaluated during the investigation, provide preliminary opinions based on our research, and provide guidance and scope to investigate potential hazards.

III. SITE DESCRIPTION

The subject property is known as Assessor's Parcel No. 352-750-15-00, Lot 15, per Recorded Map No. 8482, in the Mount Soledad area of the City and County of San Diego, State of California. Refer to Figure No. I, the Vicinity Map, for the site location.

The roughly rectangular-shaped site is 0.770-acre in size. The site consists of a relatively level to gently sloping, undeveloped southern portion where the new development is proposed. The northern portion of the site is a densely vegetated, relatively steep, northerly descending slope. Vegetation consists of weeds, grasses, native shrubs and mature trees.

The site is currently unoccupied with no structures or associated improvements. The site is bordered on the east by a single-family residence at a slightly lower elevation; on the west by a single-family residence at a slightly higher elevation; on the north by an unpaved portion of Hillside Drive approximately two-thirds down the slope; and on the south by Via Casa Alta, from where the site is also accessed.

The elevation across the site ranges from approximately 695 feet above Mean Sea Level (MSL) along the northern property line, to 794 feet above MSL in the southwestern corner. Information concerning elevations across the site was obtained from the Topographic Survey, undated, by Ciremele Surveying Inc.

IV. FIELD INVESTIGATION, OBSERVATIONS & SAMPLING

The field investigation was performed in two phases. The first phase consisted of a surface reconnaissance and advancement of a 30-inch large diameter boring (LDB-1; see Figure No. IIIa) in the building pad area utilizing a truck-mounted drill rig with bucket auger. The large diameter boring was advanced to a depth of 80 feet below existing grade and our geologist was lowered into the boring to log in situ three-dimensional structural components, and gather data on subsurface conditions. In particular, the potential presence of shear zones was investigated to evaluate if the southern portion of the site is underlain by a landslide.

The second phase consisted of excavation of a trench (T-1; see Figure No. IIIb) across the building pad area utilizing a track-mounted hoe for the purpose of investigating if active faulting crosses the building pad area of the proposed development. The trench was excavated to a depth of up to 9 feet and a minimum of 3½ feet into formational soils across the entire length of the trench. The placement and total length of the trench was strategically located to intersect mapped faults of the area (Kennedy, 1975) and any potential strands of mapped faults within a 30-degree orientation within the building pad area of the proposed structure.

The soils encountered in the large diameter boring and trench were continuously logged in the field by our geologist and described in accordance with the Unified Soil Classification System (refer to Appendix A). The approximate locations of the large diameter boring, trench and site-specific geology are shown on the Plot Plan and Site-Specific Geologic Map, Figure No. II.

Representative soil samples for laboratory geotechnical testing were obtained from the large diameter boring and trench at selected depths appropriate to the investigation. Sampling consisted of the collection of disturbed bulk samples and relatively undisturbed chunk samples to aid in classification and for appropriate laboratory testing. A 3-inch outer diameter hand driven sampler was also used to obtain undisturbed ring samples. All samples were returned to our laboratory for evaluation and testing. Exploratory boring and trench logs were prepared on the basis of our observations and laboratory test results and are attached as Figure Nos. IIIa-d.

The exploratory boring and trench logs and related information depict 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 locations. Also, the passage of time may result in changes in 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 residential development. Test results are presented on the exploratory boring and trench logs, Figure Nos. IIIa-d and the Laboratory Test Results, Figure Nos. IVa-b. The following tests were conducted on the sampled soils:

1. *Laboratory Compaction Characteristics (ASTM D1557-12[2021])*
2. *Determination of Percentage of Particles Smaller than #200 Sieve (ASTM D1140-17)*
3. *Expansion Index (ASTM D4829-19)*
4. *Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions (ASTM D3080-11)*
5. *Radiocarbon Age Dating by High Probability Density Range Method (HPD): INTCAL20*

Laboratory compaction values (ASTM D1557-12[2021]) 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 soils and soil compaction conditions to be anticipated during any future grading operation. The test results are presented on the exploratory boring and trench logs at the appropriate sample depths and Figure Nos. IVa-b.

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 the exploratory boring and trench logs at the appropriate sample depths and on Figure Nos. IVa-b.

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	***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 visual classification and our laboratory test results of 90 and 92, the sandy fat clay slopewash, and lean clay/clayey sands argillic terrace materials overlying the upper 1 to 3 feet of the site possess a high potential for expansion. Based on our visual classification and experience with similar Cabrillo Formation sandstone materials, it is our opinion that the formational materials underlying the site and encountered in our exploratory boring and trench possess a very low to low potential for expansion.

Radiocarbon age dating was performed on three samples by a third-party testing laboratory, Beta Analytic, Inc., using the High Probability Density Range Method (HPD): IntCal20. The naturally occurring unstable carbon-14 isotope undergoes beta decay into the stable nitrogen-14 isotope, with a half-life of 5,370 (±40) years. By comparing the ratio of residual carbon-14 to stable carbon-12 and carbon-13 isotopes, the age of the sample can be determined. The three samples were obtained in the slopewash materials and were dated at 3,530 (±30), 3,060 (±30) and 2,090 (±30) years before present. The test results are presented in Appendix C.

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 associated improvements.

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).

In the Coastal Plain region, where the subject property is located, 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 Nacion 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).

As indicated previously, 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 Geological Survey (CGS), 2018, a "Holocene-active fault" is one that has had surface displacement within Holocene time, the last 11,700 years. In addition, "pre-Holocene fault" is a fault whose recency of past movement is older than 11,700 years.

A three-tier fault classification is used as follows:

- *Active Faults* had demonstrable surface displacement during the Holocene time, where Holocene time is the geological epoch that began 11,700 years before present.
- *Potentially Active Faults* had demonstrable surface displacement during Quaternary time, but Holocene surface displacement is indeterminate.
- *Inactive Faults* are pre-Quaternary faults where the Quaternary period timeline is approximately 1.6 million years ago.

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). Although this earthquake was widely felt, no significant damage was reported. 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. These were felt in San Diego but no injuries or damage was reported. 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. These were also felt in San Diego, swaying high-rise buildings, but again no significant damage or injuries were reported.

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. The event caused widespread damage to structures, closure of businesses, government offices and schools, power outages, displacement of people from their homes and injuries in the nearby major metropolitan areas of Mexicali in Mexico and Calexico in Southern 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. Some structures in the San Diego area experienced minor damage and there were some injuries. 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. The earthquake was felt throughout Southern California, with strong shaking near the epicenter. 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.

VII. SITE-SPECIFIC SOIL & GEOLOGIC DESCRIPTION

Our field work, reconnaissance and review of the *"Geologic Map of San Diego, 30'x60' Quadrangle, CA,"* by Kennedy and Tan, 2008, indicate that the site is underlain at relatively shallow depth by upper Cretaceous-aged Cabrillo sandstone (Kcs) formational materials. In the southern portion of the site, weathered argillic marine terrace materials of the middle to early Pleistocene-aged Very Old Paralic Deposits, ($Qvop_{10}$) were encountered overlying Cabrillo formational materials. Terrace materials and materials characteristic of middle Eocene-aged Ardath Shale (Ta) lithology were encountered in transverse cracks crossing our test trench and the site (see Structure below). Of particular significance, unbroken or offset $Qvop_{10}$ (±850 ka) deposits were observed to overlie the infilled transverse cracks. The remainder of the investigated central portion of the site is overlain by approximately 1 to 3 feet of surficial slopewash materials (Qsw). As shown on the cross section (Figure No. IIId), the slopewash materials dated between 2,090 (±30) and 3,530 (±30) years before present were also not offset by any of the transverse crack features. Figure

No. V presents a plan view geologic map (Kennedy and Tan, 2008) of the general area of the site.

A. Stratigraphy

Slopewash (Qsw): Slopewash materials were encountered overlying the central portion of the site and were observed in both the large diameter boring LDB-1 and trench T-1, ranging from 1 to 3 feet in thickness. The encountered slopewash consists of moist, dark brown, sandy fat clay (CH). In the large diameter boring, approximately 10% of the slopewash materials were observed to be angular to rounded gravels and cobbles, and some tree roots were also observed. As mentioned above, slopewash materials were carbon 14-dated between 2090 (±30) and 3530 (±30) years before present.

The slopewash is very stiff and, in our opinion, has a high expansion potential. In our opinion, due to the high expansion potential, the slopewash is not suitable to support loads from new foundations or additional fill. Review and evaluation of the final grading plan will be required to determine if all of the high expansion potential soils can be utilized on site. Refer to Figure Nos. IIIa-d and IVa-b for details.

Marine Terrace Deposits/Very Old Paralic Deposits ($Qvop_{10}$): Very Old Paralic Deposit materials, also known as Marine Terrace Deposits, were encountered in the southern portion of the site and observed in trench T-1. The encountered terrace materials consist of slightly moist, dark reddish brown argillic lean clay/clayey sand (CL/SC) and a near vertical lens of dry to slightly moist, reddish brown to orangish brown silty gravel with sand (GM) infilling a transverse crack in the Cabrillo Formation. The basal contact of the 850± ka Very Old Paralic Deposits were not offset across the transverse cracking in the Cabrillo Formation.

The material in the transverse crack was observed to contain up to approximately 40% rounded gravels and cobbles. The terrace materials are dense/very stiff. In our opinion the argillic lean clay/clayey sand has a high expansion potential and the silty gravel with sand has a low expansion potential. Furthermore, the argillic terrace materials are not suitable in their current condition for support of loads from new foundations or additional fill due to their high expansion potential. Review and evaluation of the final grading plan will be required to determine if all of the high expansion potential soils can be utilized on site. Refer to Figure Nos. IIIa-d and IVa-b for details. Kennedy and Tan (2008), describe the Very Old Paralic Deposits, Unit 10, as *"Poorly sorted, moderately permeable, reddish-brown, interfingered strandline, beach, estuarine and colluvial deposits composed of siltstone, sandstone and conglomerate."*

Cabrillo Formation Sandstone (Kcs): The Cabrillo formation sandstone underlies the entire project area at a relatively shallow depth. The encountered Cabrillo sandstone formational materials consist of fine- to medium-grained, slightly moist, yellowish brown, silty sand (SM). The formational materials encountered are dense to very dense, and in our opinion, have a very low to low expansion potential. Minor amounts of sandy silt (ML) and lean clay (CL) materials, possibly originating from the Ardath Shale or a Cabrillo formation lithologic unit similar to the Ardath Shale, were observed to be infilling traverse cracks. Refer to Figure Nos. IIIa-d and Figure Nos. IVa-b for details. In our opinion, the Cabrillo formational materials are suitable in their current condition to support additional fill or loads from the proposed additions or improvements. Kennedy and Tan (2008), describe the Cabrillo Formation as *"Mostly massive medium-grained sandstone."*

B. Structure

Geologic structure was observed in the Cabrillo Formation during the large-diameter boring and trenching phases of the field investigation. Generally, the Cabrillo Formation was observed to have massive structure. However, at depths of 52 feet and 76 feet in the large diameter boring (Figure Nos. IIIa-c), bedding attitudes of N80°E, 20°N and N80°E, 26°N, respectively, were observed. These are in close agreement with the N70°E, 25°N attitude recorded by Kennedy approximately 300 feet northwest and on the same northerly sloping hillside as the subject property.

General observations of geologic structure in the exploratory large diameter boring:

1. Clay and calcium carbonate filled fractures were observed, with the fracture planes generally being near vertical and random in bearing.
2. Minor caliche and conglomerate veins were observed in random orientation.
3. Minor fractures with no offset were observed, often with iron oxide staining or discolored sandstone.
4. Concretions in the sandstone were common.
5. Fractures were generally observed to be healed or infilled with clay or sandy materials. Open fractures were not observed. Indications of continuous shearing or brecciation were not observed.

General observations of geologic structure in the exploratory trench:

1. Minor fractures were observed in the Cabrillo Formation, generally near vertical and healed with calcium carbonate materials.

2. High-angle separations in the Cabrillo Formation from approximately 1 to 3 feet wide were generally observed to be infilled with Ardath Shale type material and Very Old Paralic Deposit, Unit 10 material. The orientation of these separations was generally close to east-west to northeast-southwest and the dip direction was generally approximately 45 to 80 degrees downslope. No indications of recent movement were observed in these fractures, and generally they appear to be healed.

3. The two largest separations in the Cabrillo Formation (up to approximately 10 feet wide) were observed to be infilled with materials characteristic of the Ardath Shale Formation. The larger separation zones are oriented N70^{O}E, 64^{O}NW and N90^{O}E, 54^{O}N. Due to the 54° and 64° dipping surfaces, and the inherent strength characteristics of the predominant Cabrillo Formation, it is our opinion that the geologic structure is neutral with respect to global stability of the site. Slope stability calculations have been performed along geologic cross section A-A' and are presented in Appendix D.

4. Two linear structural features are shown on the Geologic Hazards Map as Zone 12 (i.e., potentially active, inactive, presumed inactive or activity unknown) crossing Via Casa Alta. It is important to note that the two features were placed on the Geologic Hazards Map based on mapping by Kennedy, 1975. These two short linear features were eliminated as faults by Kennedy and Tan, 2008, in their update map of the San Diego Quadrangle (see Figure No. V for an excerpt of this map) and, in our opinion, are not faults but are most likely due to tectonic uplift breakage.

As shown on the geologic map (Figure No. V) and geologic hazards map (Figure No. VI), two linear features cross Via Casa Alta. The westernmost feature crosses Via Casa Alta approximately 8 parcels west of the subject property and the eastern feature crosses Via Casa Alta 4 lots to the west and across the subject property in an easterly direction. The eastern feature was encountered by our firm crossing the lower northwest corner of the lot, adjacent to the fire station, during the 1997 development of the property. The short eastern and western linear features are oriented at 50- to 80-degree angles, respectively, to the Mount Soledad branch of the Rose Canyon Fault zone, which crosses the lower northern flank of Mount Soledad. The Via Casa Alta features are not, therefore, aligned with the Rose Canyon Fault zone primary stress relief system. In our opinion, they are not faults with the potential for offset in response to accumulating strain relief but are more likely a result of intraformational breakage of tectonic origin due to structural deformation resulting in the 350 feet of Mount Soledad uplift.

We note that the western feature as mapped by Kennedy in 1975 passes under the Lindavista Formation (Qln)/Very Old Paralic Deposits ($Qvop_{10}$) without offsetting them. This indicates the breakage feature predates deposition of the 850± ka paralic deposits and the most recent uplift of Mount Soledad. The eastern feature was not mapped crossing the paralic deposit on the Via Casa Alta ridgeline but we consider it to also predate deposition of the 850± ka Very Old Paralic Deposits and the Mount Soledad 350 feet of uplift.

In summary, it is our opinion that, although the Via Casa Alta formational breakage features are faulting by definition, they are not part of the Rose Canyon Fault zone primary stress relief system that would warrant considering them active or potentially active faults. We consider it much more likely that both of the Via Casa Alta linear features, which were eliminated from the 2008 Kennedy and Tan geologic map, are tectonic in origin and predate deposition of the ridgeline capping Very Old Paralic Deposits and the most recent 350-foot Mount Soledad uplift event.

Furthermore, it is our opinion that the risk of structurally significant damage to these features as a result of sympathetic movement in response to a Rose Canyon Fault zone event is nominal. Given all of the above, it is our opinion that an active or potentially active fault does not underlie the subject property.

VIII. GEOLOGIC HAZARDS

Our review of the City of San Diego Seismic Safety Study, Geologic Hazards Map, Sheet 29 (2008) indicates that the site is located in a geologic hazard area designated as Geologic Hazard Categories (GHC) 12 and 27. An excerpt of the map is presented in Figure No. VI, Seismic Hazard Map Excerpt and Legend. GHC 12 is a fault zone category described as *"Potentially Active, Inactive, Presumed Inactive, or Activity Unknown."* GHC 27 is a slide prone formation category described as *"Otay, Sweetwater, and others."*

As previously described, based on our reconnaissance, the data obtained from our field investigation, and the Kennedy 1975 and Kennedy and Tan 2008 geologic map, it is our opinion that an active or potentially active fault does not underlie the site. Despite the evidence of ancient landsliding in the Cabrillo Formation on the northern flank of Mount Soledad, in our opinion, the site is not underlain by an active landslide or a high-risk, slide prone formation. Furthermore, our review of the *"Earthquake Zones of Required Investigation, La Jolla Quadrangle"* by the California Geological Survey (CGS), dated September 23, 2021, indicates the site is not within the

"Earthquake Fault Zones." No significant geologic hazards are known to exist on the subject site that would prohibit the proposed construction.

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

A. Local and Regional Faults

Reference to the geologic map of the area (Kennedy and Tan, 2008), Figure No. V, indicates that no faults are shown to cross the site. In our explicit professional opinion, neither an active fault nor a potentially active fault underlies the site.

Newport-Inglewood-Rose Canyon Fault Zone System: The Rose Canyon portion of the Newport-Inglewood-Rose Canyon Fault Zone is mapped approximately 0.25-mile northeast of the site and the offshore portion of the Newport-Inglewood portion is mapped approximately 24 miles northwest of the site. The offshore portion of the Newport-Inglewood Fault Zone is described as a right-lateral, local reverse slip associated with fault steps (SCEDC, 2022). The reported length is 46.2 miles extending in a northwest-southeast direction. Surface trace is discontinuous in the Los Angeles Basin, but the fault zone can easily be noted there by the existence of a chain of low hills extending from Culver City to Signal Hill. South of Signal Hill, it roughly parallels the coastline until just south of Newport Bay, where it heads offshore, and becomes the *Newport-Inglewood-Rose Canyon Fault Zone*. A significant earthquake (M6.4) occurred along this fault on March 10, 1933. Since then, no additional significant events have occurred. The fault is believed to have a slip rate of approximately 0.6-mm/yr with an unknown recurrence interval. This fault is believed capable of producing an earthquake of M6.0 to M7.4 (Grant Ludwig and Shearer, 2004).

Rose Canyon Fault Zone: The Rose Canyon Fault Zone is the southern section of the Newport-Inglewood-Rose Canyon Fault Zone system mapped in the San Diego County area as trending north-northwest to south-southeast from Oceanside to San Diego and generally north-south into San Diego Bay, through Coronado and offshore downtown San Diego, from where it appears to head southward. The Rose Canyon Fault Zone system is considered to be a complex zone of onshore and offshore, en echelon right lateral, strike slip, oblique reverse, and oblique normal faults. This fault is considered to be capable of generating an M6.9 earthquake (EERI, 2021) and is considered microseismically active, although no significant recent earthquakes since 1862 (Legg and Agnew, 1979) are known to have occurred on the fault.

Investigative work on faults that are part of the Rose Canyon Fault Zone at the Police Administration and Technical Center in downtown San Diego, at the SDG&E facility in Rose Canyon, and within San Diego Bay and elsewhere within downtown San Diego, has encountered offsets in Holocene (geologically recent) sediments (Singleton et al., 2019). These findings confirm Holocene displacement on the Rose Canyon Fault, which was designated an "*active*" fault in November 1991 (Hart and Bryant, 1997).

Rockwell (2010) has suggested that the Rose CFZ underwent a cluster of activity including 5 major earthquakes in the early Holocene, with a long period of inactivity following, suggesting major earthquakes on the RCFZ behaves in a cluster-mode, where earthquake recurrence is clustered in time rather than in a consistent recurrence interval. With the most recent earthquake (MRE) nearly 160 years ago, it is suggested that a period of earthquake activity on the RCFZ may have begun.

Rockwell (2010) and a compilation of the latest research implies a long-term slip rate of approximately 1 to 2 mm/year.

Coronado Bank Fault: The Coronado Bank Fault is located approximately 12.6 miles southwest of the site. Evidence for this fault is based upon geophysical data (acoustic profiles) and the general alignment of epicenters of recorded seismic activity (Greene, 1979). The Oceanside earthquake of M5.3, recorded July 13, 1986, is known to have been centered on the fault or within the Coronado Bank Fault Zone. Although this fault is considered active, due to the seismicity within the fault zone, it is significantly less active seismically than the Elsinore Fault (Hileman, 1973). It is postulated that the Coronado Bank Fault is capable of generating a M7.6 earthquake and is of great interest due to its close proximity to the greater San Diego metropolitan area.

San Diego Trough Fault Zone: The San Diego Trough Fault Zone is mapped approximately 23 miles west-southwest of the site at its closest point. This fault is described as a right-lateral type fault with a length of at least 93.2 miles and a slip rate of roughly 1.5 mm/yr. The most recent surface rupture is of Holocene age (SCEDC, 2022).

San Clemente Fault Zone: The San Clemente Fault Zone is mapped approximately 45 miles southwest of the site at its closest point. This fault is described as a right-lateral and vertical offsets type fault with a length of at least 130.5 miles described as essentially continuous with the San Isidro fault zone, off the coast of Mexico and a slip rate of roughly 1.5 mm/yr. The most recent surface rupture is of Holocene age (SCEDC, 2022).

Elsinore Fault: The Temecula and Julian sections of the Elsinore Fault Zone are located approximately 38 to 56 miles northeast and east of the site. The Elsinore Fault Zone extends approximately 200 kilometers (125 miles) from the Mexican border to the northern end of the Santa Ana Mountains. The Elsinore Fault zone is a 1- to 4-mile-wide, northwest-southeast-trending zone of discontinuous and en echelon faults extending through portions of Orange, Riverside, San Diego, and Imperial Counties. Individual faults within the Elsinore Fault Zone range from less than 1 mile to 16 miles in length. The trend, length and geomorphic expression of the Elsinore Fault Zone identify it as being a part of the highly active San Andreas Fault system.

Like the other faults in the San Andreas system, the Elsinore Fault is a transverse fault showing predominantly right-lateral movement. According to Hart, et al. (1979), this movement averages less than 1 centimeter per year. Along most of its length, the Elsinore Fault Zone is marked by a bold topographic expression consisting of linearly aligned ridges, swales and hallows. Faulted Holocene alluvial deposits (believed to be less than 11,700 years old) found along several segments of the fault zone suggest that at least part of the zone is currently active.

Although the Elsinore Fault Zone belongs to the San Andreas set of active, northwest-trending, right-slip faults in the southern California area (Crowell, 1962), it has not been the site of a major earthquake in historic time, other than a M6.0 earthquake near the town of Elsinore in 1910 (Richter, 1958; Toppozada and Parke, 1982). However, based on length and evidence of late-Pleistocene or Holocene displacement, Greensfelder (1974) has estimated that the Elsinore Fault Zone is reasonably capable of generating an earthquake with a magnitude as large as M7.5. Study and logging of exposures in trenches placed in Glen Ivy Marsh across the Glen Ivy North Fault (a strand of the Elsinore Fault Zone between Corona and Lake Elsinore), suggest a maximum earthquake recurrence interval of 300 years, and when combined with previous estimates of the long-term horizontal slip rate of 0.8 to 7.0 mm/year, suggest typical earthquake magnitudes of M6.0 to M7.0 (Rockwell et al., 1985). The Working Group on California Earthquake Probabilities (2008) has estimated that there is a 11 percent probability that an earthquake of M6.7 or greater will occur within 30 years on this fault.

San Jacinto Fault: The San Jacinto Fault is located 60 to 82 miles northeast of the site. The San Jacinto Fault Zone consists of a series of closely spaced faults, including the Coyote Creek Fault, that form the western margin of the San Jacinto Mountains. The fault zone extends from its junction with the San Andreas Fault in San Bernardino, southeasterly toward the Brawley area, where it continues south of the international border as the Imperial Transform Fault (Rockwell et al., 2014).

The San Jacinto Fault zone has a high level of historical seismic activity, with at least 10 damaging earthquakes (M6.0 to M7.0) having occurred on this fault zone between 1890 and 1986. Earthquakes on the San Jacinto Fault in 1899 and 1918 caused fatalities in the Riverside County area. Offset across this fault is predominantly right-lateral, similar to the San Andreas Fault, although some investigators have suggested that dip-slip motion contributes up to 10% of the net slip (Ross et al., 2017).

The segments of the San Jacinto Fault that are of most concern to major metropolitan areas are the San Bernardino, San Jacinto Valley and Anza segments. Fault slip rates on the various segments of the San Jacinto are less well constrained than for the San Andreas Fault, but the available data suggest slip rates of 12 ±6 mm/yr for the northern segments of the fault, and slip rates of 4 ±2 mm/yr for the southern segments. For large ground-rupturing earthquakes on the San Jacinto fault, various investigators have suggested a recurrence interval of 150 to 300 years. The Working

Group on California Earthquake Probabilities (2008) has estimated that there is a 31 percent probability that an earthquake of M6.7 or greater will occur within 30 years on this fault. Maximum credible earthquakes of M6.7, M6.9, and M7.2 are expected on the San Bernardino, San Jacinto Valley and Anza segments, respectively, capable of generating peak horizontal ground accelerations of 0.48g to 0.53g in the County of Riverside. A M5.4 earthquake occurred on the San Jacinto Fault on July 7, 2010.

The United States Geological Survey has issued the following statements with respect to the recent seismic activity on southern California faults:

> The San Jacinto fault, along with the Elsinore, San Andreas, and other faults, is part of the plate boundary that accommodates about 2 inches/year of motion as the Pacific plate moves northwest relative to the North American plate. The largest recent earthquake on the San Jacinto fault, near this location, the M6.5 1968 Borrego Mountain earthquake April 8, 1968, occurred about 25 miles southeast of the July 7, 2010, M5.4 earthquake. This M5.4 earthquake follows the 4th of April 2010, Easter Sunday, M7.2 earthquake, located about 125 miles to the south, well south of the US Mexico international border. A M4.9 earthquake occurred in the same area on June 12th at 8:08 pm (Pacific Time). Thus, this section of the San Jacinto fault remains active.
>
> Seismologists are watching two major earthquake faults in southern California. The San Jacinto fault, the most active earthquake fault in southern California, extends for more than 100 miles from the international border into San Bernardino and Riverside, a major metropolitan area often called the Inland Empire. The Elsinore fault is more than 110 miles long, and extends into the Orange County and Los Angeles area as the Whittier fault. The Elsinore fault is capable of a major earthquake that would significantly affect the large metropolitan areas of southern California. The Elsinore fault has not hosted a major earthquake in more than 100 years. The occurrence of these earthquakes along the San Jacinto fault and continued aftershocks demonstrates that the earthquake activity in the region remains at an elevated level. The San Jacinto fault is known as the most active earthquake fault in southern California. Caltech and USGS seismologist continue to monitor the ongoing earthquake activity using the

Caltech/USGS Southern California Seismic Network and a GPS network of more than 100 stations.

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: Based upon our geotechnical investigation as well as information provided on the Geologic Maps by Kennedy (1975) and Kennedy and Tan (2008), it is our opinion that the site is not underlain by the ancient landslide complex that exists lower on the northern flank of Mount Soledad. Refer to Section VII of this report, Site-Specific Soil and Geologic Description, subsection B, Structure, under

"*General Observations of Geologic Structure in the Exploratory Trench,"* (Numbers 1-4 beginning on page 15), for our description and analysis regarding the encountered Cabrillo Formation structural features.

Further review of the geologic map (Kennedy and Tan, 2008) and review of the aerial photographs (4-11-53, AXN-8M-1 and 2) show no conclusive geomorphic evidence that the site is underlain by a recent or active landslide.

Slope Stability: Slope stability analysis has been performed along geologic cross section A-A'. Refer to Appendix D for slope stability calculations. We performed a static and pseudo-static analysis with a seismic coefficient of 0.15g. We also performed a saturated surficial stability analysis for an assumed soil saturation up to 3.28 feet (1 meter). Our analysis indicates the site is stable with a global and surficial factor of safety of 1.5 for static conditions, and 1.15 for seismic loading. Upon review of the final grading plan, slope stability analysis will be performed and additional recommendations provided, if warranted.

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. In the areas of the proposed habitable structures, the risk of liquefaction of formational materials 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.

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.

The site is located approximately 1 mile from the exposed coastline and at an elevation of approximately 695 to 794 feet above MSL. There is 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.

C. Geologic Hazards Summary

No significant geologic hazards are known to exist on the site that would prohibit the construction of the proposed residence 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 new additions 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 the site is underlain at a depth of approximately 2 to 3 feet below existing ground surface by relatively stable formational materials and is suited for the proposed residence and associated improvements provided the recommendations herein are implemented. Furthermore, based on our current understanding of the proposed construction, it is our explicit opinion that the proposed site development 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.

In our explicit professional opinion, no active or potentially active faults or landslides underlie the site in the area of the proposed construction.

IX. GROUNDWATER

Groundwater was not was encountered during our field investigation. 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 grading operations can 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 appearance of 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.

It must be understood that unless discovered during initial site exploration or encountered during site grading operations, it is extremely difficult to predict if or where perched or true groundwater conditions may appear in the future. When site 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 grading operations, 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.

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.

> *The geotechnical consultant that has prepared documents in support of an approved permit is considered the geotechnical consultant of record. A change of geotechnical consultant of record must be processed if the*

project's geotechnical consultant is changed after a permit has been issued and before the project is as-built and closed. The new geotechnical consultant must prepare a Transfer of Geotechnical Responsibility letter. If the new geotechnical consultant utilized the geotechnical investigation and test data prepared by the previous geotechnical consultants of record, the new geotechnical consultant must reference the geotechnical reports approved for the project and must state that they agree with the data, recommendations and conclusions contained in those reports. The new consultant must also state that the data, recommendations and conclusions are valid for the proposed construction. For grading permits, the specific drawing number must be included in the statement. Alternatively, the new geotechnical consultant has the option of conducting an independent geotechnical investigation. A change of geotechnical consultant of record after a grading permit has been issued will require a formal construction change to the grading plans.

We recommend that the planned residential development and external improvements, including flatwork, be founded on properly compacted structural fill soils or suitably dense formational soils, supported by conventional, individual-spread and/or continuous footings. Existing slopewash soils and trench backfill soils across the project area are not suitable in their current condition to support the loads from structures or additional fill soils. Furthermore, slopewash soils should not be used as structural fill material. Existing formational materials are suitable for use as recompacted fill soils are selectively removed during grading. Fill soils across the site will be required to be compacted to at least 90 percent relative compaction.

A. Site Preparation and Earthwork

1. *Stripping:* The areas of proposed development should be stripped of existing vegetation within the areas of proposed new construction. This includes any roots from existing trees and shrubbery. 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.

2. *Excavation:* For the new development, any slopewash below the grade of the bearing surfaces of footings and slabs should be removed and selectively stockpiled or removed from the site. Existing fill soils used to backfill the exploratory trenches are also to be removed and recompacted. It should be anticipated that the depth of removal will be up to 8 feet in the areas of the exploratory trench, and approximately 2 to 3 feet in all other areas. Recompaction of these existing fill materials should be done in accordance with Recommendation Nos. 3, 4 and 5 below. Based on the observations of our exploratory trench, as well as our experience with similar materials in the project area, it is our opinion that the existing fill soils and slopewash should 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. Variability in excavating the subsurface materials should be expected across the project area.

   The areal extent and final depth required to remove the existing fill and slopewash soils should be confirmed by our representatives during the excavation work based on their examination of the soils being exposed. Dense formational soils shall be exposed at the bottom of excavation before any fill soils are placed. The lateral extent of the excavation and recompaction should be at least 5 feet beyond the edge of any areas to receive exterior improvements, where feasible, or to the depth of excavation or fill at that location, whichever is greater.

3. *Subgrade Preparation:* After the required excavations have been made in the areas of new improvements, the exposed subgrade soils in areas to receive new fill and/or slab-on-grade improvements should be scarified to 6 inches in depth, moisture conditioned, and compacted to the requirements for structural fill. Where planned cuts expose any highly expansive materials in the building areas, these soils should be scarified and moisture conditioned to at least 5 percent over optimum moisture and placed in landscape areas where the effects of soil expansion are inconsequential.

4. *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 in general areas. 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. Oversize material and organics should be selectively removed from the fill material prior to compaction operations.

5. *Structural Fill Compaction:* All structural fill, and areas to receive any associated improvements, should be compacted to a minimum degree of compaction of 90 percent based upon ASTM D1557-12[2021]. 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. For low expansive soils, the moisture content should be within 2 percent of optimum. High expansive soils to be exposed during general grading operations should be moisture conditioned to at least 5 percent over optimum moisture content for highly expansive soils, and placed in landscape areas where the effects of soil expansion are inconsequential.

Any rigid improvements founded on the existing undocumented fill 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.

6. *Chloride and Soluble Sulfate Testing*: Large concentrations of chlorides will adversely affect any ferrous metals such as iron and steel. Soil with a chloride concentration greater than or equal to 500 ppm (0.05 percent) or more is considered corrosive to ferrous metals. The chloride content of the near surface soils should be tested at the completion of grading and before foundation excavations. Test results should be evaluated by an engineer specializing in soil corrosivity. The primary cause of deterioration of concrete in foundations and other below ground structures is the corrosive attack by soluble sulfates present in the soil and groundwater. The soluble sulfate content of the near surface soils should be tested at the completion of grading and before

foundation excavations. Test results should be evaluated by an engineer specializing in soil corrosivity. Cement type recommendations for concrete specifications should be provided by the structural engineer based on the soluble sulfate test results.

It is noted that ***Geotechnical Exploration Inc.***, does not practice corrosion engineering and our assessment here should be construed as an aid to the owner or owner's representative. A corrosion specialist should be consulted for any specific design requirement.

7. *Trench and Retaining Wall Backfill:* All utility trenches and retaining walls should be backfilled with properly compacted fill. 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. 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. Soil compaction testing by nuclear method ASTM D6938-17a or sand cone method ASTM D1556-15e1 should be performed for every 2 feet of fill placement by a representative of ***Geotechnical Exploration, Inc.*** in conventional retaining wall and trench backfill areas as well in general fill or backfill areas.

   Backfill soils placed behind retaining walls should be installed as early as the retaining walls are capable of supporting lateral loads. Backfill soils behind retaining walls should be low expansive (Expansion Index less than 50 per ASTM D4829-19).

8. _Observations and Testing_: As stated in CBC 2019, _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 fill 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 indicates that it is mandatory that a representative of this firm (responsible engineering firm) perform observations and fill compaction testing during grading and backfilling operations to verify that the 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 that 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 (unless supported by deep foundations or caissons) but all site improvements such as stairways, patios, pools and pool decking, retaining walls, 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.

Often after primary building pad grading and swimming pool excavation, it is not uncommon for the geotechnical engineer of record to not be notified of grading performed outside the footprint of the project primary structures. As a result, settlement damage of site improvements such as patios, pool and pool decks, exterior landscape walls and walks, and structure access stairways can occur. It is therefore strongly recommended that the project general contractor, grading contractor, and others tasked with completing the project, be advised and acknowledge the importance of adequate and comprehensive observation and testing of soils intended to support the project they are working on. The project geotechnical engineer of record must be contacted and requested to provide these services.

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. Requests for services are to be made through our office telephone number (858) 549-7222 and the telephone number of the GEI personnel assigned to the project.

B. Seismic Design Criteria

9. *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.

10. *Seismic Design Criteria:* The proposed structure should be designed in accordance with the 2019 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 32.8397 degrees and a longitude of -117.2511 degrees, utilizing a program titled "S*eismic Design Map Tool" and* provided by the USGS through SEAOC, which provides a solution for ASCE 7-16 utilizing digitized files for the Spectral Acceleration maps.

11. *Structure and Foundation Design*: The design of the new structures and foundations should be based on Seismic Design Category D, Risk Category II.

12. *Spectral Acceleration and Design Values*: The structural seismic design, when applicable, should be based on the following values, which are based on the site location, soil characteristics, and seismic maps by USGS, as required by the 2019 CBC. A response Spectrum Acceleration (SA) vs. Period (T) for the site is also included in Appendix B. The Site Class C (Very Dense Soil and Soft Rock) values for this property are:

TABLE I
Mapped Spectral Acceleration Values and Design Parameters

S_s	S_1	F_a	F_v	S_{ms}	S_{m1}	S_{ds}	S_{d1}
1.416g	0.494g	1.2	1.5	1.699g	0.742g	1.133g	0.494g

C. Foundation Recommendations

13. *Footings*: We recommend that all buildings and pertinent associated improvements be supported on adequately bearing formational materials or properly recompacted structural fill soils prepared in accordance with Recommendation Nos. 2, 3, 4 and 5. No footings should be underlain by undocumented fill or loose soils. All footings for two- to three-story structures should be founded at least 24 inches below lowest adjacent soil finished grade. All footings should be reinforced with at least four No. 5 bars or more as specified by the structural designer. A minimum clearance of 3 inches should be maintained between steel reinforcement and the bottom or sides of the footing.

    The bearing surfaces of footings located adjacent to utility trenches should be situated below an imaginary 1.0:1.0 plane projected upward from the bottom edge of the adjacent utility trench. Otherwise, the utility trenches should be excavated farther from the footing locations. Footings located adjacent to the tops of slopes should be extended sufficiently deep in order to provide at least 7 feet of horizontal cover between the slope face and outside edge of the footing at the footing bearing level.

    In order for us to offer an opinion as to whether the footings are founded on soils of sufficient load bearing capacity and with the necessary 7 feet of horizontal cover to the slope face, it is essential that our representative inspect the footing excavations prior to the placement of reinforcing steel or forms.

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.

14. *Bearing Values*: At the recommended depths, footings on formational or properly recompacted fill soils may be designed for allowable bearing pressures of 2,500 pounds per square foot (psf) for combined dead and live loads and 3,325 psf for all loads, including wind or seismic. The footings should, however, have a minimum depth of 18 inches and 12 inches wide. An increase in soil allowable static bearing can be used as follows: 900 psf for each additional foot over 1½ feet in depth, and 500 psf for each additional foot in width over 1 foot, to a total allowable static bearing pressure not exceeding 4,500 psf. The static soil bearing value may be increased one-third for seismic and wind load analysis

15. *Lateral Loads:* Lateral load resistance for the structure supported on footing foundations may be developed in friction between the foundation bottoms and the supporting subgrade. An allowable friction coefficient of 0.35 is considered applicable. An additional allowable passive resistance equal to an equivalent fluid weight of 270 pounds per cubic foot (pcf) acting against the foundations may be used in design provided the footings are poured neat against the dense formational or properly compacted fill materials. These lateral resistance values assume a level surface in front of the footing for a minimum distance of three times the embedment depth of the footing and any shear keys, but not less than 7 feet from a descending slope face, measured from effective top of foundation. New retaining walls supporting surcharge loads or affected by upper foundations should consider the effect of those upper loads.

16. *Settlement:* Settlement under structural design loads is expected to be within tolerable limits for the proposed structures. For footings designed in accordance with the recommendations presented in the preceding paragraphs, we anticipate that total settlements should not exceed 1 inch and that post-construction differential settlement angular rotation should be less than 1/240.

D. Concrete Slab On-Grade Criteria

Slabs on-grade may only be used on new, properly compacted fill or when founded on adequately bearing formational soils.

17. *Minimum Floor Slab Thickness and Reinforcement:* Based on our experience, we have found that, for various reasons, floor slabs occasionally crack. Therefore, we recommend that all slabs on-grade contain at least a minimum amount of reinforcing steel to reduce the separation of cracks, should they occur. Slab subgrade soil should be verified by a ***Geotechnical Exploration, Inc***. representative to have the proper moisture content within 48 hours prior to placement of the vapor barrier and pouring of concrete.

    Soil moisture content should be kept above the optimum prior to waterproofing or vapor barrier placement under the new concrete slab. For interior areas in the new building, we recommend a 5-inch-thick slab reinforced with No. 4 steel bars spaced 18 inches apart. Interior slabs on grade shall be provided with control joints specified by the structural engineer.

    We note that shrinkage cracking can result in reflective cracking in brittle flooring surfaces such as stone and tiles. It is imperative that if movement intolerant flooring materials are to be utilized, the flooring contractor and/or architect should provide specifications for the use of high-quality isolation membrane products installed between slab and floor materials.

18. *Slab Moisture Emission:* Although it is not the responsibility of geotechnical engineering firms to provide moisture protection recommendations, as a service to our clients we provide the following discussion and suggested minimum protection criteria. Actual recommendations should be provided by the project architect and waterproofing consultants or product manufacturer. It is recommended to contact the vapor barrier manufacturer to schedule a pre-construction meeting and to coordinate a review, in-person or digital, of the vapor barrier installation.

Soil moisture vapor can result in damage to moisture-sensitive floors, some floor sealers, or sensitive equipment in direct contact with the floor, in addition to mold and staining on slabs, walls and carpets. The common practice in Southern California is to place vapor retarders made of PVC, or of polyethylene. PVC retarders are made in thickness ranging from 10- to 60-mil. Polyethylene retarders, called visqueen, range from 5- to 10-mil in thickness. These products are no longer considered adequate for moisture protection and can actually deteriorate over time.

Specialty vapor retarding and barrier products possess higher tensile strength and are more specifically designed for and intended to retard moisture transmission into and through concrete slabs. The use of such products is highly recommended for reduction of floor slab moisture emission.

The following American Society for Testing and Materials (ASTM) and American Concrete Institute (ACI) sections address the issue of moisture transmission into and through concrete slabs: ASTM E1745-17 Standard Specification for

Plastic Water Vapor Retarders Used in Contact Concrete Slabs; ASTM E1643-18a Standard Practice for Selection, Design, Installation, and Inspection of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs; ACI 302.2R-06 Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials; and ACI 302.1R-15 Guide to Concrete Floor and Slab Construction.

18.1 Based on the above, we recommend that the vapor barrier consist of a minimum 15-mil extruded polyolefin plastic (no recycled content or woven materials permitted). Permeance as tested before and after mandatory conditioning (ASTM E1745 Section 7.1 and subparagraphs 7.1.1-7.1.5) should be less than 0.01 perms (grains/square foot/hour/per inch of Mercury) and comply with the ASTM E1745-17 Class A requirements. Installation of vapor barriers should be in accordance with ASTM E1643-18a. The basis of design is 15-mil Stego Wrap vapor barrier placed per the manufacturer's guidelines. Reef Industries Vapor Guard membrane has also been shown to achieve a permeance of less than 0.01 perms. We recommend that the slab be poured directly on the vapor barrier, which is to be placed directly on 4 inches of Class II base layer or 3/8-inch maximum diameter crushed rock gravel on the prepared properly compacted smooth subgrade soil surface.

18.2 Common to all acceptable products, vapor retarder/barrier joints must be lapped at least 6 inches. Seam joints and permanent utility penetrations should be sealed with the manufacturer's recommended tape or mastic. Edges of the vapor retarder should be extended to terminate at a location in accordance with ASTM E1643-18a or to an alternate location that is acceptable to the project's structural engineer.

All terminated edges of the vapor retarder should be sealed to the building foundation (grade beam, wall, or slab) using the manufacturer's recommended accessory for sealing the vapor retarder to pre-existing or freshly placed concrete. Additionally, in actual practice, stakes are often driven through the retarder material, equipment is dragged or rolled across the retarder, overlapping or jointing is not properly implemented, etc. All these construction deficiencies reduce the retarder's effectiveness. In no case should retarder/barrier products be punctured or gaps be allowed to form prior to or during concrete placement. Vapor barrier-safe screeding and forming systems should be used that will not leave puncture holes in the vapor barrier, such as Beast Foot (by Stego Industries) or equivalent.

18.3 Vapor retarders/barriers do not provide full waterproofing for structures constructed below free water surfaces. They are intended to help reduce or prevent vapor transmission and/or capillary migration through the soil and through the concrete slabs. Waterproofing systems must be designed and properly constructed if full waterproofing is desired. The owner and project designers should be consulted to determine the specific level of protection required.

18.4 Following placement of any concrete floor slabs, sufficient drying time must be allowed prior to placement of floor coverings. Premature placement of floor coverings may result in degradation of adhesive materials and loosening of the finish floor materials.

19. *Exterior Slab Thickness and Reinforcement:* Exterior slab reinforcement and control joints should be designed by the project Structural Engineer. As a minimum for protection of on-site improvements, we recommend that all exterior pedestrian concrete slabs be at least 4 inches thick, reinforced with No. 3 bars at 15-inch centers, both ways at the center of the slab, and contain adequate isolation and control joints and be sealed with elastomeric joint sealant.

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. Slabs on-grade may only be used on dense formational soils or properly compacted fill soils.

E. Retaining Wall Design Criteria

20. *Design Parameters – Unrestrained:* The active earth pressure to be utilized in the design of any cantilever site retaining walls, utilizing low-expansion potential [EI less than 50] imported soils as backfill should be based on an Equivalent Fluid Weight of 38 pcf (for level backfill only). For 2.0:1.0 sloping backfill, the cantilever retaining walls should be designed with an equivalent fluid pressure of 52 pcf. Unrestrained retaining walls should be backfilled with imported or on-site very low to low expansion potential soils. Restrained building retaining walls should be designed for 56 pcf for level imported low expansion potential soil backfill, and use a conversion load factor of 0.47 for vertical surcharge loads to be converted to uniform lateral surcharge loads. Temporary cantilever shoring walls supporting on-site low expansive

formational soils can use an active pressure of 40 pcf and a conversion factor of 0.35 to convert vertical uniform surcharge to horizontal uniform pressure. For passive resistance, use the value of 685 pcf times the diameter of the soldier pile, times the depth of embedment below the grade excavation in front of the piles. To reduce the expansion potential of on-site soils, the low expansive backfill soils should extend behind the walls at least a distance equal to the height of the wall. The upper 1 foot of backfill may consist of properly compacted on-site soils, and should be provided with proper surface drainage.

21. *Design Parameters – Restrained:* Permanent site restrained building retaining walls supporting low expansion potential 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). Restrained shoring walls supporting on site high expansion potential soils, should be designed for 71 pcf soil pressure and a vertical to lateral load conversion factor of 0.60. 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 included in the wall design pressure. A conversion factor of 0.56 pcf may be used to convert vertical uniform surcharge loads to lateral uniform pressure behind a restrained retaining wall with imported low expansion potential level backfill and 0.76 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 expansion potential soils with EI less than 50, and should be placed from the heel of the foundation to the ground surface within a distance equal to the equal height, and passing by the heel of the foundation and the back face of the retaining wall. When imported low expansion potential backfill soils cannot be placed due to property line proximity, the retaining walls should be designed for the expansive soil pressures recommended above.

22. *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 20 pcf. This seismic increment is waived for restrained walls. If the walls are designed as unrestrained walls, then the seismic load should be added to the static soil pressure.

23. *Retaining Wall Drainage:* The preceding design pressures assume that the walls are backfilled with 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 as a minimum 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 board 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. Actual waterproofing recommendations to prevent water access to below grade spaces must be provided by the retaining wall/waterproofing contractor or project architect.

Geotechnical Exploration, Inc. will assume no liability for damage to structures or improvements that is attributable to poor drainage. In order to improve the potential of maintaining below grade spaces in a dry condition, we recommend that consideration be given to placing lower-level wall subdrains at least 1 foot ***below*** the bottom of the lower-level slabs (refer to Figure No. VI).

24. *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 proper wall sealing, geofabric installation, protection board (if needed), drain depth below interior floor or yard surface, pipe percent slope to the outlet, etc.

25. *Retaining Wall Backfill:* 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. Sand slurry backfill may be used behind the lagging of the shoring walls. Medium to high expansion potential on-site soils should not be used as retaining wall backfill material.

F. Swimming Pool Recommendations

26. *Excavation and Foundations:* We recommend that the proposed pool bear on the underlying undisturbed formational materials. Plans should be furnished to us indicating the location and dimensions of the pool for our review prior to construction.

27. *Pool Shell Loads:* The pool should be designed using a static earth pressure of 75 pcf for the highly expansive soil condition and a seismic pressure increment of 20 pcf if the pool depth is 6 feet or deeper. If the pool is to be raised above the adjacent grade or located within 10 feet of the existing descending slope face, free standing walls must be incorporated into the design. Free standing walls should be designed to resist a water pressure of 62.4 pcf. No cut/fill transition line should underlie the pool. A maximum 5 feet of fill differential thickness should be used beneath the pool shell to help reduce potential differential soil settlement. Additional recommendations may be provided as warranted after pool plans are reviewed by our firm before construction starts.

28. *Deck Subgrade, Slab and Drainage:* Any existing loose fill soils supporting a planned pool deck should be properly moisture-conditioned and compacted prior to steel and concrete placement per the requirements of Recommendations 2, 3, 4 and 5.

    Proper drainage with area drains should be provided in the pool deck area. The pool deck slab should be at least 5 inches thick and be reinforced with at least No. 3 bars every 18 inches apart unless designed as a structural slab (which might require still more reinforcing) with supports properly spaced to span the design distances. Adequately spaced control joints should be placed by the contractor and sealed with elastomeric joint sealant. Joint spacing should not exceed 12 feet apart. Joints should also be placed at re-entrant corners. The control joints should be placed within 12 hours after concrete placement and penetrate at least one-quarter the thickness of the slab. All joints should be sealed with elastomeric joint sealant. Drainage around the pool deck should be positive to drain away from the deck's perimeter and the slope face into area drains. Proper soil moisture content should be confirmed within 48 hours prior to concrete placement.

G. ***Driveway Recommendations***

29. *Pervious Pavers*: We recommend that if pervious pavers are desired for the driveway, subject to automobile and fire truck traffic, the driveway should be underlain by 1 inch of bedding sand No. 8 on 12 inches of crushed rock miscellaneous base. The upper 12 inches of the subgrade soils should be compacted to a minimum degree of compaction of 90 percent and the base layer to at least 95 percent relative compaction. A collector perforated pipe should be placed in the lower areas of subgrade and outlet in an appropriate low point. For driveways with sloping surface concrete, curbs may be needed to help reduce potential for lateral movement of the paver blocks.

30. *PCC Pavement:* We recommend that if Portland Cement Concrete is desired for the driveway specially if the pavement slope exceeds 5 percent, the driveway should have a thickness of 6 inches and the concrete should be underlain by 12 inches of crushed miscellaneous base. The upper 12 inches of the subgrade soils as well as the base layer should be compacted to a minimum degree of compaction of 90 percent. The concrete should conform to Section 201 of The Standard Specifications for Public Works Construction, 2019 Edition, for Class 560-C-3250 and be reinforced with No. 4 bars on 18-inch centers, both ways, placed at midheight in the slab. Control and isolation joints should be provided with elastomeric joint sealant.

H. ***Slopes***

31. *Permanent Slopes*: Any new cut or fill slopes should be constructed at an inclination no steeper than 2.0:1.0 (horizontal to vertical). Proper vegetation placed on slope surfaces will help prevent or reduce soil erosion. Irrigation of vegetation should be kept to the minimum for plant survival.

32. *Temporary Slopes*: Based on our subsurface investigation work, laboratory test results, and engineering analysis, temporary slopes should be stable for a maximum slope height of up to 10 feet at a slope ratio of 1.5:1.0 in the existing fill soils, and at 0.75:1.0 in dense formational soils. Our representative, however, should observe temporary slope excavations, and if variability in the subsurface materials is observed, additional temporary slope recommendations will be presented at that time. Localized sloughing or raveling of soils exposed on the slopes should be anticipated.

    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 method of operation. No soil stockpiles or surcharge may be placed or exist within a horizontal distance of 10 feet from the top of the excavation.

33. *Slope Top/Face Performance:* The soils that occur in close proximity to the top or face of even properly compacted fill or dense natural ground cut slopes often possess poor lateral stability. The degree of lateral and vertical deformation depends on the inherent expansion and strength characteristics of the soil

types comprising the slope, slope steepness and height, loosening of slope face soils by burrowing rodents, and irrigation and vegetation maintenance practices, as well as the quality of compaction of fill soils. Structures and other improvements could suffer damage due to these soil movement factors if not properly designed to accommodate or withstand such movement. New fill or cut slopes should be constructed at a 2.0:1.0 slope gradient. Any existing steeper slopes that will remain should be analyzed for stability.

34. *Slope Top Structure Performance:* Rigid improvements such as top-of-slope walls, columns, decorative planters, concrete flatwork and other similar types of improvements can be expected to display varying degrees of separation typical of improvements constructed at the top of a slope. The separations result primarily from slope top lateral and vertical soil deformation processes. These separations often occur regardless of being underlain by cut or fill slope material. Proximity to a slope top is often the primary factor affecting the degree of separations occurring.

I. Site Drainage Considerations

35. *Surface Drainage:* Adequate measures should be taken to properly finish-grade the site after the new improvements are in place. Drainage waters from this site and adjacent properties should be directed away from the footings, slabs, and slopes, onto the natural drainage direction for this area or into properly designed and approved drainage facilities provided by the project civil engineer. Proper subsurface and surface drainage will help reduce the potential for waters to seek the level of the bearing soils under the wall footings or other extensive improvements.

Failure to observe this recommendation could result in undermining, soil expansion, and possible differential settlement of the retaining wall or other improvements or cause other moisture-related problems. Currently, the 2019 CBC requires a minimum of 1 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. The surface gradient adjacent to structures must drain away as indicated in the 2019 CBC at least 5 percent within 5 feet from the perimeter.

Due to the possible build-up of groundwater (derived primarily from rainfall and irrigation), excess moisture is a common problem behind retaining walls that may be planned. These problems are generally in the form of water seepage through walls and mineral staining. In order to minimize the potential for moisture-related problems to develop, the backfill side of all retaining walls must be adequately waterproofed and drained.

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.

37. *Planter Drainage:* Planter areas and planter boxes should be sloped to drain away from the foundations. Planter boxes should be constructed with a closed bottom and a subsurface drain, installed in gravel, with the direction of subsurface and surface flow away from the footings to an adequate drainage facility.

38. *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 proper wall sealing, geofabric installation, protection board (if needed), drain depth below interior floor or yard surface, pipe percent slope to the outlet, etc.

J. General Recommendations

39. *Cal-OSHA*: Where not superseded by specific recommendations presented in this report, trenches, excavations, and temporary slopes at the subject site should be constructed in accordance with Title 8, Construction Safety Orders, issued by Cal-OSHA.

40. *Project Start Up Notification:* In order to reduce any work delays during site excavation and development, our firm should be contacted at least 48 hours before any required 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 our observations of the excavations. If 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 before the correction of the observed problem (i.e., deepening the footing excavation, compacting or removal of loose soil in the bottom of the excavation, etc.).

41. *Construction Best Management Practices (BMPs):* Sufficient BMPs must be installed to prevent silt, mud, or other construction debris from being tracked into the adjacent street(s) or stormwater 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 a breach in the installed construction BMPs.

    All stockpiles of uncompacted soil and/or building materials that are 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 higher. 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 and 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 Preliminary Geotechnical and Geologic Fault Investigation"* for the project. In addition, the compaction of any fill soils placed during site grading work must be observed and tested by the soil engineer. It is the responsibility of the general contractor to comply with the requirements on the approved plans and the local building ordinances. All/any retaining wall and trench backfill should be properly compacted. ***Geotechnical Exploration, Inc.*** will assume no liability for damage occurring due to improperly compacted 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, background review and laboratory analysis, as well as our experience with similar soils and natural ground materials located in the County of San Diego. 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 excavation begins. 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.

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 wall plans, especially with respect to the height and location of the proposed wall structure, this report must be presented to us for immediate review and possible revision.

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.

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 final project plans once they are available, to verify that our recommendations are adequately incorporated in the plans. Additional or revised recommendations may be necessary after our review.

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. 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. 21-13556** will expedite a reply to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

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JOB NO. 21-13556
July 2022

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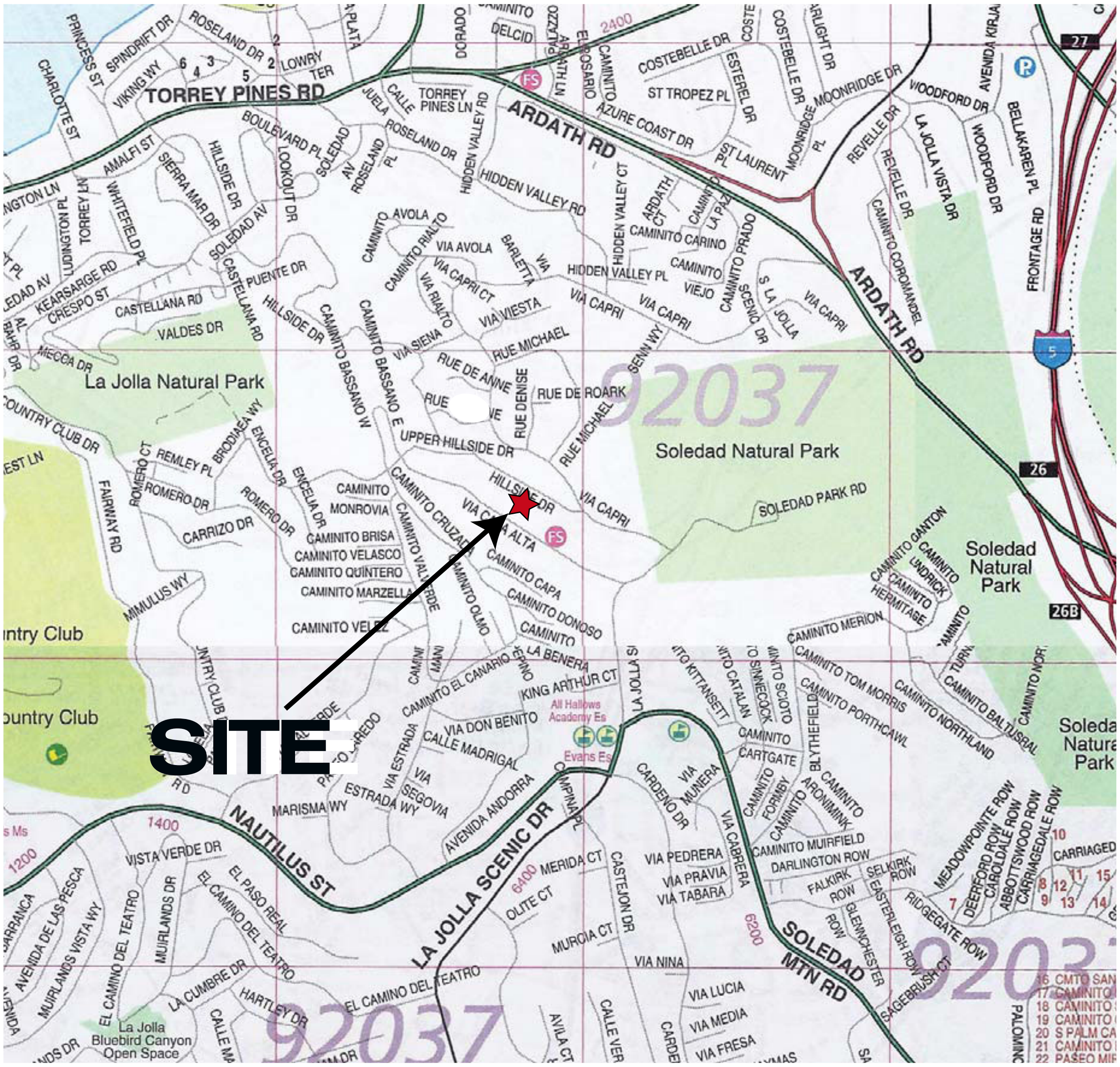
VICINITY MAP

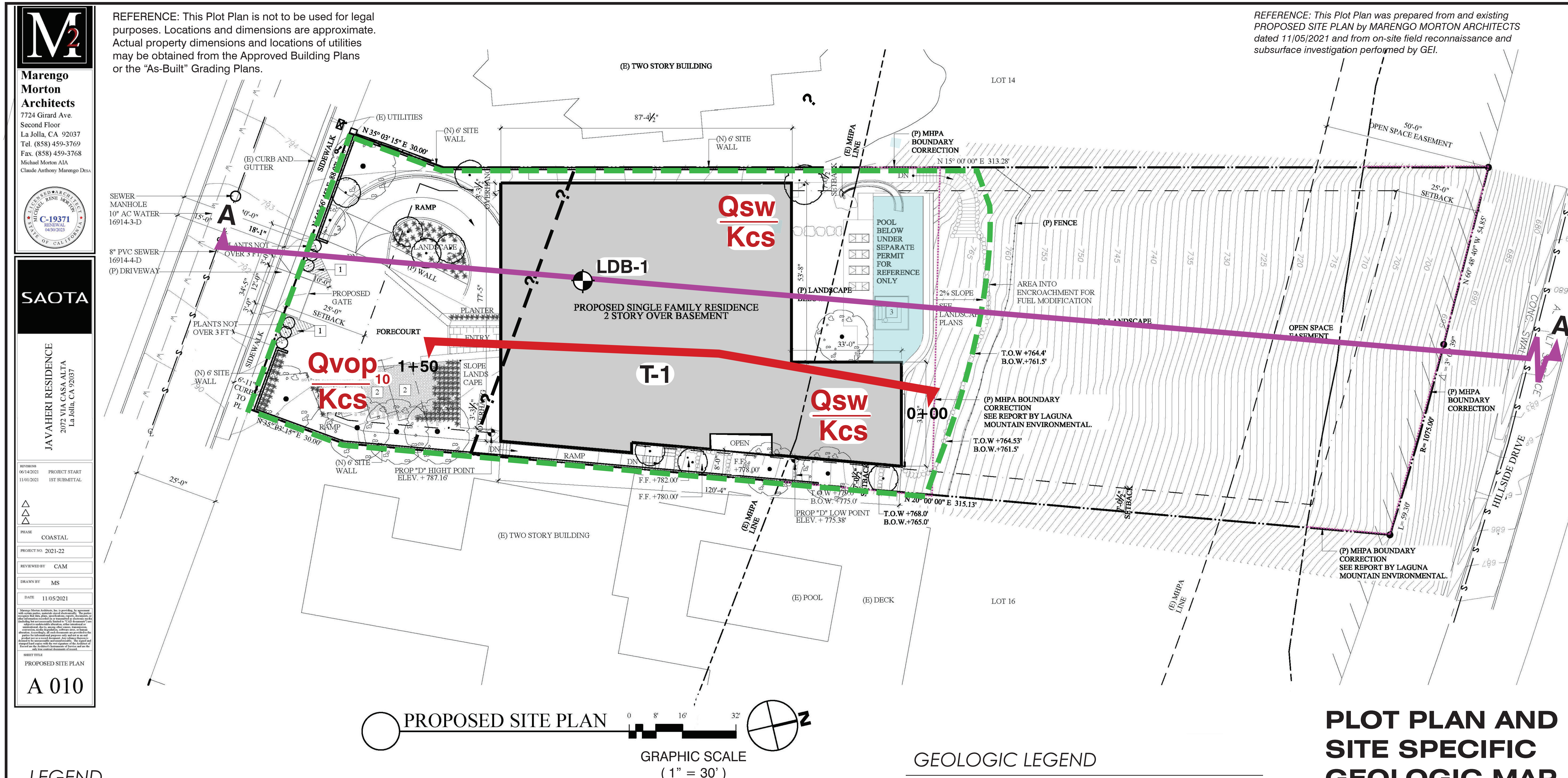


Thomas Bros Guide San Diego County pg 1227-H7

Javaheri Residence
2072 Via Casa Alta
La Jolla, CA.

Figure No. I
Job No. 21-13556

Geotechnical Exploration, Inc.



LEGEND

T-1 (1+50 — 0+00)	Approximate Location of Exploratory Trench with Station Numbers
LDB-1	Approximate Location of Large Diameter Boring
	Approximate Limits of Geotechnical Investigation
A — A'	Geologic Cross Section
	Location of Proposed Structures
	Location of Proposed Swimming Pool

GEOLOGIC LEGEND

Qsw	Slopewash
Qvop 10	Very Old Paralic Deposits, Unit 10
Kcs	Cabrillo Formation - sandstone
? ?	Approximate Geologic Contact

PLOT PLAN AND SITE SPECIFIC GEOLOGIC MAP

Javaheri Residence
2072 Via Casa Alta
La Jolla, CA.
Figure No. IIa
Job No. 21-13556

Geotechnical Exploration, Inc.

July 2022

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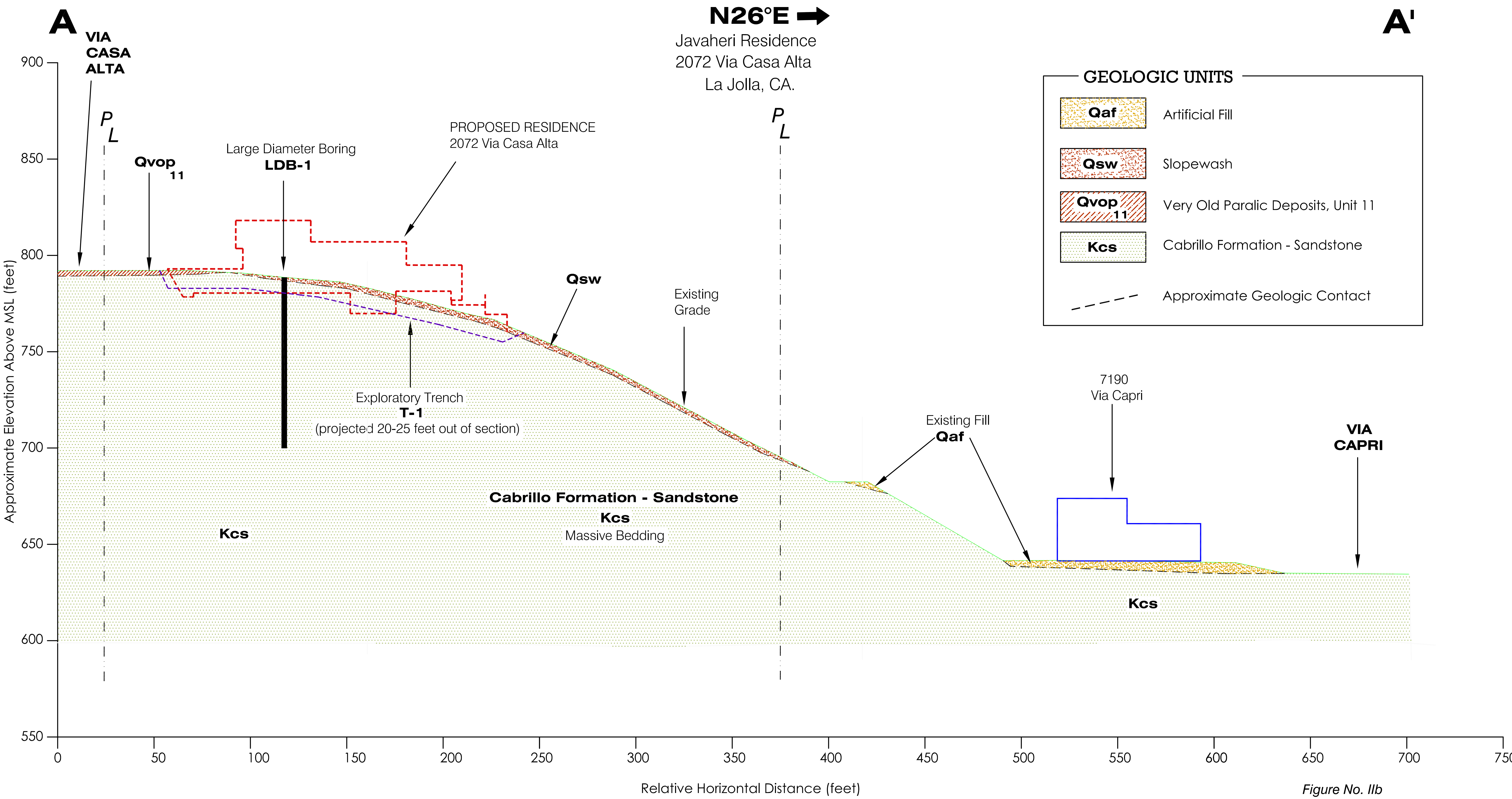
GEOLOGIC CROSS-SECTION A-A'

N26°E

Javaheri Residence
2072 Via Casa Alta
La Jolla, CA.



Scale: 1" = 50'
(Horizontal and Vertical)

NOTE: This Geologic 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.

Figure No. IIb
Job No. 21-13556

Geotechnical Exploration, Inc.

July 2022

19-12176-AA.dwg

Geotechnical Exploration, Inc.	**EQUIPMENT:** Bucket auger drill rig
	DIMENSION & TYPE OF EXCAVATION: 30-inch diameter boring
DATE LOGGED: December 20, 2021	
LOGGED BY: JKH/MM	**SURFACE ELEVATION:** ± 790' Above Mean Sea Level
REVIEWED BY: LDR	**GROUNDWATER/SEEPAGE DEPTH:** Not Encountered

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain Size, Density, Moisture, Color)	U.S.C.S	PASSING NO. 200 SIEVE (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of MDD)	EXPAN (+%) CONSOL (-%)	EXPANSION INDEX	BLOW COUNTS / FT	SAMPLE O.D. (in)
2			**SLOPEWASH (Qsw)** **SANDY FAT CLAY.** Very stiff. Moist. Dark brown. Approx. 10% angular to rounded gravels and cobbles. Some roots.	CH										
4 6			**CABRILLO FORMATION SANDSTONE (Kcs)** **SILTY SAND.** Fine- to medium-grained. Dense. Slightly moist. Yellowish brown. Micaceous. Disturbed by adjacent Rose Canyon Fault Zone and resultant tectonic uplift of Mount Soledad.	SM										
8			@8': Clay filled fractures, near vertical.											
10			@10': Calcium carbonate filled fracture, near vertical, dip ±70°SE.											
12			@12': 1/4" caliche veins with 1/2-2" wide pebble conglomerate, N55°E, dip 72°SE. @13.5': Pebble conglomerate terminates SE side of boring.											
14			@14': Calcium carbonate filled fracture, near vertical, ±78°SE, 1/2" wide.											
16			@15': Discontinuous offsets in gray and tan sandstone with random calcium carbonate veining.											
18			@18' Infilled fractures, 1/4-1/2" wide. @19' Some concretionary masses.											
20														
22														
24			@24': 3-4" layer of remolded clay, discontinuous, dips ±15°											
26			@25': drag folded on east side with N70°E trend, near vertical thin discontinuous clay veins with some calcium carbonate filling on east side.											
28														

GROUNDWATER	**JOB NUMBER:** 21-13556	LOG NO. **LDB-1**
BULK BAG SAMPLE	**JOB NAME:** Javaheri Residence	
1 IN-PLACE SAMPLE		
MODIFIED CALIFORNIA SAMPLE	**SITE LOCATION:** 2072 Via Casa Alta, La Jolla, CA	FIGURE NO. **IIIa**
H IN-PLACE HAND-DRIVE SAMPLE		
STANDARD PENETRATION TEST		

Geotechnical Exploration, Inc.

DATE LOGGED: December 20, 2021
LOGGED BY: JKH/MM
REVIEWED BY: LDR

EQUIPMENT: Bucket auger drill rig
DIMENSION & TYPE OF EXCAVATION: 30-inch diameter boring
SURFACE ELEVATION: ± 790' Above Mean Sea Level
GROUNDWATER/SEEPAGE DEPTH: Not Encountered

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain Size, Density, Moisture, Color)	U.S.C.S	PASSING NO. 200 SIEVE (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of MDD)	EXPAN (+%) CONSOL (-%)	EXPANSION INDEX	BLOW COUNTS / FT	SAMPLE O.D. (in)
			Numerous high angle fractures in sandstone.											
		1	@29-30': Concretion. 1/4-1/2" olive gray clay N-S, 68°E. Discontinuous, truncated by high angle olive gray clay veins.			11.6	119.5			99				
30		Bulk bag			34			12.3	120.3					
			@30': 5 blows with kelly bar.											
32			@32': Low to high angle fractures filled with olive brown sand fractures in light brown sandstone.											
			@33-40': Massive fine- to medium-grained sandstone, micaceous, fracturing continued, as noted above.											
34														
36														
38														
40			@40': Concretion. Slight fracturing, no open fractures, generally massive, some reddish brown iron oxide staining.											
42														
44														
			@45-46': Concretion, west wall.											
46														
48														
			@49': Large concretion, used rock auger.											
50														
52			@52': Massive fine- to medium-grained sandstone continued below contact N80°E, 20°N. Thin gray clayey sand veins, truncated, light brown sandstone with iron staining along veins.											
54														
56			@56': Concretion on west wall.											

GROUNDWATER
BULK BAG SAMPLE
1 IN-PLACE SAMPLE
MODIFIED CALIFORNIA SAMPLE
H IN-PLACE HAND-DRIVE SAMPLE
STANDARD PENETRATION TEST

JOB NUMBER: 21-13556
JOB NAME: Javaheri Residence
SITE LOCATION: 2072 Via Casa Alta, La Jolla, CA

LOG NO. **LDB-1**

FIGURE NO. **IIIb**

Geotechnical Exploration, Inc.

DATE LOGGED: December 20, 2021	**EQUIPMENT:** Bucket auger drill rig
LOGGED BY: JKH/MM	**DIMENSION & TYPE OF EXCAVATION:** 30-inch diameter boring
REVIEWED BY: LDR	**SURFACE ELEVATION:** ± 790' Above Mean Sea Level
	GROUNDWATER/SEEPAGE DEPTH: Not Encountered

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION — DESCRIPTION AND REMARKS (Grain Size, Density, Moisture, Color)	U.S.C.S	PASSING NO. 200 SIEVE (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of MDD)	EXPAN (+%) CONSOL (-%)	EXPANSION INDEX	BLOW COUNTS / FT	SAMPLE O.D. (in)
58														
60			@59': Some fractures, none open above concretion on west wall with some gravel, generally massive. N40°E, 5°SE discontinuous 1/8-1/4" remolded olive brown clay, truncated 3" on north wall, clay replaced by iron staining and calcium carbonate, 1' long zone on NW wall.											
62														
64			@64-66': Dark gray clay block on west wall.											
66			@66': 6" wide high angle dark gray clay infilling with light brown sandstone clasts.											
68			@69': Tapers out, depositional, parrallel iron staining.											
70			@70': Dark clay infilled fracture 2-3" wide, near vertical and discontinuous.											
72			@71': Truncated zone 12" wide deposition with parallel iron staining.											
			@73': Sandstone concretions continue.											
74														
76			@76': Becomes fractured claystone-sandstone fragments, wavy surface N80°E, ±26°NW. Massive fine- to medium-grained sandstone with occassional gravels: light brown, iron staining continued with vertical zones of dark brown to olive gray clay filled fractures, not brecciated, no sign of shearing, occassional sandstone clasts, blocky fractured claystone.											
78		2												
80			Bottom of boring at 80 feet.											
82														
84														

GROUNDWATER	**JOB NUMBER:** 21-13556		
BULK BAG SAMPLE	**JOB NAME:** Javaheri Residence	**LOG NO.**	**LDB-1**
1 IN-PLACE SAMPLE			
MODIFIED CALIFORNIA SAMPLE	**SITE LOCATION:** 2072 Via Casa Alta, La Jolla, CA		
H IN-PLACE HAND-DRIVE SAMPLE		**FIGURE NO.**	**IIIc**
STANDARD PENETRATION TEST			

EXPLORATORY TRENCH T-1

A-A' East Side Wall

LEGEND

Qsw	SLOPEWASH:	SANDY FAT CLAY (CH). Very stiff. Moist. Dark brown.
Qvop¹ 11	Very Old Paralic Deposits, Unit 11 ARGILLIC TERRACE MATERIALS:	LEAN CLAY/CLAYEY SAND (CL/SC). Fine-to coarse-grained sand. Dense/very stiff. Slightly moist. Dark reddish brown.
Qvop² 11	Very Old Paralic Deposits, Unit 11 TERRACE MATERIALS:	SILTY GRAVEL WITH SAND (GM). Fine - to coarse - grained. Dense. Dry to slightly moist. Reddish brown to orangish brown. Approx. 40% rounded fine to coarse gravels and cobbles up to 6" diameter.
Ta¹	ARDATH SHALE:	SANDY SILT (ML). Very stiff. Slightly moist. Yellowish brown to light gray to reddish brown. Blocky structure. Friable.
Ta²	ARDATH SHALE:	LEAN CLAY (CL). Very stiff to hard. Slightly moist. Yellowish brown to light gray to orangish brown. Blocky structure. Friable. Indurated.
Kcs	CABRILLO SANDSTONE:	SILTY SAND (SM). Fine - to medium - grained. Dense. Slightly moist. Yellowish brown. Micaceous.

Approximate Geologic Contact

3 Carbon 14 Sample Location

Bulk Sample Location

NOTE: This Exploratory Trench 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.

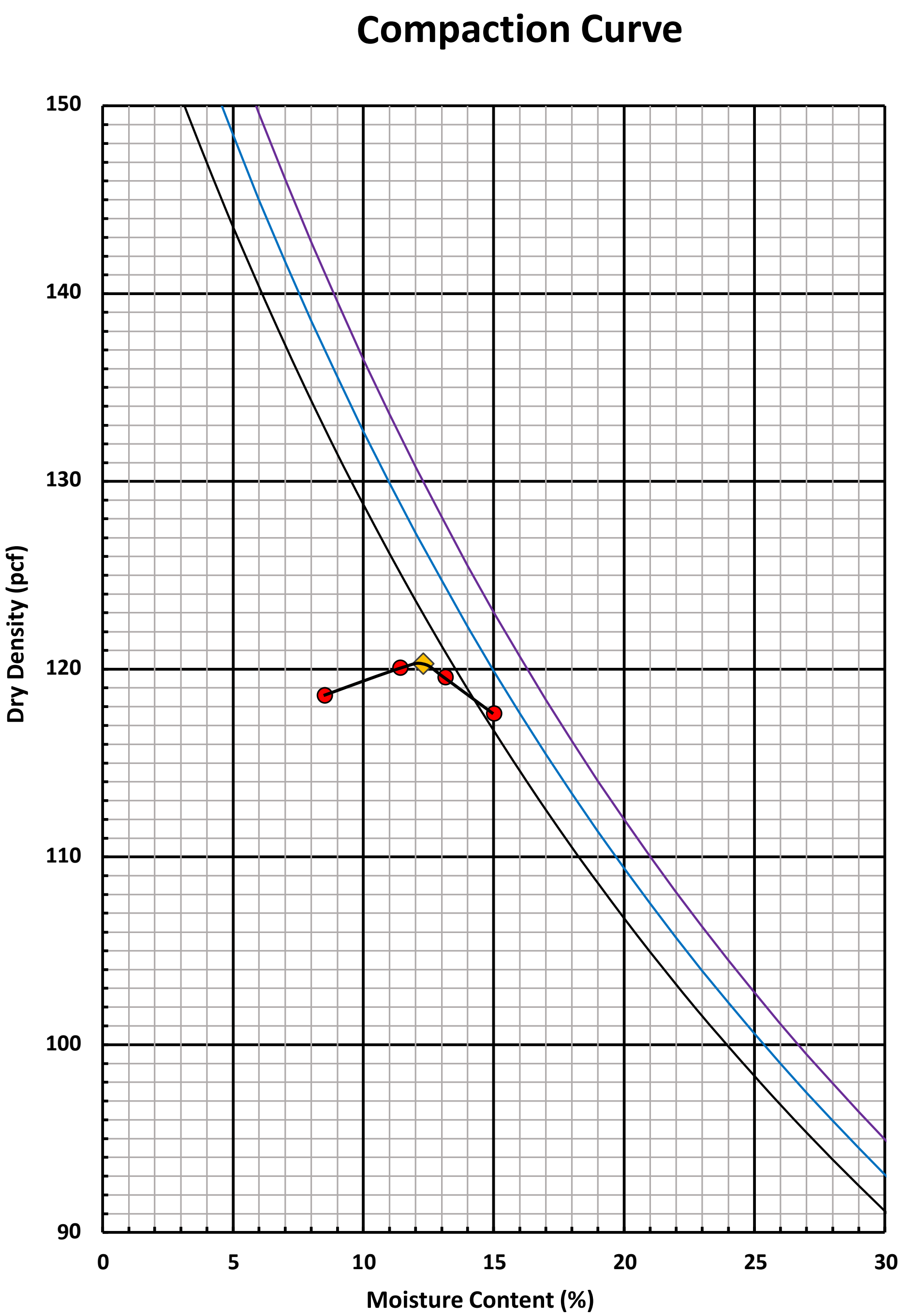
Javaheri Residence
2072 Via Casa Alta
La Jolla, CA.
Figure No. IIId
Job No. 21-13556

Geotechnical Exploration, Inc.

July 2022

16-11286-1-AA

Laboratory Test Results



Source of Material	**LDB-1**
Depth	**30-31 FT.**
Description of Material	**SILTY SAND (SM) Yellowish brown**
Test Method	**ASTM D1557 Method A**

TEST RESULTS

Maximum Dry Density (PCF)	**120.3**
Optimum Moisture Content (%)	**12.3**
Expansion Index (EI)	
% Passing #200	**34%**

Curves of 100% Saturation for

Specific Gravity equal to:

2.80

2.70

2.60

Geotechnical Exploration, Inc.

MOISTURE-DENSITY RELATIONSHIP

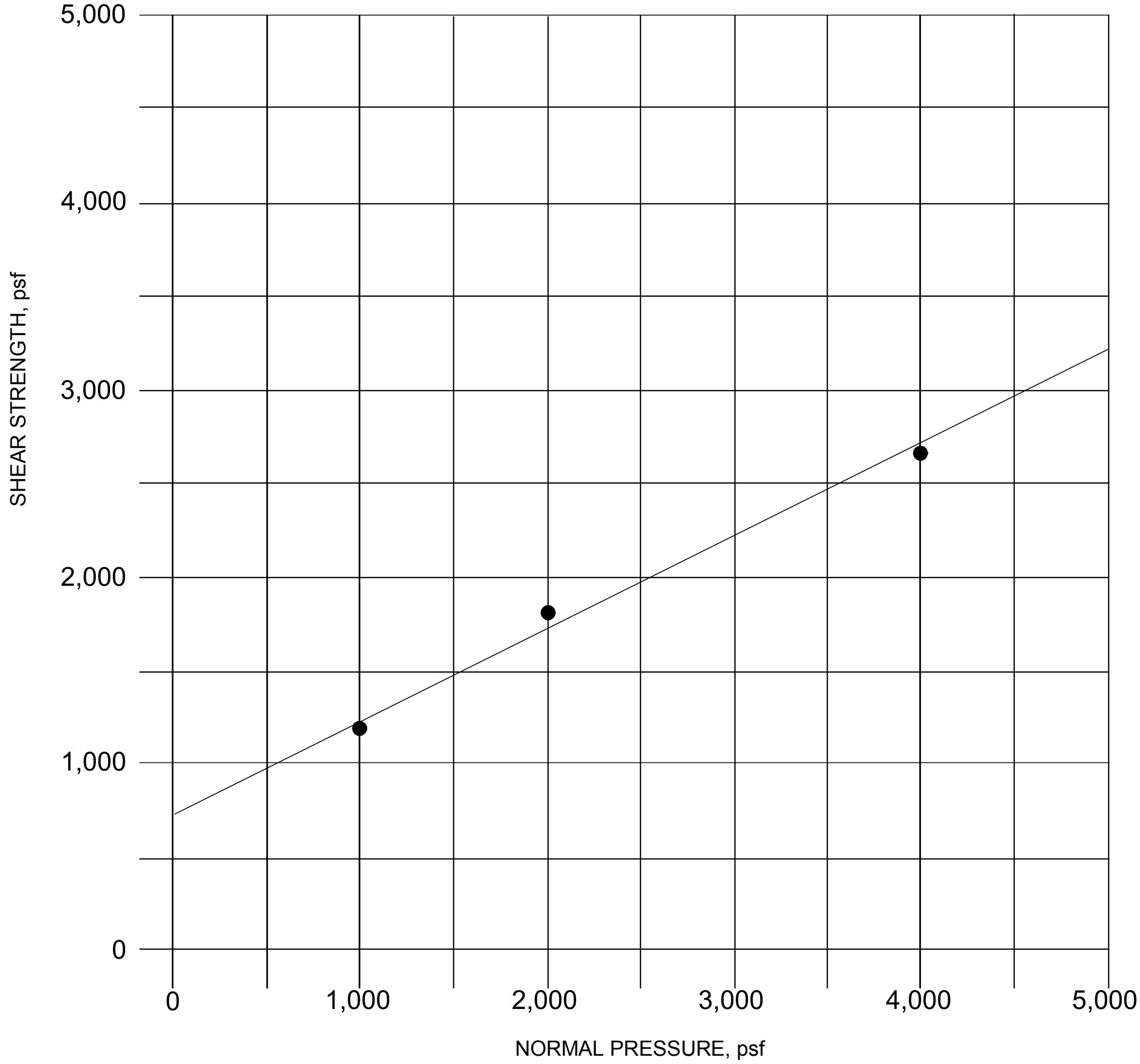
Figure Number: IVa

Job Name: Javaheri Residence

Site Location: 2072 Via Casa Alta, La Jolla

Job Number: 21-13556

Laboratory Test Results



	Specimen I.D.	Classification	γ_d	MC%	c	ϕ
●	**T-1 @ 29-29.5'**	**SILTY SAND (SM), Undisturbed, Saturated**			**756**	**25.9**

Geotechnical Exploration, Inc.

DIRECT SHEAR TEST

Figure Number: IVb
Job Name: Javaheri Residence
Site Location: 2072 Via Casa Alta, La Jolla, CA
Job Number: 21-13556

EXCERPT FROM

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

By

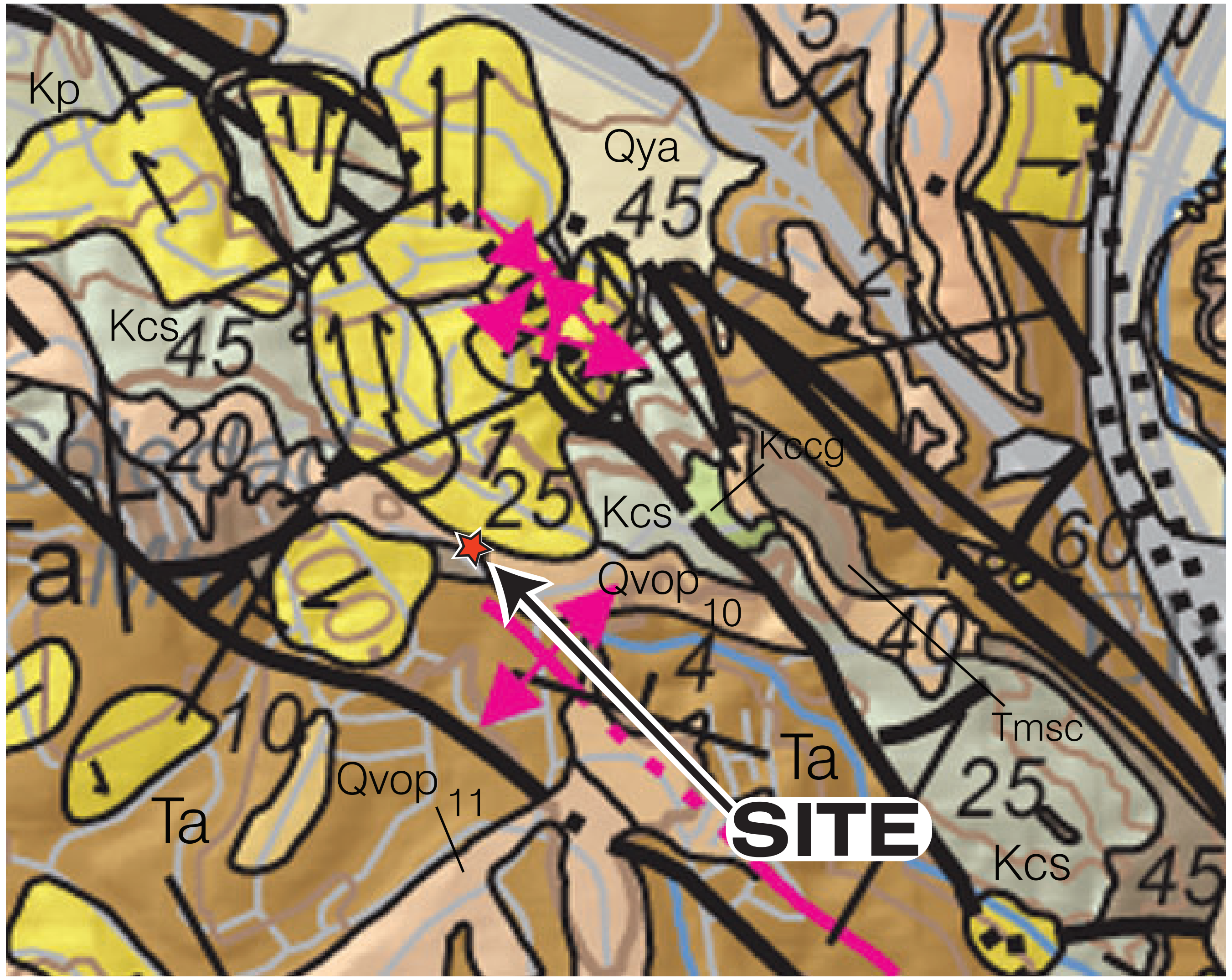
Michael P. Kennedy[1] and Siang S. Tan[1]

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



Javaheri Residence
2072 Via Casa Alta
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.

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

Qya	Young Alluvial Deposits
$Qvop_{11}$	Very Old Paralic Deposits, unit 11
$Qvop_{10}$	Very Old Paralic Deposits, unit 10
Ta	Ardath Shale
Tmsc	Mount Soledad Formation - cobble conglomerate
Kcs	Cabrillo Formation - Sandstone
Kccg	Cabrillo Formation - cobble conglomerate
Kp	Point Loma Formation

Figure No. V
Job No. 21-13556

Geotechnical Exploration, Inc.

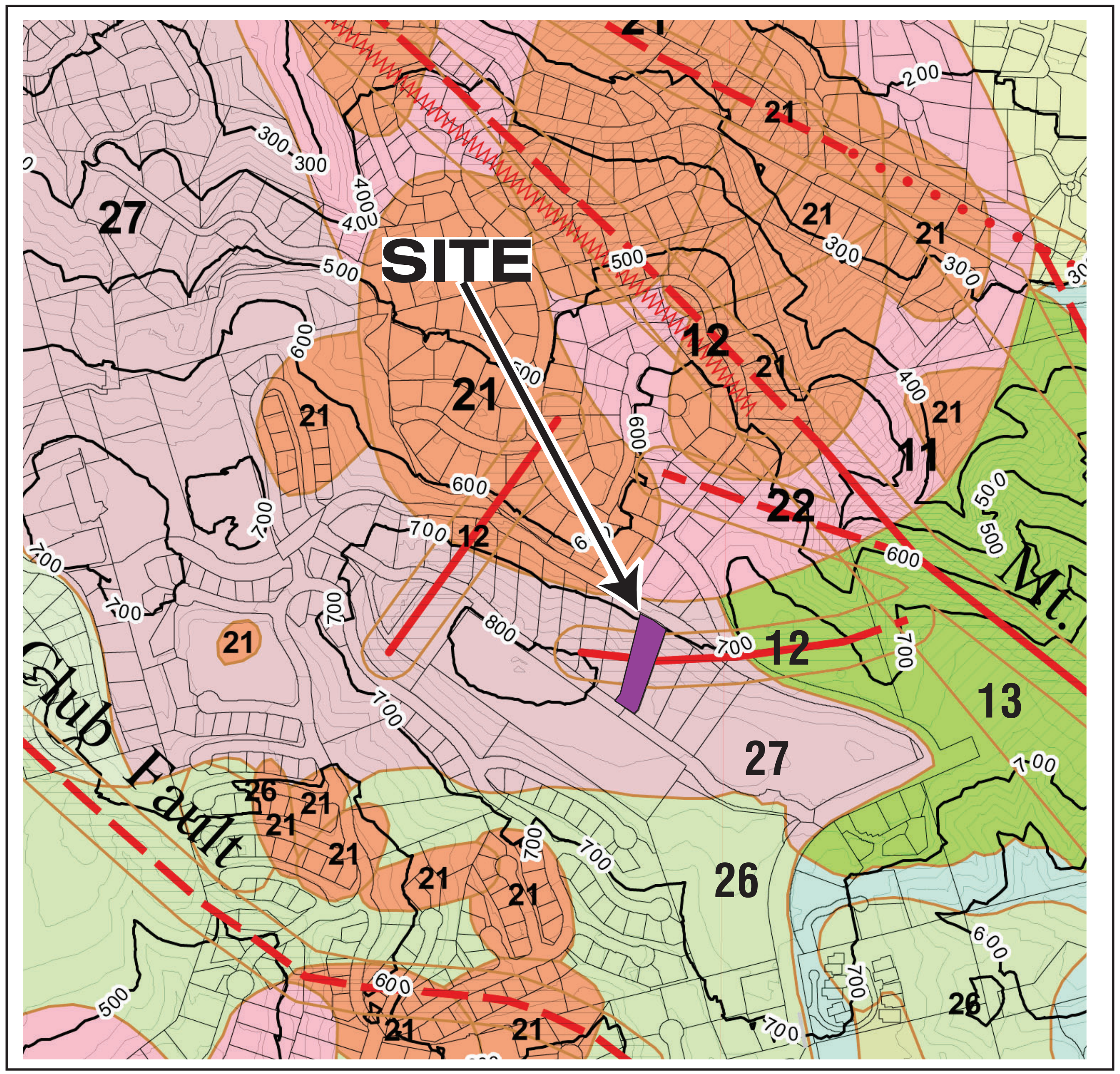
June 2022

JAVAHERI-2008-GEO.ai

Geologic Hazards Map Excerpt
from City of San Diego
Seismic Safety Study
Geologic Hazards and Fault Map
Sheet 29

Development Services Department

DATE: 4/3/2008



Javaheri Residence
2072 Via Casa Alta
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

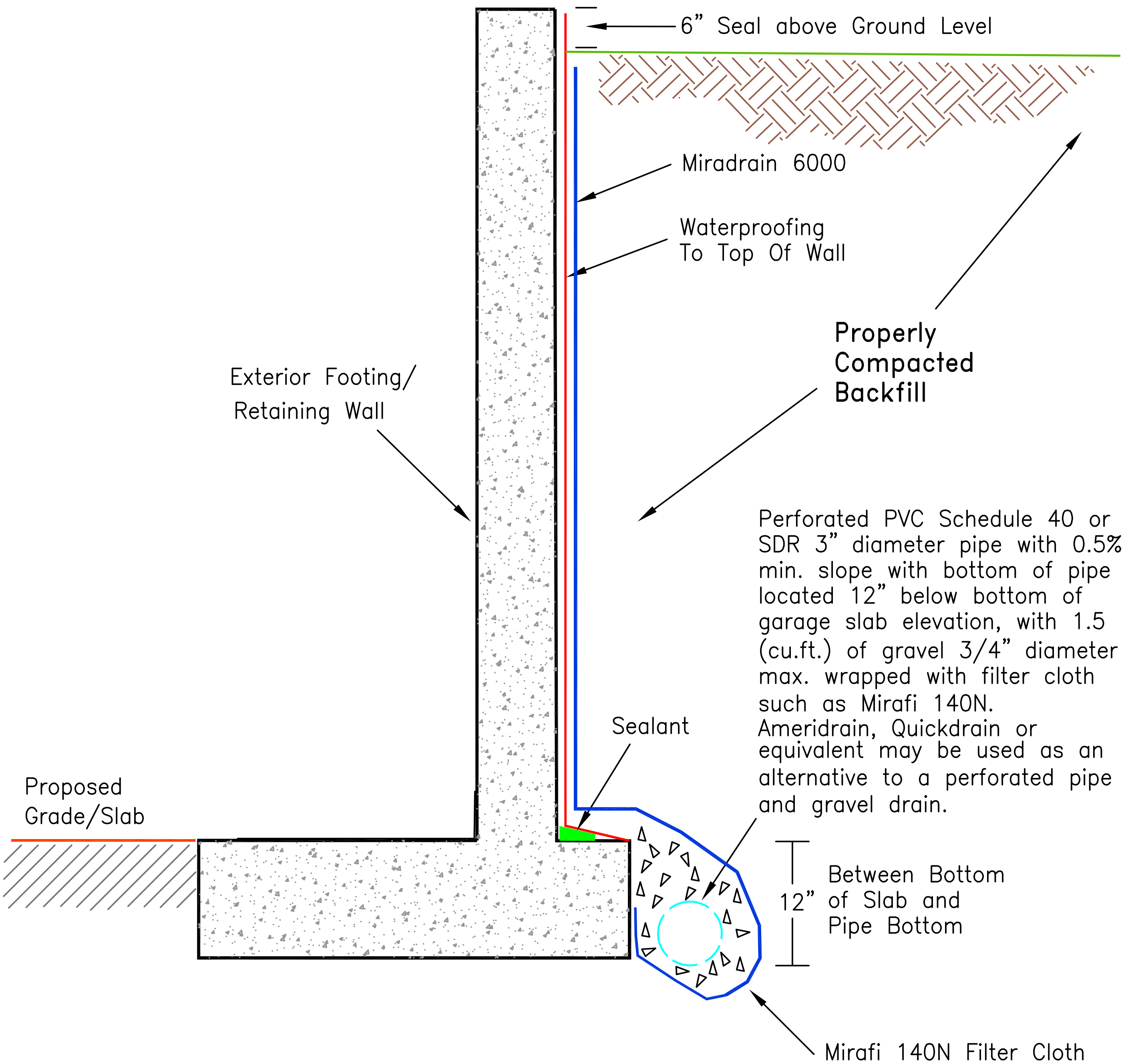
Figure No. VI
Job No. 21-13556

Geotechnical Exploration, Inc.

June 2022

JAVAHERI-SEIS-.ai

SUBGRADE RETAINING WALL DRAINAGE RECOMMENDATIONS



NOT TO SCALE

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

Figure No. VII
Job No. 21-13556

Geotechnical Exploration, Inc.

June 2022

21-13556-VII

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, lean 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

OSHPD

Latitude, Longitude: 32.8397, -117.2511

Date	7/22/2022, 10:32:17 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	C - Very Dense Soil and Soft Rock

Type	Value	Description
S_S	1.416	MCE_R ground motion. (for 0.2 second period)
S_1	0.494	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.699	Site-modified spectral acceleration value
S_{M1}	0.742	Site-modified spectral acceleration value
S_{DS}	1.133	Numeric seismic design value at 0.2 second SA
S_{D1}	0.494	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	1.2	Site amplification factor at 0.2 second
F_v	1.5	Site amplification factor at 1.0 second
PGA	0.647	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.777	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.416	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.637	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.272	Factored deterministic acceleration value. (0.2 second)
S1RT	0.494	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.558	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.8	Factored deterministic acceleration value. (1.0 second)
PGAd	0.942	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.865	Mapped value of the risk coefficient at short periods
C_{R1}	0.886	Mapped value of the risk coefficient at a period of 1 s

APPENDIX C

Radiocarbon Age Dating Report

Beta Analytic, Inc.
4985 SW 74th Court
Miami, FL 33155 USA
Tel: 305-667-5167
Fax: 305-663-0964
info@betalabservices.com

ISO/IEC 17025:2017-Accredited Testing Laboratory

June 06, 2022

Mr. Jay Heiser
Geotechnical Exploration, Inc.
7420 Trade Street
San Diego, CA 92121
United States

RE: Radiocarbon Dating Results

Dear Mr. Heiser,

Enclosed are the radiocarbon dating results for four samples recently sent to us. As usual, the method of analysis is listed on the report with the results and calibration data is provided where applicable. The Conventional Radiocarbon Ages have all been corrected for total fractionation effects and where applicable, calibration was performed using 2020 calibration databases (cited on the graph pages).

The web directory containing the table of results and PDF download also contains pictures, a cvs spreadsheet download option and a quality assurance report containing expected vs. measured values for 3-5 working standards analyzed simultaneously with your samples.

Reported results are accredited to ISO/IEC 17025:2017 Testing Accreditation PJLA #59423 standards and all chemistry was performed here in our laboratory and counted in our own accelerators here. Since Beta is not a teaching laboratory, only graduates trained to strict protocols of the ISO/IEC 17025:2017 Testing Accreditation PJLA #59423 program participated in the analyses.

As always Conventional Radiocarbon Ages and sigmas are rounded to the nearest 10 years per the conventions of the 1977 International Radiocarbon Conference. When counting statistics produce sigmas lower than +/- 30 years, a conservative +/- 30 BP is cited for the result unless otherwise requested. The reported d13C values were measured separately in an IRMS (isotope ratio mass spectrometer). They are NOT the AMS d13C which would include fractionation effects from natural, chemistry and AMS induced sources.

When interpreting the results, please consider any communications you may have had with us regarding the samples.

Thank you for prepaying the analyses. As always, if you have any questions or would like to discuss the results, don't hesitate to contact us.

Sincerely,

Digital signature on file

Ronald E. Hatfield President

Beta Analytic, Inc.
4985 SW 74th Court
Miami, FL 33155 USA
Tel: 305-667-5167
Fax: 305-663-0964
info@betalabservices.com

ISO/IEC 17025:2017-Accredited Testing Laboratory

REPORT OF RADIOCARBON DATING ANALYSES

Jay Heiser — Report Date: June 06, 2022

Geotechnical Exploration, Inc. — Material Received: May 20, 2022

Laboratory Number	Sample Code Number	Conventional Radiocarbon Age (BP) or Percent Modern Carbon (pMC) & Stable Isotopes	
Beta - 628129	**21-13556-1**	**3530 +/- 30 BP**	IRMS δ13C: -23.8 o/oo

(94.0%)	**1945 - 1765 cal BC**	**(3894 - 3714 cal BP)**
(1.4%)	**1759 - 1750 cal BC**	**(3708 - 3699 cal BP)**

Submitter Material: Organic Sediment/Gyttja
Pretreatment: (organic sediment) acid washes
Analyzed Material: Organic sediment
Analysis Service: AMS-Standard delivery
Percent Modern Carbon: 64.44 +/- 0.24 pMC
Fraction Modern Carbon: 0.6444 +/- 0.0024
D14C: -355.60 +/- 2.41 o/oo
Δ14C: -361.19 +/- 2.41 o/oo (1950:2022)
Measured Radiocarbon Age: (without d13C correction): 3510 +/- 30 BP
Calibration: BetaCal4.20: HPD method: INTCAL20

Results are ISO/IEC-17025:2017 accredited. No sub-contracting or student labor was used in the analyses. All work was done at Beta in 4 in-house NEC accelerator mass spectrometers and 4 Thermo IRMSs. The "Conventional Radiocarbon Age" was calculated using the Libby half-life (5568 years), is corrected for total isotopic fraction and was used for calendar calibration where applicable. The Age is rounded to the nearest 10 years and is reported as radiocarbon years before present (BP), "present" = AD 1950. Results greater than the modern reference are reported as percent modern carbon (pMC). The modern reference standard was 95% the 14C signature of NIST SRM-4990C (oxalic acid). Quoted errors are 1 sigma counting statistics. Calculated sigmas less than 30 BP on the Conventional Radiocarbon Age are conservatively rounded up to 30. d13C values are on the material itself (not the AMS d13C). d13C and d15N values are relative to VPDB. References for calendar calibrations are cited at the bottom of calibration graph pages.

Beta Analytic, Inc.
4985 SW 74th Court
Miami, FL 33155 USA
Tel: 305-667-5167
Fax: 305-663-0964
info@betalabservices.com

Beta Analytic
TESTING LABORATORY

ISO/IEC 17025:2017-Accredited Testing Laboratory

REPORT OF RADIOCARBON DATING ANALYSES

Jay Heiser — Report Date: June 06, 2022

Geotechnical Exploration, Inc. — Material Received: May 20, 2022

Laboratory Number	Sample Code Number	Conventional Radiocarbon Age (BP) or Percent Modern Carbon (pMC) & Stable Isotopes	
Beta - 628130	**21-13556-2**	**3060 +/- 30 BP**	IRMS δ13C: -24.1 o/oo

(91.0%)	**1412 - 1257 cal BC**	**(3361 - 3206 cal BP)**
(4.4%)	**1247 - 1227 cal BC**	**(3196 - 3176 cal BP)**

Submitter Material: Organic Sediment/Gyttja
Pretreatment: (organic sediment) acid washes
Analyzed Material: Organic sediment
Analysis Service: AMS-Standard delivery
Percent Modern Carbon: 68.32 +/- 0.26 pMC
Fraction Modern Carbon: 0.6832 +/- 0.0026
D14C: -316.78 +/- 2.55 o/oo
Δ14C: -322.70 +/- 2.55 o/oo (1950:2022)
Measured Radiocarbon Age: (without d13C correction): 3040 +/- 30 BP
Calibration: BetaCal4.20: HPD method: INTCAL20

Results are ISO/IEC-17025:2017 accredited. No sub-contracting or student labor was used in the analyses. All work was done at Beta in 4 in-house NEC accelerator mass spectrometers and 4 Thermo IRMSs. The "Conventional Radiocarbon Age" was calculated using the Libby half-life (5568 years), is corrected for total isotopic fraction and was used for calendar calibration where applicable. The Age is rounded to the nearest 10 years and is reported as radiocarbon years before present (BP), "present" = AD 1950. Results greater than the modern reference are reported as percent modern carbon (pMC). The modern reference standard was 95% the 14C signature of NIST SRM-4990C (oxalic acid). Quoted errors are 1 sigma counting statistics. Calculated sigmas less than 30 BP on the Conventional Radiocarbon Age are conservatively rounded up to 30. d13C values are on the material itself (not the AMS d13C). d13C and d15N values are relative to VPDB. References for calendar calibrations are cited at the bottom of calibration graph pages.

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ISO/IEC 17025:2017-Accredited Testing Laboratory

REPORT OF RADIOCARBON DATING ANALYSES

Jay Heiser	Report Date: June 06, 2022
Geotechnical Exploration, Inc.	Material Received: May 20, 2022

Laboratory Number	Sample Code Number	Conventional Radiocarbon Age (BP) or Percent Modern Carbon (pMC) & Stable Isotopes	
Beta - 628131	**21-13556-3**	**2090 +/- 30 BP**	IRMS δ13C: -24.3 o/oo

(91.2%)	**178 - 38 cal BC**	**(2127 - 1987 cal BP)**
(2.8%)	**13 cal BC - 4 cal AD**	**(1962 - 1946 cal BP)**
(1.3%)	**196 - 185 cal BC**	**(2145 - 2134 cal BP)**

Submitter Material: Organic Sediment/Gyttja
Pretreatment: (organic sediment) acid washes
Analyzed Material: Organic sediment
Analysis Service: AMS-Standard delivery
Percent Modern Carbon: 77.09 +/- 0.29 pMC
Fraction Modern Carbon: 0.7709 +/- 0.0029
D14C: -229.09 +/- 2.88 o/oo
Δ14C: -235.77 +/- 2.88 o/oo (1950:2022)
Measured Radiocarbon Age: (without d13C correction): 2080 +/- 30 BP
Calibration: BetaCal4.20: HPD method: INTCAL20

Results are ISO/IEC-17025:2017 accredited. No sub-contracting or student labor was used in the analyses. All work was done at Beta in 4 in-house NEC accelerator mass spectrometers and 4 Thermo IRMSs. The "Conventional Radiocarbon Age" was calculated using the Libby half-life (5568 years), is corrected for total isotopic fraction and was used for calendar calibration where applicable. The Age is rounded to the nearest 10 years and is reported as radiocarbon years before present (BP), "present" = AD 1950. Results greater than the modern reference are reported as percent modern carbon (pMC). The modern reference standard was 95% the 14C signature of NIST SRM-4990C (oxalic acid). Quoted errors are 1 sigma counting statistics. Calculated sigmas less than 30 BP on the Conventional Radiocarbon Age are conservatively rounded up to 30. d13C values are on the material itself (not the AMS d13C). d13C and d15N values are relative to VPDB. References for calendar calibrations are cited at the bottom of calibration graph pages.

Calibration of Radiocarbon Age to Calendar Years

(High Probability Density Range Method (HPD): INTCAL20)

(Variables: d13C = -23.8 o/oo)

Laboratory number **Beta-628129**

Conventional radiocarbon age **3530 ± 30 BP**

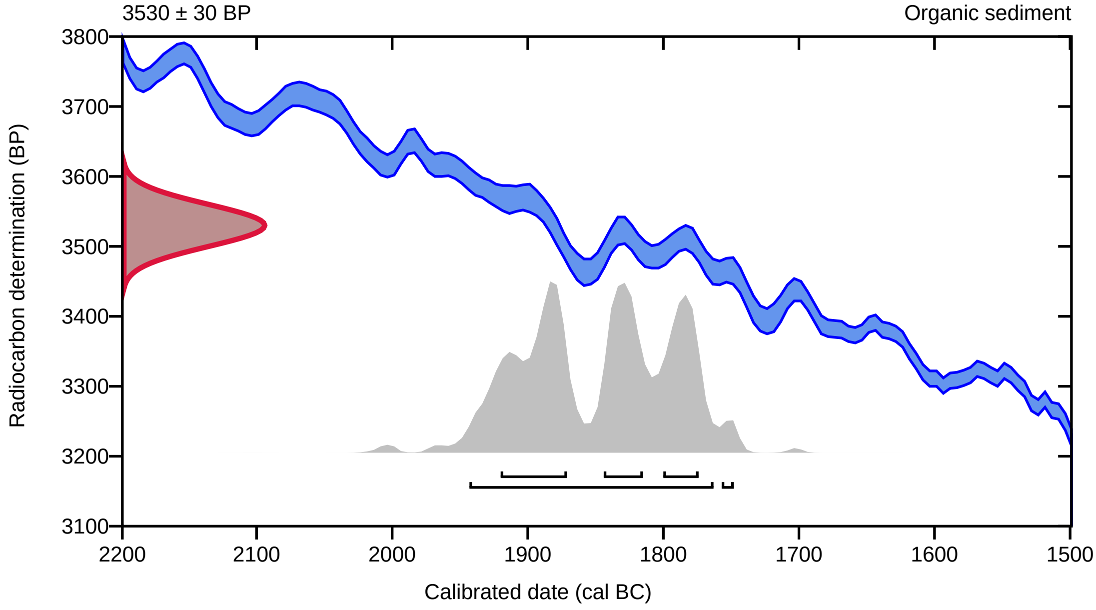
95.4% probability

(94%)	1945 - 1765 cal BC	(3894 - 3714 cal BP)
(1.4%)	1759 - 1750 cal BC	(3708 - 3699 cal BP)

68.2% probability

(28.6%)	1922 - 1873 cal BC	(3871 - 3822 cal BP)
(21.3%)	1846 - 1817 cal BC	(3795 - 3766 cal BP)
(18.3%)	1802 - 1776 cal BC	(3751 - 3725 cal BP)

21-13556-1



Database used

INTCAL20

References

References to Probability Method

Bronk Ramsey, C. (2009). Bayesian analysis of radiocarbon dates. Radiocarbon, 51(1), 337-360.

References to Database INTCAL20

Reimer, et al., 2020, Radiocarbon 62(4):725-757.

Beta Analytic Radiocarbon Dating Laboratory

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Calibration of Radiocarbon Age to Calendar Years

(High Probability Density Range Method (HPD): INTCAL20)

(Variables: d13C = -24.1 o/oo)

Laboratory number **Beta-628130**

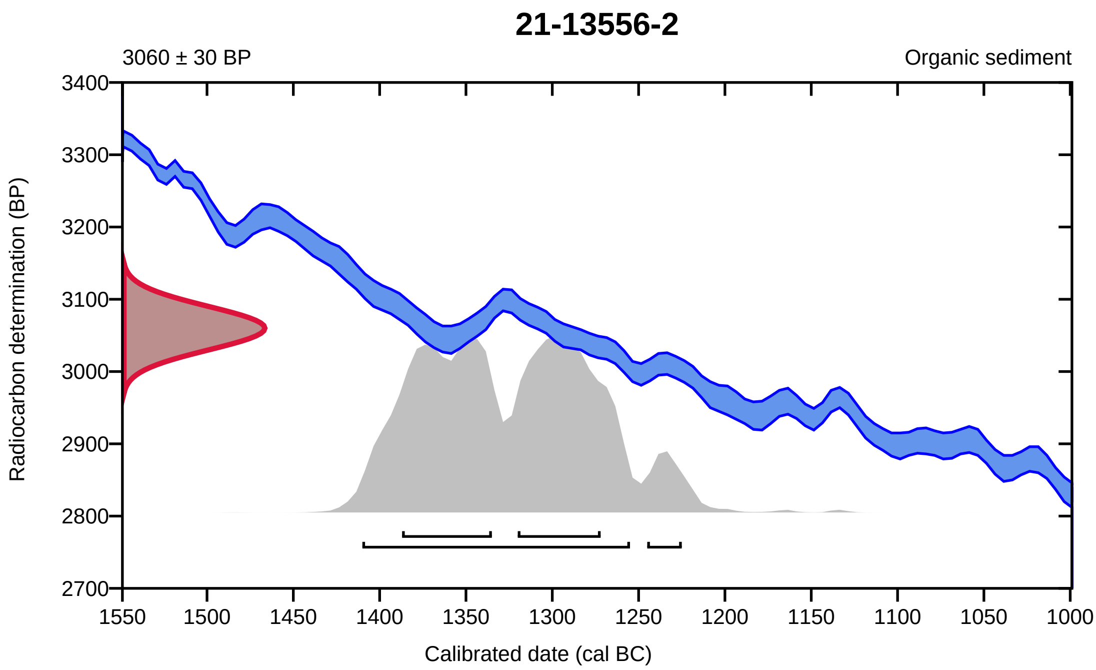
Conventional radiocarbon age **3060 ± 30 BP**

95.4% probability

(91%)	1412 - 1257 cal BC	(3361 - 3206 cal BP)
(4.4%)	1247 - 1227 cal BC	(3196 - 3176 cal BP)

68.2% probability

(35.9%)	1389 - 1337 cal BC	(3338 - 3286 cal BP)
(32.3%)	1322 - 1274 cal BC	(3271 - 3223 cal BP)



Database used

INTCAL20

References

References to Probability Method

Bronk Ramsey, C. (2009). Bayesian analysis of radiocarbon dates. Radiocarbon, 51(1), 337-360.

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Calibration of Radiocarbon Age to Calendar Years

(High Probability Density Range Method (HPD): INTCAL20)

(Variables: d13C = -24.3 o/oo)

Laboratory number **Beta-628131**

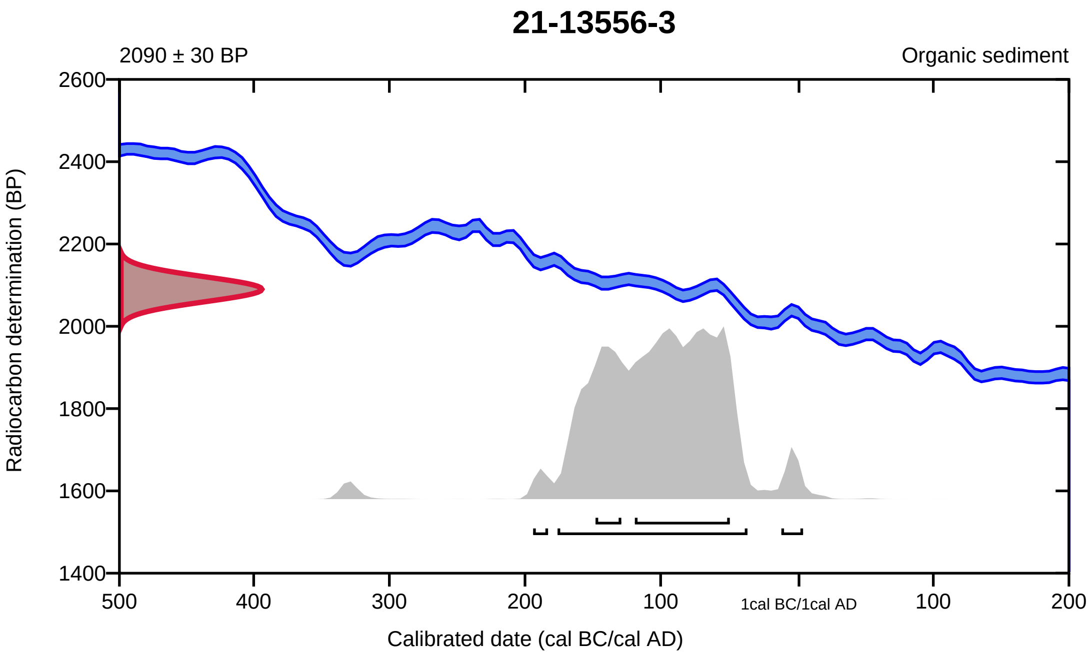
Conventional radiocarbon age **2090 ± 30 BP**

95.4% probability

(91.2%)	178 - 38 cal BC	(2127 - 1987 cal BP)
(2.8%)	13 cal BC - 4 cal AD	(1962 - 1946 cal BP)
(1.3%)	196 - 185 cal BC	(2145 - 2134 cal BP)

68.2% probability

(54.6%)	121 - 51 cal BC	(2070 - 2000 cal BP)
(13.6%)	150 - 131 cal BC	(2099 - 2080 cal BP)



Database used

INTCAL20

References

References to Probability Method

Bronk Ramsey, C. (2009). Bayesian analysis of radiocarbon dates. Radiocarbon, 51(1), 337-360.

References to Database INTCAL20

Reimer, et al., 2020, Radiocarbon 62(4):725-757.

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Quality Assurance Report

This report provides the results of reference materials used to validate radiocarbon analyses prior to reporting. Known-value reference materials were analyzed quasi-simultaneously with the unknowns. Results are reported as expected values vs measured values. Reported values are calculated relative to NISTSRM-1990C and corrected for isotopic fractionation. Results are reported using the direct analytical measure percent modern carbon (pMC) with one relative standard deviation. Agreement between expected and measured values is taken as being within 2 sigma agreement (error x 2) to account for total laboratory error.

Report Date: June 09, 2022
Submitter: Mr. Jay Heiser

QA MEASUREMENTS

Reference 1

Expected Value: 129.41 +/- 0.06 pMC
Measured Value: 129.44 +/- 0.35 pMC
Agreement: Accepted

Reference 2

Expected Value: 0.42 +/- 0.04 pMC
Measured Value: 0.42 +/- 0.04 pMC
Agreement: Accepted

Reference 3

Expected Value: 96.69 +/- 0.50 pMC
Measured Value: 97.40 +/- 0.29 pMC
Agreement: Accepted

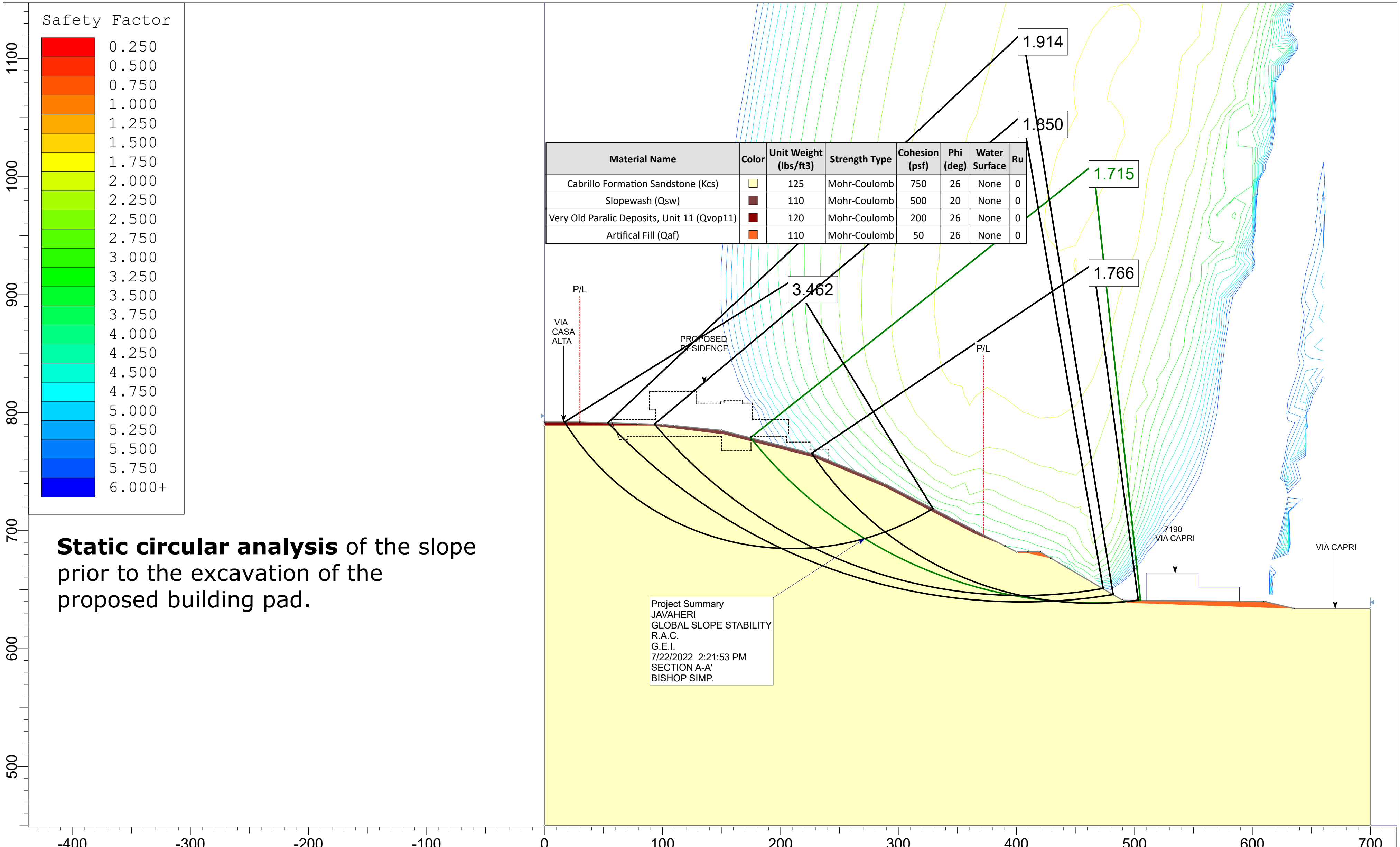
COMMENT: All measurements passed acceptance tests.

Validation: Digital signature on file

Date: June 09, 2022

APPENDIX D

Slope Stability Analysis



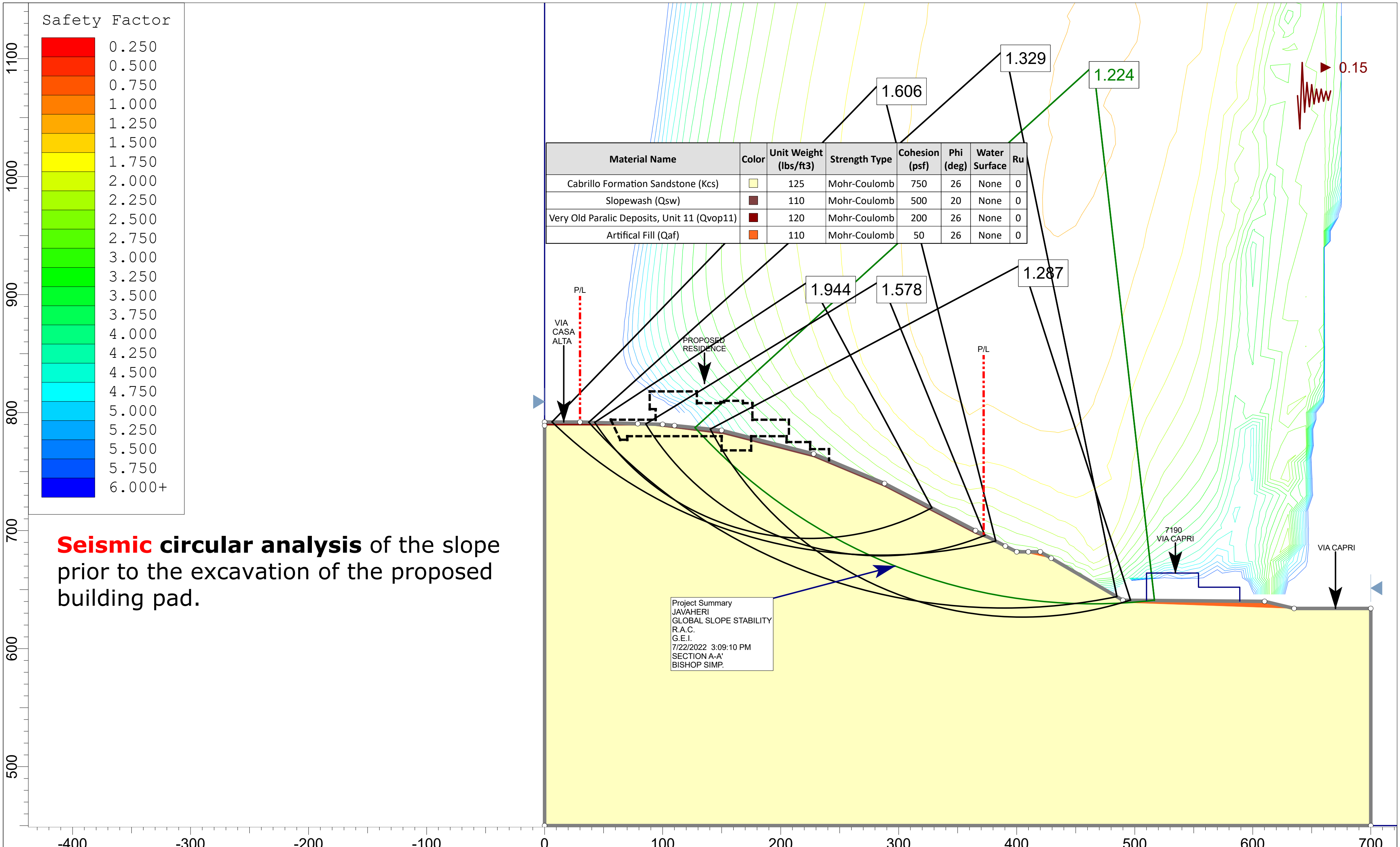
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Cabrillo Formation Sandstone (Kcs)		125	Mohr-Coulomb	750	26	None	0
Slopewash (Qsw)		110	Mohr-Coulomb	500	20	None	0
Very Old Paralic Deposits, Unit 11 (Qvop11)		120	Mohr-Coulomb	200	26	None	0
Artifical Fill (Qaf)		110	Mohr-Coulomb	50	26	None	0

Static circular analysis of the slope prior to the excavation of the proposed building pad.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	JAVAHERI		**SECTION A-A'**
Analysis Description	GLOBAL SLOPE STABILITY		
Drawn By R.A.C.	*Scale* 1:1350	*Company* G.E.I.	
Date 7/22/2022 2:21:53 PM		*File Name* JOB NO. 21-13556_S(A)_01.slim	



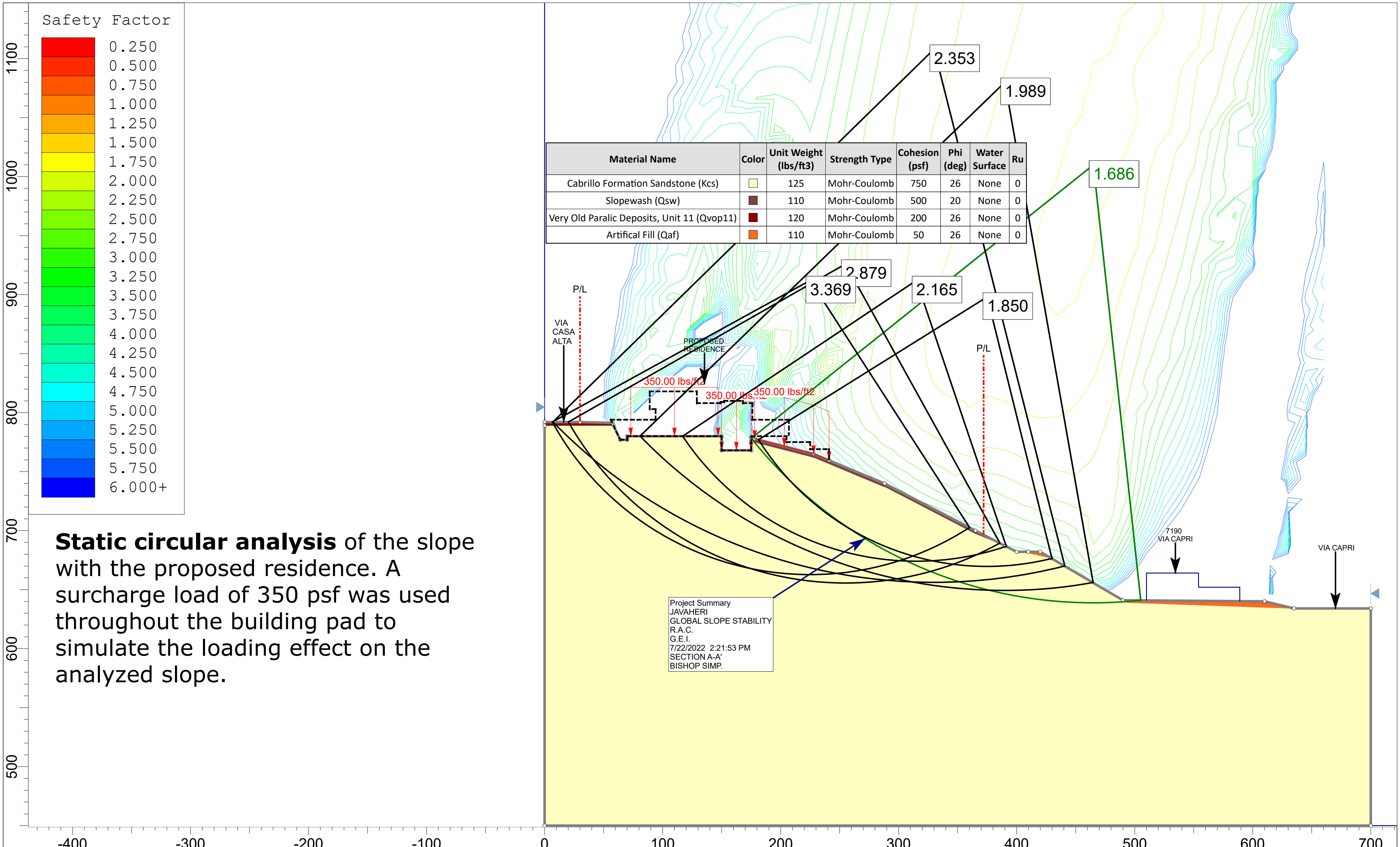
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Cabrillo Formation Sandstone (Kcs)		125	Mohr-Coulomb	750	26	None	0
Slopewash (Qsw)		110	Mohr-Coulomb	500	20	None	0
Very Old Paralic Deposits, Unit 11 (Qvop11)		120	Mohr-Coulomb	200	26	None	0
Artifical Fill (Qaf)		110	Mohr-Coulomb	50	26	None	0

Seismic circular analysis of the slope prior to the excavation of the proposed building pad.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	JAVAHERI	**SECTION A-A'**	
Analysis Description	GLOBAL SLOPE STABILITY		
Drawn By R.A.C.	*Scale* 1:1350	*Company* G.E.I.	
Date 7/22/2022 3:09:10 PM		*File Name* JOB NO. 21-13556_S(A)_01w_0.15gSHAKE.slim	



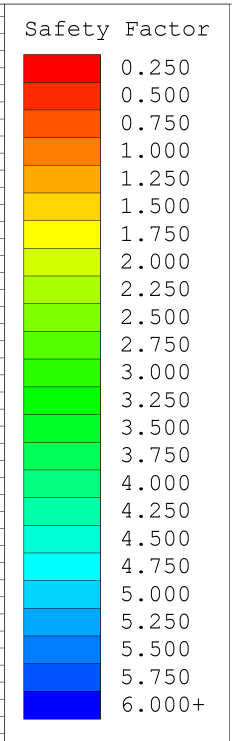
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Cabrillo Formation Sandstone (Kcs)		125	Mohr-Coulomb	750	26	None	0
Slopewash (Qsw)		110	Mohr-Coulomb	500	20	None	0
Very Old Paralic Deposits, Unit 11 (Qvop11)		120	Mohr-Coulomb	200	26	None	0
Artifical Fill (Qaf)		110	Mohr-Coulomb	50	26	None	0

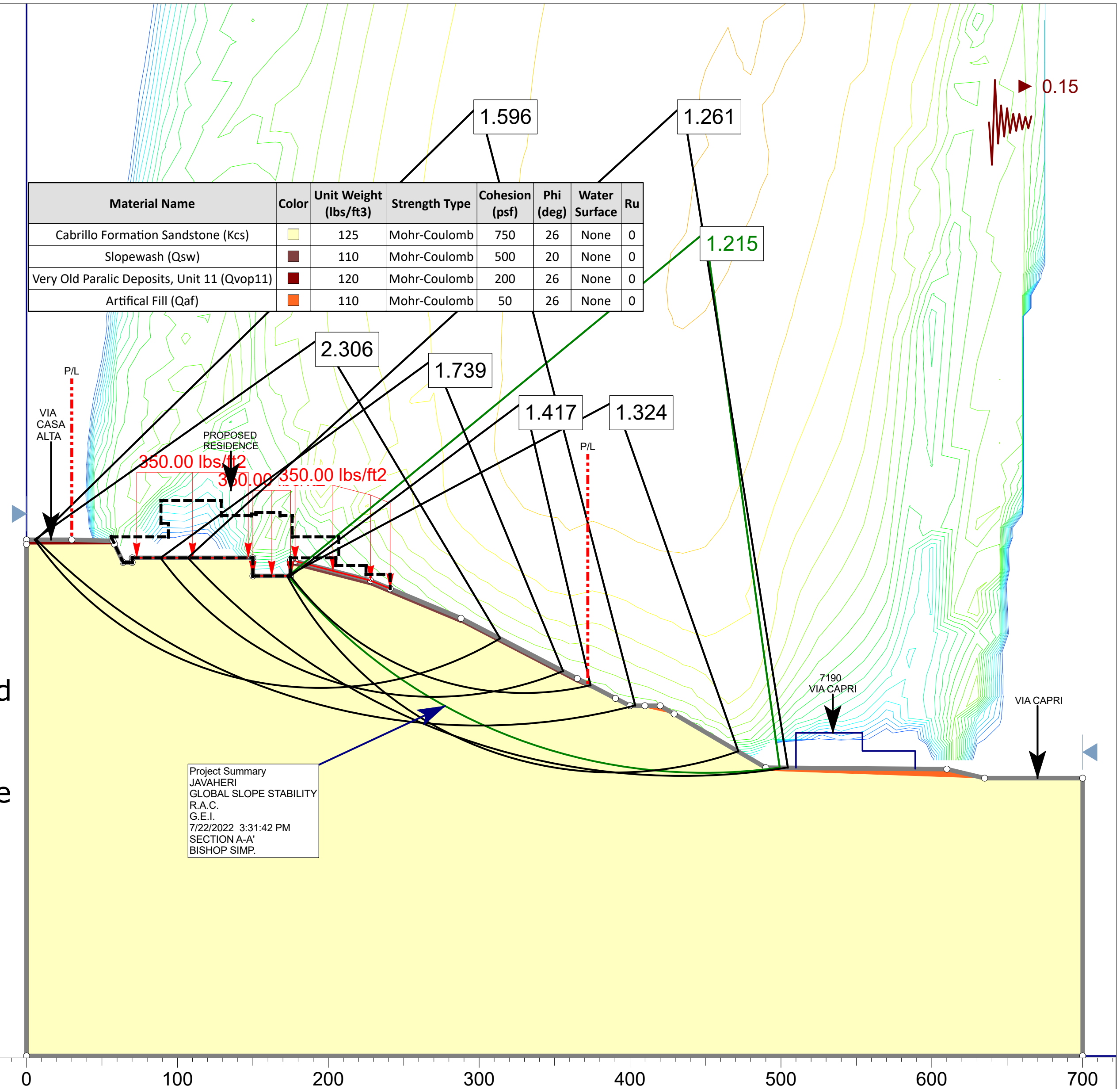
Static circular analysis of the slope with the proposed residence. A surcharge load of 350 psf was used throughout the building pad to simulate the loading effect on the analyzed slope.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	JAVAHERI		**SECTION A-A'**
Analysis Description	GLOBAL SLOPE STABILITY		
Drawn By	R.A.C.	*Scale* 1:1350	*Company* G.E.I.
Date	7/22/2022 2:21:53 PM	*File Name*	JOB NO. 21-13556_S(A)_02.slim





Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
Cabrillo Formation Sandstone (Kcs)		125	Mohr-Coulomb	750	26	None	0
Slopewash (Qsw)		110	Mohr-Coulomb	500	20	None	0
Very Old Paralic Deposits, Unit 11 (Qvop11)		120	Mohr-Coulomb	200	26	None	0
Artifical Fill (Qaf)		110	Mohr-Coulomb	50	26	None	0

Seismic circular analysis of the proposed slope with the proposed residence. A surcharge load of 350 psf was used throughout the building pad to simulate the loading effect on the analyzed slope.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	JAVAHERI		**SECTION A-A'**
Analysis Description	GLOBAL SLOPE STABILITY		
Drawn By R.A.C.	*Scale* 1:1350	*Company* G.E.I.	
Date 7/22/2022 3:31:42 PM		*File Name* JOB NO. 21-13556_S(A)_02w_0.15gSHAKE.slim	

SURFICIAL FAILURE

Υ_t	Υ_w	Υ'	z_w
pcf	pcf	pcf	ft
110	62.4	47.6	3.28

EQUATION 1

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

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.,

SECTION A-A'				
SOIL TYPE	**c (psf)**	**ϕ'(°)**	**α(°)**	**F.O.S.**
Slopewash (Qsw)	500	20	27	3.735

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
Υ_t	The total unit weight (Soil with moisture)
Υ_w	The unit weight of the water
Υ'	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.

Slopes with Factor of Safety values **ABOVE** 1.50 are stable.