Appendix D

Report of Geotechnical Investigation Update

REPORT OF GEOTECHNICAL INVESTIGATION UPDATE

Proposed Breeze Townhome Project
1200 Block of S. Nevada Street
Oceanside, California

JOB NO. 15-10805
28 September 2018

Prepared for:

Oceanside-Nevada, LP

Geotechnical Exploration, Inc.

SOIL AND FOUNDATION ENGINEERING • GROUNDWATER • ENGINEERING GEOLOGY

28 September 2018

Job No. 15-10805

Oceanside-Nevada, LP
P.O. Box 531
Rancho Santa Fe, CA 92067
Attn: Mr. Howard A. Jacobs

Subject: **Report of Geotechnical Investigation Update**
Proposed Breeze Townhome Project
A.P.N. 152-121-06, 152-123-05 and 152-320-11
1200 Block of S. Nevada Street
Oceanside, California

Dear Mr. Jacobs:

In accordance with your request and our agreement, ***Geotechnical Exploration, Inc.*** has prepared this update of our previous geotechnical report for the geotechnical and general geologic conditions at the subject property in Oceanside. We understand that the site is being developed with luxury townhomes and associated improvements. Our most recent field exploration work was performed on July 15, 2016. The original field work was performed on July 1, 2015. In addition, we reviewed our previous report, as well as previous geotechnical reports for the property prepared by Vinje & Middleton Engineering, Inc., dated November 26, 2001, March 7, 2003, and September 13, 2004.

In our opinion, if the conclusions and recommendations presented in this report are implemented during project planning, design and site preparation, the site will be suited for a new residential townhome project and associated improvements.

This opportunity to be of service is sincerely appreciated. Should you have any questions concerning the following report, please do not hesitate to contact us. Reference to our **Job No. 15-10805** will expedite a response to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

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1200 Block of S. Nevada Street
Oceanside, California

JOB NO. 15-10805

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

I. PROJECT DESCRIPTION

It is our understanding, based on information provided by the property owner, that the property is to be developed to receive a two- and three-story, multi-unit townhome project. Some units will include living areas over the individual unit garage and other units will include a living area at a level lower than the respective unit garage (street level). No large parking structure buildings will be included in the modified project. The buildings are to be constructed of standard-type building materials utilizing conventional foundations with concrete slab on-grade floors. No extensive shoring is anticipated to accommodate the lower level individual garages or basement living area of the proposed structures. A long interior driveway will provide access to the site from Ditmar Street. Only emergency, utility and service vehicles will enter the site from Nevada Street and Oceanside Boulevard through controlled access gates.

Conceptual construction plans for development of the site have been provided to us for the preparation of this report, however, when completed, the final grading and foundation plans should be made available for our review.

II. SCOPE OF WORK

The scope of work performed for this investigation included a review of previous geotechnical reports for the property prepared by Vinje & Middleton Engineering, Inc., dated November 26, 2001, March 7, 2003, and September 13, 2004, review of available published information pertaining to the site geology, a site geologic reconnaissance and subsurface exploration program, laboratory testing, geotechnical engineering analysis of the research, field and laboratory data, and the preparation of this second update geotechnical report. The data obtained and the analyses performed were for the purpose of providing geotechnical design and construction criteria and recommendations for the project earthwork, building foundations, and slab on-grade floors.

Vinje & Middleton, Inc., performed a preliminary geotechnical investigation that included eight exploratory trenches, two borings and laboratory testing for the subject site between 2001 and 2004. The following Vinje & Middleton reports were reviewed:

1. Preliminary Geotechnical Investigation -- Proposed Bluff Apartments, Nevada Street, City of Oceanside, dated November 26, 2001 (Job #01-345-P).

2. Supplemental Geotechnical Report -- Proposed Bluff Apartments, South Ditmar Street, City of Oceanside, dated March 7, 2003 (Job #01-345-P).

3. Geotechnical Update Report and Grading Plan Review -- Proposed Bella Terra Condominiums, Nevada Street, Oceanside, California, dated September 13, 2004 (Job #01-345-P).

III. SUMMARY OF GEOTECHNICAL & GEOLOGIC FINDINGS

Our subsurface geotechnical investigation revealed that the property is underlain at relatively shallow depth by dense bedrock of the Tertiary-age San Onofre Breccia (Tso), which is overlain by 2 to 4 feet of variable density fill soils, topsoils and colluvium. In their present condition, the surficial soils (fill soils, topsoils and colluvium) will not provide a stable base for the proposed residential structures and associated improvements. Excavation for the basement-level specific unit garages should result in the removal of the surficial soils at all basement locations.

We recommend that the surficial existing fill, loose top soils and colluvium be removed and recompacted as part of site preparation prior to the addition of any new fill or exterior hardscape or structural improvements. The bedrock materials have very good bearing strength characteristics, are of low expansion potential, and are suitable for support of proposed structural loads and new recompacted fill soil.

All foundations for the new residential structures, retaining walls and improvements should be founded into the underlying dense bedrock or properly compacted fill soils. Advanced planning for excavating of deeper drainage or utility lines is recommended so that after foundations or concrete improvements are placed there is no need to use heavy equipment for excavating adjacent to those concrete structures or improvements.

If dense bedrock is encountered at the finish pad elevation, the use of a hydraulic breaker (especially for excavation of utility trenches and foundations) may be required. In addition, if dense bedrock is encountered at finish pad elevation, it may be necessary to construct fill building pads at least 3 feet thick to provide at least 1½ feet of properly compacted fill under the bottom of the foundations to facilitate excavation of foundations and utility trenches. On-grade improvements such as garage slabs, concrete hardscape and/or pavements may be underlain by a minimum of 12 inches of properly compacted fill soils underlain by adequate bearing natural soil to provide a level subgrade. The designer and contractors should plan ahead if excavating deeper than 3 feet below finish grade for utility and drainage line trenching. Rock breaking for these trenches should be performed during rough grading rather than at the time of pipe installation during construction. The same is applicable if other below-grade structures are planned.

In our opinion, the site is suited for multi-family residential construction provided our recommendations are implemented during site development. No geologic hazards exist on or near the site that would prohibit the construction of new residential improvements. Conventional construction techniques and materials can be utilized. Conceptual construction plans have been provided to us for the preparation of this report, however, when finalized they should be made available for our review.

IV. SITE DESCRIPTION

The property consists of four parcels: Parcel 1 is known as Assessor's Parcel No. 152-121-06; Parcel 2 is known as Assessor's Parcel No. 152-123-05; Parcel 3 is known as Assessor's Parcel No. 152-320-11; and Parcel 4 is known as Assessor's Parcel No. 152-123-20, in the City of Oceanside, County of San Diego, State of California. For the location of the site, refer to the Vicinity Map, Figure No. I.

The irregular, roughly arcuate-shaped site, consisting of approximately 2.66 acres, is located on the north and south sides of the cul-de-sac at the southeast end of Nevada Street. The property consists of a gentle, southeast sloping vacant lot with a slope along the east side of the property that descends to the track bed for the

NCTD (North County Transit District) railroad right-of-way. The slope ranges in height from 35 to 52 feet at gradients ranging from 0.75:1.0 to 1.5:1.0 (horizontal to vertical). Near-vertical conditions are locally present along some of the base-of-slope areas.

Elevations across the lot range from a high of approximately 71 feet above mean sea level (MSL) along the central west side of the property to lows of approximately 35 feet MSL near the southeastern corner of the property and 20 feet MSL near the northeastern corner. Information concerning approximate site elevations was obtained from a topographic survey map prepared by bHA, Inc., dated September 2018.

The property is bordered on the north by Oceanside Boulevard, on the south by an existing, large mobile-home park, on the west by multi-family residential properties slightly higher in elevation, and on the east by southeasterly descending slopes that abut the NCTD (North County Transit District) railroad right-of-way at their base (for Site Plan, refer to Figure No. II).

V. __FIELD INVESTIGATION__

On July 15, 2016, we placed two exploratory borings at the site for evaluation of slope stability and providing shoring recommendations. The borings were excavated to depths of 24 and 25 feet. During our original investigation in July 2015, eight exploratory trenches were placed on the site (for exploratory boring and trench locations, refer to Figure No. II). The trenches were excavated to depths ranging from 4 to 6 feet in order to evaluate the existing bedrock and surficial soils and to define a soil profile across the site.

Observations were also made of the exposed bedrock materials where not concealed by vegetation coverage in the east perimeter slope. In addition, we reviewed the Vinje & Middleton logs of the eight exploratory trenches and two borings placed on the site in 2001. The trenches extended to depths ranging from 5 to 8 feet and the borings were placed to depths of 16 and 36 feet. Two 3-foot-deep test pits were also placed by Vinjie & Middleton in 2003.

The soils encountered in the exploratory borings and trenches were observed and logged by our field representative and samples were taken of the predominant soils. Excavation logs have been prepared on the basis of our observations and laboratory testing. The results have been summarized on Figure Nos. III and IV. The predominant soils have been classified in general conformance with the Unified Soil Classification System (refer to Appendix A).

VI. LABORATORY TESTS AND SOIL INFORMATION

Extensive laboratory testing was performed by Vinje & Middleton during their work in 2001 and 2003. In addition, supplemental laboratory testing was performed by our firm as part of the updated report preparation. The following laboratory soil tests were performed by Vinjie & Middleton in 2001 and 2003 (as reported in their reports dated November 26, 2001, and March 7, 2003) and GEI in 2016.

1. *Test Method for Penetration Test and Split-barrel Sampling of Soils (ASTM D1586)*
2. *Laboratory Compaction Characteristics (ASTM D1557)*
3. *Moisture Content (ASTM D2216)*
4. *Standard Test Method for Bulk Specific Gravity and Density of Compacted Bituminous Mixtures using Coated Samples (ASTM D1188)*
5. *Standard Test Method for Penetration Test and Split Barrel Sampling of Soils (ASTM D1586)*
6. *Determination of Percentage of Particles Smaller than #200 Sieve (ASTM D1140)*
7. *Standard Test Method for Expansion Index of Soils (ASTM 4829)*
8. *Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions (ASTM D3080)*

Laboratory Compaction Characteristics (ASTM D1557) 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 gives qualitative information regarding soil compaction conditions to be anticipated during any future grading operation.

Moisture content (ASTM D2216) and density measurements (ASTM D1586 and 1188) were performed using ASTM methods. These tests help to establish the in situ moisture and density of samples retrieved from exploratory excavations. The dry soil weight was compared to the laboratory maximum dry density of the same soil to determine relative compaction.

The *Determination of Percentage of Particles Smaller Than #200 (ASTM D1140)* aids in classification of tested soils based on their fine material content and provides qualitative information related to engineering characteristics such as expansion potential, permeability, and shear strength.

The expansion potential of soils is determined, when necessary, utilizing the *Standard Test Method for Expansion Index of Soils (ASTM D4829)*. 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

Direct shear tests (ASTM D3080) were performed on remolded soil samples in order to evaluate strength characteristics of the surficial and bedrock soils. Shear tests were performed with a constant strain rate direct shear machine. One sample was tested as unsaturated and one as saturated; both were then sheared under various normal loads.

Based on our review of their laboratory test data, our observations of the primary soil types on the project site, and our previous experience with laboratory testing of similar soils, our Geotechnical Engineer has assigned values for the angle of internal friction and cohesion to those soils that will provide significant lateral support or bearing functions on the project. These values were utilized in assigning the recommended bearing values as well as active and passive earth pressure design criteria for proposed foundations. Our laboratory tested bedrock soils yielded an angle of internal friction of 35 and 38 degrees and cohesion values of 702 and 2,151 psf for saturated and unsaturated samples, respectively.

VII. 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, metasedimetary 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 Ranges form 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 offshore from San Diego) (Berger and Schug, 1991).

In California, major earthquakes can generally be correlated with movement on active faults. As defined by the California Division of Mines and Geology (Hart, E.W., 1980), an *"active"* fault is one that has had ground surface displacement within

Holocene time (about the last 11,000 years). Additionally, faults along which major historic earthquakes have occurred (about the last 210 years in California) are also considered to be active (Association of Engineering Geologist, 1973). The California Division of Mines and Geology defines a "potentially active" fault as one that has had ground surface displacement during Quaternary time, that is, between 11,000 and 1.6 million years (Hart, E.W., 1980).

During recent history (prior to April 2010), the San Diego County area was relatively quiet seismically. No fault ruptures or major earthquakes have been experienced in historic time within the greater San Diego area. Since earthquakes have been recorded by instruments (since the 1930s), the San Diego area has experienced scattered seismic events with Richter magnitudes (M) generally less than M4.0. During June 1985, a series of small earthquakes occurred beneath San Diego Bay, three of which had recorded magnitudes of M4.0 to M4.2. In addition, the Oceanside earthquake of July 13, 1986, located approximately 26 miles offshore of the City of Oceanside, had a magnitude of M5.3 (Hauksson and Jones, 1988).

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 in magnitude 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 4, 2010, a large earthquake occurred in Baja California, Mexico. It was widely felt throughout the U.S. southwest including Phoenix, Arizona and San Diego, California. It significantly affected Mexicali, Mexico. This M7.2 event, the Sierra El Mayor earthquake, occurred in northern Baja California, approximately 40 miles south of the Mexico-USA border, at relatively 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 historic seismicity, and it has recently been seismically active, though this is the largest event to strike in this area since 1892. The April 4, 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 killed two people in Mexicali, caused widespread damage to structures, closure of businesses, government offices and schools, power outages, displacement of people from their homes and injured over 200 people in the nearby major metropolitan areas of Mexicali and adjacent Calexico in Southern California. Estimates of the cost of the damage range to over $100 million.

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 4, 2010, main event, recorded at stations in San Diego and reported by the California Strong Motion Instrumentation Program (CSMIP), ranged up to 0.058g. Aftershocks from this event continue to the date of this report along the trend northwest of the original event, including within San Diego County, closer to the San Diego metropolitan area. There have been hundreds of these earthquakes including events up to M5.7.

On July 7, 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. Seismologists expect continued aftershock activity.

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 February 25, 1980, M5.0 on October 31, 2001, and M5.2 on June 12, 2005. The biggest earthquake near this location was the M6.0 Buck Ridge earthquake on March 25, 1937.

VIII. SITE-SPECIFIC SOIL & GEOLOGIC DESCRIPTION

A. Stratigraphy

Our field work, reconnaissance and review of the "Geologic Map of North-Central Coastal Area of San Diego County, California" (F. Harold Weber, 1982) and the geologic maps by Kennedy and Tan, 2005 and 2008, *"Geologic Map of Oceanside, 30'x60' Quadrangle, California,"* indicate that the site is underlain by formational bedrock soils of the San Onofre Breccia (Tso). The encountered soil profile consists of 2 to 4 feet of fill/topsoil/colluvium overlying the bedrock. Figure No. V presents an excerpt of the 2008 geologic map of the general area of the site.

Fill Soils (Qaf): Minor amounts of surficial fill soils were encountered on the lot. The fill soils consist of tan-brown to gray-brown, silty, fine- to medium-grained sand. They are of variable density, dry to damp, and of very low expansion potential. The fill soils are not suitable in their current condition for support of loads from structures or additional fill. Refer to Figure No. III for details.

Topsoils/Colluvium: The topsoils and colluvium consist of approximately 1 to 3 feet of variable density, brown silty sand underlain by dark gray-brown and orange sandy clay with some roots and rock fragments. The topsoils are considered to be of low expansion potential and the clayey colluvial soils are considered to be of medium expansion potential. The topsoils/colluvium are not suitable in their current condition for support of loads from structures or additional fill. Refer to Figure No. III for details.

San Onofre Breccia (Tso): Formational bedrock soils of the San Onofre Breccia (Tso) were encountered at shallow depths ranging from 1½ to 4 feet at all exploratory excavation locations. The encountered formational materials consist of breccias with up to 80 percent rock (generally up to 12 inches in diameter, although some rock could be up to 3 feet or more in diameter) in a yellow-brown, red-brown, gray, olive-gray and orange, clayey and silty sand matrix. The encountered bedrock was dense and backhoe excavation met refusal on the bedrock at several excavation locations. Refer to Figure No. III for the excavation logs and Figure No. V for an excerpt of the 2008 geologic map of the general area of the site.

B. Structure

As depicted on the referenced geologic maps (Kennedy and Tan, 2008; Figure No. V), the underlying formational breccia is relatively flat lying and has no adverse geologic structure. No faults or landslides are mapped on the site. Aerial photograph review indicates that the site is not underlain by landslides or unstable natural slopes.

IX. GEOLOGIC HAZARDS

The following is a discussion of the geologic conditions and hazards common to the Oceanside area, 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, Figure No. V (Kennedy and Tan, 2008), indicates that no faults are mapped on the site. In our explicit professional opinion, neither an active fault nor a potentially active fault underlies the site.

Newport-Inglewood Fault: The offshore portion of the Newport-Inglewood Fault Zone is located approximately 4.5 miles northwest of the project site. A significant earthquake (M6.4) occurred along this fault on March 10, 1933. Since then, no additional 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 (SCEC, 2004).

Rose Canyon Fault: The northern portion of the Rose Canyon Fault is located approximately 5.4 miles west of the subject site and is mapped trending north-south from Oceanside to downtown San Diego, from where it appears to head southward into San Diego Bay, through Coronado and offshore. The Rose Canyon Fault Zone is considered to be a complex zone of onshore and offshore, en echelon strike slip, oblique reverse, and oblique normal faults. The Rose Canyon Fault is considered to be capable of causing an earthquake of magnitude M7.2 per the California Geologic

Survey (2002) and considered microseismically active, although no significant recent earthquake is 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. These findings confirm Holocene displacement on the Rose Canyon Fault, which was designated an "active" fault in November 1991 (Hart and Bryant, 2007, Fault-Rupture Hazard Zones in California, California Geological Survey Special Publication 42).

Coronado Bank Fault: The Coronado Bank Fault is located approximately 21.3 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 an M7.6 earthquake and is of great interest due to its close proximity to the greater San Diego metropolitan area.

Elsinore Fault: The Elsinore Fault is located approximately 23 to 60 miles east and northeast of the site. The fault extends approximately 200 km (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 11,000 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, 1985). More recently, the California Geologic Survey (2002) considers the Elsinore Fault capable of producing an earthquake of M6.8 to M7.1.

Palos Verde Fault: The Palos Verde Fault is located approximately 33.9 miles north/northwest of the site and is considered to be a northern extension of the Coronado Bank Fault Zone. The Palos Verde fault is believed to have a slip rate of approximately 0.1 to 3.0 mm/yr with an unknown recurrence interval. This fault is believed capable of producing an earthquake of M6.0 to M7.0, possibly greater (SCEC, 2004).

San Jacinto Fault: The San Jacinto Fault is located 46.2 to 67.8 miles to the 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 (Earth Consultants International [ECI], 2009).

The San Jacinto Fault Zone has a high level of historical seismic activity, with at least 10 damaging (M6.0 to M7.0) earthquakes having occurred on this fault zone between 1890 and 1986. Earthquakes on the San Jacinto 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 (ECI, 2009).

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 (WGCEP, 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, (ECI, 2009). 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 on 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.

Whittier Fault: The Whittier Fault is located approximately 48.8 miles north of the site. The Whittier Fault is believed to have a slip rate of approximately 2.5 to 3.0 mm/yr with an unknown recurrence interval. This fault is believed capable of producing an earthquake of M6.0 to M7.2 (SCEC, 2004).

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 1-mile-long surface-rupture length could be expected (Greensfelder, 1974). Our investigation indicates that the subject site is not directly on a known 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 on 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 notable to significant damage. It is our opinion that the most serious damage to the site would be caused by a large earthquake originating on a strand of the Rose Canyon Fault Zone. Although the chance of such an event is remote, it could occur within the useful life of the structure. Ground shaking from earthquakes on active southern California faults and active faults in northwestern Mexico is the greatest geologic hazard. The Modified Mercalli Index is presented as Appendix B.

Landslides: Based upon our geologic reconnaissance, review of the geologic maps (Kennedy and Tan, 2008; and Weber, 1982), and review of the USDA stereo-pair aerial photographs AXN-9M-192 & 193, dated April 14, 1953, that depict the area of the site before development there are no known or suspected ancient landslides located on the site.

Slope Stability: The existing slopes along the east side of the property are comprised of relatively high strength, very dense, silty sand formational breccia materials and are regarded as stable. Temporary slopes are anticipated to have good stability if they are constructed in accordance with our recommendations.

We performed updated slope stability analyses on selected cross sections based on information obtained from our exploratory excavations, the laboratory test results from retrieved soil samples collected during the drilling, our field review of site conditions, review of aerial photos, review of pertinent documents and geologic maps of the area, and our experience with similar formational units in this area of the City of Oceanside. The slope stability analyses were performed along various cross sections oriented perpendicular to the sloping lot from north to south. The locations of the cross sections are presented on the updated Plot Plan, Figure No. IIb.

Gross slope stability was calculated using the *SLIDE 6* program. Based on our analysis, the factor of safety against the gross failure of proposed project slopes is in excess of 1.5. Refer to Slope Stability Analysis, Appendix D.

Shallow slope failure analysis on representative existing slopes yielded a factor of safety higher than 1.5. It is our opinion the site slopes should remain stable if proper drainage and irrigation practices are maintained. Refer to Slope Stability Analysis, Appendix D.

Liquefaction: The liquefaction of saturated sands during earthquakes can result in major damage to buildings. Liquefaction is the process in which soils are transformed into a dense fluid that will flow as a liquid when unconfined. It occurs principally in loose, saturated sands and silts when they are sufficiently shaken by an earthquake and cause large deformation on fine-grained and soft clayey soils.

On this site, the risk of liquefaction of foundation materials due to seismic shaking is considered to be remote due to the dense nature of the natural-ground material, the anticipated high density of the proposed recompacted fill, and the lack of a shallow static groundwater surface under the site. No soil liquefaction or soil strength loss is anticipated to occur due to a seismic event.

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, meteoric 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.

Based on review of the *San Diego County Multi-Jurisdiction Hazard Mitigation Plan San Diego County, California* (2010) there were eight *"Proclaimed States of Emergency for Weather/Storms"* in San Diego County between 1950 and 2005. In January and February 1983, the strongest-ever El Nino-driven coastal storms caused over $116 million in beach and coastal damage. Thirty-three homes were destroyed and 3,900 homes and businesses were damaged. Other coastal storms that caused notable damage were during the El Nino winters of 1977-78, 1997-98, 2003-04, and during the storm of January 20, 1981.

Wave heights and run-up elevations from tsunami along the San Diego Coast have historically fallen within the normal range of the tides (Joy 1968). The largest tsunami effect recorded in San Diego since 1950 was May 22, 1960, which had a maximum wave height 2.1 feet (NOAA, 1993). In this event, 80 meters of dock were destroyed and a barge sunk in Quivera Basin. Other tsunamis felt in San Diego County occurred on November 5, 1952, with a wave height of 2.3 feet caused by an earthquake in Kamchatka; March 9, 1957, with a wave height of 1.5 feet; May 22, 1960, at 2.1 feet; March 27, 1964, with a wave height of 3.7 feet and September 29, 2009, with a wave height of 0.5 feet. It should be noted that damage does not necessarily occur in direct relationship to wave height, illustrated by the fact that the damage caused by the 2.1-foot wave height in 1960 was worse than damage caused by several other tsunamis with higher wave heights.

The site is located approximately ½-mile from the Pacific Ocean strand line at an elevation of 40 to 70 feet above MSL. It is unlikely that a tsunami would affect the lot. In addition, the site is not mapped within a possible inundation zone on the California Geological Survey's 2009 "Tsunami Inundation Map for Emergency Planning, Oceanside Quadrangle, San Diego County."

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is located on the north side and near the western end of the Loma Alta Creek bed. Due to the property's elevation, there is minimal risk of flooding from a seiche affecting the site.

Geologic Hazards Summary: It is our opinion, based upon a review of the available geologic maps, our research, and our site investigation, that the site is underlain by relatively stable formational materials (and shallow fill soils, topsoils and colluvium to be recompacted), and is suited for the proposed residential structures and associated improvements provided the recommendations herein are implemented. No significant geologic hazards are known to exist on the site that would prevent the proposed construction. In our professional opinion, no "active" or "potentially active" faults underlie the project site.

The most significant geologic hazard at the site is anticipated ground shaking from earthquakes on active Southern California and Baja California faults. The United States Geologic Survey has issued statements indicating that seismic activity in Southern California may continue at elevated levels with increased risk to major metropolitan areas near the Elsinore and San Jacinto faults. These faults are too far from the subject property to present a seismic risk. To date, the nearest known "active" faults to the subject site are the northwest-trending Rose Canyon Fault, Newport-Inglewood Fault and the Coronado Bank Fault.

X. <u>GROUNDWATER</u>

A true or significant groundwater condition was ***not*** encountered at the explored excavation locations. Minor seepage was encountered at the location of exploratory trench T-2. We do not expect significant perched water/seepage or groundwater problems to develop in the future ***if proper drainage is implemented and maintained on the property***.

It should be kept in mind that grading operations will change surface drainage patterns and/or reduce permeabilities due to the densification of compacted soils. Such changes of surface and subsurface hydrologic conditions, plus irrigation of landscaping or significant increases in rainfall, may result in the appearance of surface or near-surface water at locations where none existed previously. The damage from such water is expected to be localized and cosmetic in nature, if good positive drainage is implemented, as recommended in this report, during and at the completion of construction.

Subsurface drainage with a properly designed and constructed subdrain system will be required, along with continuous back drainage, behind proposed below-grade building walls and retaining walls.

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 fill or formational soils are fine-grained or very dense, 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.

XI. RECOMMENDATIONS

The following conclusions and recommendations are based upon the practical field investigation conducted by our firm and resulting laboratory tests, in conjunction with our knowledge and experience with similar soils in the Oceanside area of the County of San Diego.

Due to the encountered dense bedrock at relatively shallow depths, excavation deeper than 3 feet may require the use of bigger than usual excavation equipment or a rock hydraulic breaker. Similarly, trenches for water, sewer, gas, and electricity lines may also require the use of a rock hydraulic breaker. We recommend that an experienced earthwork contractor familiar with conditions in this area of San Diego County review this report and evaluate the on-site conditions to determine the appropriate excavation techniques. Preferably, the utility and drainage line trench undercutting should be done during rough grading and not after the footing excavations are completed. Difficulty during excavation should be considered when planning foundation excavation procedures.

It is our opinion that the site is suitable for the planned residential project provided the recommendations herein are incorporated during design and construction. Further, it is our opinion that site development will not destabilize neighboring properties or induce the settlement of adjacent structures if designed and constructed in accordance with our recommendations. Shoring will be required along the northern property boundary adjacent to the existing structures and improvements.

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 and test the site earthwork and installation of foundations. Accordingly, we recommend that the following paragraph be included on the grading and foundation plans for the project:

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

A. Seismic Design Criteria

1. *Seismic Data Bases:* An estimation of the peak ground acceleration and the repeatable high ground acceleration (RHGA) likely to occur at the project site is based on the known significant local and regional faults within 100 miles of the site. In addition, we have reviewed a listing of the known historic seismic events that have occurred within 100 miles of the site at an M5.0 or greater since the year 1800, and the probability of exceeding the experienced ground accelerations in the future based upon the historical record. Estimations of site intensity are provided as Modified Mercalli Index values (see Appendix B).

2. *Seismic Design Criteria:* The proposed structures should be designed in accordance with the 2016 CBC, which incorporates by reference the ASCE 7-

10 for seismic design. We recommend the following parameters be utilized. We have determined the mapped spectral acceleration values for the site based on latitude 33.1849 degrees and longitude -117.3659 degrees, utilizing a program titled "*U.S. Seismic Design Maps and Tools*" provided by the USGS, which provides a solution for ASCE 7-10 (2016 CBC) utilizing digitized files for the Spectral Acceleration maps.

3. *Structure and Foundation Design*: The design of new structures and foundations should be based on Seismic Design Category D.

4. *Spectral Acceleration and Design Values*: The structural seismic design, when applicable, should be based on the following values that are based on the site location, soil characteristics, and seismic maps by USGS, as required by the 2016 CBC. The design Spectral Acceleration (SA) vs. Period (T) is shown on Appendix C. The Site C 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}
1.173	0.451	1.031	1.549	1.209	0.699	0.806	0.466

B. Preparation of Soils for Site Development

5. *Clearing and Stripping:* Vegetation on the site should be removed prior to the preparation of the building pad and areas of new improvements. 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 properly compacted fill.

Vegetation clearing should extend to at least 5 feet beyond the perimeter borders of structures and/or improvements.

6. *Treatment of Existing Loose Surficial Fill/Topsoils/Colluvium:* Excavation for the basement-level parts of pertinent units will result in the removal of all loose surficial soils at the basement locations. It is recommended that any loose surficial soils that remain after the necessary site excavations be removed and recompacted. The anticipated depth of loose soil removal is approximately 2 to 4 feet. A 3-foot-deep undercut should be provided if a transition (cut/fill) line crosses the building pad. If most of the area under a building pad is dense natural soils (i.e., approximately 70 percent of the total area is cut), the remaining 30 percent corresponding to fill may be compacted to at least 95 percent of maximum dry density of the soil. Rough grading undercutting of an entire building pad may be preferred at some locations considering that excavating for utility and foundation trenches may be difficult.

   Recompaction work should consist of (a) removing remaining loose surficial soils down to native dense formational materials; (b) scarifying, moisture conditioning, and compacting the exposed subgrade soils; and (c) replacing the excavated material as compacted structural fill.

   The areal extent and depth required to remove loose surficial soils should be confirmed by our representatives during the excavation work based on their examination of the soils being exposed. The lateral extent of the excavation and recompaction should be at least 5 feet beyond the edge of the perimeter foundations and any areas to receive exterior improvements or a lateral distance equal to the depth of soil removed at any specific location, whichever is larger. Any unsuitable materials (such as oversize rubble or rocks, and/or organic matter) should be selectively removed as directed by our representative and disposed of off-site.

Any rigid improvements founded on existing loose or soft surface soils or in areas built across cut/fill transition lines not properly treated 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.

7. *Subgrade Preparation:* After the site has been cleared, stripped, and the required excavations made, the exposed subgrade soils in areas to receive fill and/or building improvements should be scarified to a depth of 6 inches, moisture conditioned, and compacted to the requirements for structural fill. ***Excavation into dense bedrock soils will not require scarification or recompaction.*** The near-surface moisture content of fine-grained (clayey) medium or highly expansive soils, if encountered, should be compacted with a moisture content over the optimum and maintained by periodic sprinkling until within 48 hours prior to concrete placement.

8. *Expansive Soil Conditions:* We do not anticipate that significant quantities of medium or highly expansive clay soils will be encountered during grading. Should such soils be encountered and used as fill, however, they should be moisture conditioned to at least 5 percent above optimum moisture content and compacted to 88 to 92 percent in building pad areas. Soils of medium or greater expansion potential should not be used as retaining wall backfill soils.

9. *Material for Fill:* Existing on-site soils with an organic content of less than 3 percent by volume are, in general, suitable for use as fill, other than retaining wall backfill. Any required imported fill material (such as for retaining wall backfill) should be low expansive (Expansion Index of 50 or less per ASTM D4829-11). In addition, both imported and existing on-site materials used as fill should not contain rocks or lumps more than 6 inches in greatest dimension when compacted with heavy grading equipment. Retaining wall and trench backfill material should not contain material larger than 3 inches in greatest dimension. All materials used as fill should be approved by our representative ***prior*** to importing to the site. Trench backfill should include compacted bedding sand to at least 1 foot above the pipes prior to using on-site soils. Drainage and sewer trenches should be undercut at least 6 inches below the invert elevation.

10. *Fill Compaction:* All structural fill should be compacted to a minimum degree of compaction of 90 percent based upon ASTM D1557-12. In some areas, fill compaction should be at least 95 percent, i.e., for building pads where most of the pad will consist of dense formational soils and a smaller area of the pad will consist of fill, and no undercutting is to be performed. 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 moisture content that will permit proper compaction by either: (1) aerating and drying the fill if it is too wet, or (2) moistening the fill with water 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.

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

11. *Grading Requirements*: Applicable City of Oceanside grading requirements should be followed during construction of the planned improvements.

12. *Trench and Retaining Wall Backfill:* Utility trenches and retaining walls should preferably be backfilled with on-site, low-expansive or imported, low-expansive compacted fill; gravel is also a suitable backfill material but should be used only if space constraints will not allow the use of compaction equipment. Gravel can also be used as backfill around perforated subdrains protected with geofabric. All 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.

Backfill soils placed behind retaining walls and/or crawl space 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, with an Expansion Index equal to or lower than 50. All areas backfilled with gravel should be capped with a minimum 12-inch-thick layer of properly compacted on-site soils overlying Mirafi 140N filter fabric to reduce the potential for fines loss into the gravel.

C. *Design Parameters for Proposed Foundations*

In order to support the proposed structures on conventional continuous concrete foundations the following recommendations should be followed. Footings should extend into formational bedrock soils or properly compacted fill soils to a depth of 18 inches. If most of the foundations of a unit are in bedrock, the remaining footings of that unit should be supported on bedrock if excavations do not extend deeper than 5 feet, or placed in 95 percent compacted fill.

13. *Footings:* Footings for new residential structures should bear on undisturbed formational bedrock materials or properly compacted fill soils. The footings for the proposed structures should be founded at least 18 inches below the lowest adjacent finished grade and have a minimum width of 15 inches. The footings should contain at least 4 No. 5 bars (top and bottom reinforcement) to provide structural continuity and to permit spanning of local irregularities.

    If the proposed footings are located closer than 7 feet inside the top or face of slopes, they should be deepened to 1½ feet below a line beginning at a point 7 feet horizontally inside the slopes and projected outward and downward, parallel to the face of the slope and into firm soils (see Figure No. VI). Footings located adjacent to utility trenches should have their bearing surfaces situated below an imaginary 1.0:1.0 plane projected upward from the bottom edge of the adjacent utility trench. Otherwise, the trenches should be excavated farther from the footing locations. If utility or drainage pipes need to cross under footings, they should be placed in sleeve pipes embedded in concrete underneath the footing, and reinforced per details provided by the project structural engineer.

14. *Bearing Values*: At the recommended depths, footings on native, dense formational soil or properly compacted fill soil may be designed for allowable bearing pressure of qa=1,320D + 960W, in psf, where D is the depth of embedment and W is the width of the footing, in feet. The total allowable bearing should not exceed 6,000 psf for combined dead and live loads and may be increased one-third for foundation design that includes wind or seismic loads.

15. *Footing Reinforcement*: All continuous footings should contain top and bottom reinforcement to provide structural continuity and to permit spanning of local irregularities. We recommend that a minimum of four No. 5 reinforcing bars be provided in the footings; two near the top and two near the bottom. A minimum clearance of 3 inches should be maintained between steel reinforcement and the bottom or sides of the footing. Isolated square footings should contain, as a minimum, a grid of three No. 4 steel bars on 12-inch centers, both ways. In order to provide an opinion as to whether the footings are founded on soils of sufficient load bearing capacity, it is essential that our representative inspect the footing excavations prior to the placement of reinforcing steel or concrete.

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.

16. *Lateral Loads:* Lateral load resistance for structures supported on shallow foundations where encountered formation is shallow in depth may be developed in friction between the foundation bottoms and the supporting subgrade. An allowable friction coefficient of 0.46 is considered applicable. An additional allowable passive resistance equal to an equivalent fluid weight of 425 pcf acting against the foundations may be used in design provided the footings are poured neat against the adjacent undisturbed formational materials and/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. Foundation near slopes should be designed to provide a 7-foot setback from the slope face measured from top of footing face closest to the slope.

17. *Settlement:* Settlements under building loads are expected to be within tolerable limits for the proposed residences. 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 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 bearing on dense natural soils.

18. *Minimum Floor Slab Reinforcement:* Based on our experience, we have found that, for various reasons, concrete 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.

Interior floor slabs should be a minimum of 4 inches actual thickness and be reinforced with No. 3 bars on 18-inch centers, both ways, placed at midheight in the slab. If needed, up to 3 inches of crushed rock gravel may be placed between the subgrade and moisture barrier for leveling purposes. Slab subgrade soil moisture 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. Shrinkage control joints should be placed no farther than 20 feet apart and at re-entrant corners. The joints should penetrate at least 1 inch into the slab. Garage slabs and drives should be designed as discussed in Recommendation No. 24.

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. *Slab Moisture Protection and Vapor Barrier Membrane:* 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 architect and waterproofing consultants or product manufacturer.

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 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 E1745-97 (2009) Standard Specification for Plastic Water Vapor Retarders Used in Contact Concrete Slabs; ASTM E154-88 (2005) Standard Test Methods for Water Vapor Retarders Used in Contact with Earth; ASTM E96-95 Standard Test Methods for Water Vapor Transmission of Materials; ASTM E1643-98 (2009) Standard Practice for Installation of Water Vapor Retarders Used in Contact Under Concrete Slabs; and ACI 302.2R-06 Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials.

19.1 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 sub-paragraphs 7.1.1-7.1.5) should be less than 0.01 U.S. perms (grains/square foot/hour/inch of mercury (Hg) and comply with the ASTM E1745 Class A requirements. Installation of vapor barriers should be in accordance with ASTM E1643. The basis of design is 15-mil StegoWrap 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. Our suggested acceptable moisture retardant membranes are based on a report entitled "*Report of Water Vapor Permeation Testing of Construction Vapor Barrier Materials*" by Dr. Kay Cooksey, Ph.D., Clemson University, Dept. of Packaging Science, 2009-10.

The membrane may be placed directly on properly compacted smooth subgrade soils and directly underneath the slab. Proper slab curing is required to help prevent slab curling.

19.2 Common to all acceptable products, vapor retarder/barrier joints must be lapped and sealed with mastic or the manufacturer's recommended tape or sealing products. 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.

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

20. *Concrete Isolation Joints:* We recommend the project Civil/Structural Engineer incorporate isolation joints and control joints (sawcuts) to at least one-fourth the thickness of the slab in any floor designs. The joints and cuts, if properly placed, should reduce the potential for and help control floor slab cracking. We recommend that concrete shrinkage joints be spaced no farther than approximately 20 feet apart, and also at re-entrant corners. However, due to a number of reasons (such as base preparation, construction techniques, curing procedures, and normal shrinkage of concrete), some cracking of slabs can be expected.

21. *Exterior Nonstructural Concrete Slabs:* As a minimum for protection of on-site improvements, we recommend that all nonstructural concrete slabs (such as patios, sidewalks, etc.), be founded on properly compacted and tested fill or dense native formation and be underlain if needed by 2 inches (and no more than 3 inches) of compacted clean leveling sand, with No. 3 bars at 18-inch centers, both ways, at the center of the slab. Exterior concrete slabs should be at least 4 inches thick. Exterior slabs should contain adequate isolation and control joints as noted in the following paragraphs.

The performance of on-site improvements can be greatly affected by soil base preparation and the quality of construction. It is therefore important that all improvements are properly designed and constructed for the existing soil conditions. The improvements should not be built on loose soils or fills placed without our observation and testing. The subgrade of exterior improvements should be verified as properly prepared within 48 hours prior to concrete placement. A minimum thickness of 2 feet of properly recompacted soils should underlie exterior slabs on-grade for secondary improvements if not bearing on dense natural soils.

22. *Exterior Slab Control Joints*: For exterior slabs with the minimum shrinkage reinforcement, control joints should be placed at spaces no farther than 12 feet apart or the width of the slab, whichever is less, and also at re-entrant corners. Control joints in exterior slabs should be sealed with elastomeric joint sealant. The sealant should be inspected every 6 months and be properly maintained. Concrete slab joints should be dowelled or continuous steel reinforcement should be provided to help reduce any potential differential movement.

23. *Pavement*: Pavement design sections will depend largely on the subgrade soil conditions exposed after grading and should be based on R-value test results. These tests should be performed after completion of the rough grading operation. A preliminary pavement cross section, to be verified after soil R-value tests, may consist of 3 inches of asphalt concrete on 8 inches of Class II base material.

    All contemporary pavement section design methods assume proper compaction of at least the upper 12 inches of foundation soil (natural ground or compacted fill to 95 percent compaction), and/or all base material to at least 95 percent of Maximum Dry Density. We therefore recommend that the upper 12 inches of foundation soils and/or all base materials beneath driveway and parking area pavements be scarified, moisture conditioned, and compacted to a minimum of 95 percent of Maximum Dry Density. This compaction recommendation also applies to the upper soils in backfilled trenches or behind retaining walls that support pavement sections. The upper 2 feet of parking lot areas should also be properly compacted to at least 90 percent of maximum dry density. The subgrade and base layers should be compacted to at least 95 percent relative compaction.

24. *Concrete Pavement:* New concrete driveway and exterior parking slabs should be at least 5½ inches thick and rest on properly prepared and compacted subgrade soils. Interior garage slabs may be 4 inches thick. Subgrade soil for driveways, exterior parking areas and garage slabs should be dense or, if fill, be compacted to at least 95 percent of Maximum Dry Density. The driveway, parking, and garage slabs should be provided with reinforcement consisting of No. 4 bars spaced no farther than 15 inches apart in two perpendicular directions. The concrete should be at least 3,500 psi compressive strength, with control joints no farther than 15 feet apart and also at re-entrant corners. Pavement joints should be properly sealed with permanent joint sealant, as required in sections 201.3.6 through 201.3.8 of the Standard Specifications for Public Work Construction, 2015 Edition.

    Control joints should be placed within 12 hours after concrete placement or as soon as the concrete allows sawcutting without aggregate raveling. The sawcuts should penetrate at least one-quarter the thickness of the slab.

25. *Asphalt Concrete Pavement:* For asphalt concrete (A.C.) placement, we recommend a preliminary section of 3 inches of A.C. on 8 inches of Class II base gravel on properly compacted subgrade. The definitive pavement cross sections should be established after rough grading is completed and should be based on R-value soil tests performed on subgrade soils. Areas in front of trash containers should be provided with a section of concrete pavement as recommended above.

E. Slopes

To our knowledge, construction will require placement of new cut or fill slopes over 12 feet in height, primarily for the construction of the basement walls. Should portions of the site be modified with new slopes, the following recommendations should be applied.

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

27. *Project Slopes:* The existing slopes along the east property line are regarded as grossly stable with a factor of safety against failure of 1.5 or greater. We recommend that any new permanent cut or fill slopes up to 12 feet in height be constructed at an inclination no steeper than 2.0:1.0 (horizontal to vertical). All fills placed on sloping areas should be properly keyed and benched into dense formational soils.

28. *Shoring*: Based on the information obtained during our investigation, we offer the following shoring recommendations for the areas of the site where vertical or inclined temporary cuts cannot be safely used due to boundary lines or soil conditions at the site, and any other areas were required for safety and

construction purposes. We understand that soldier pile and lagging or tiebacks may be used as shoring on this project.

29. *Shotcrete Tieback Walls:* If used, shotcrete walls should be constructed from the top down by cutting the distance specified by the structural engineer before the row of tiebacks are drilled and installed. Shotcrete placement should be as specified by the structural engineer.

30. *Stockpiled Soils*: No stockpiled soils will be allowed within a distance of 10 feet of the edge of the existing top of slope or top of the excavation edge.

31. *Temporary Slopes*: Except for the existing steepened upper slope area, we do not anticipate that temporary slopes of significant height over 15 feet will be required during construction or grading. Should temporary slopes be required in properly compacted fill, they may not be cut at a slope steeper than 0.5:1.0 (h:v). In dense, cemented formational soils, vertical cuts up to 8 feet can be made in the lower part of the excavation, for a total height not exceeding 15 feet at an inclination of 0.5:1.0 (h:v) in the upper part. Depending on periodic observations by our project geologist, higher vertical cuts may be allowed. Any plans for temporary slopes in excess of the 15-foot maximum must be presented to our office prior to grading to allow time for review and specific recommendations, if warranted. Proper drainage away from the excavation should be provided at all times. As previously indicated, no surcharge shall be placed within 10 feet from the top of temporary slopes.

In consideration of the high-angle, near-vertical slopes along the east side of the property that are at least 75 years old (and other vertical slopes in the area), it is our opinion in situ strength cohesions are in excess of 250 to 600 psf. We utilized 400 psf in formulation of our recommendations for temporary vertical cut slopes up to 15 feet in height cut in dense, cemented formational soils.

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

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.

32. *Tieback Recommendations*: We understand that for the design of tieback shotcrete walls, the Structural Engineer needs only the soil basic recommendations as follows. The coefficient (ka) to calculate the soil pressure is 0.27; the soil unit weight is 130 pcf, and the soil pressure distribution may be rectangular with a uniform soil pressure of 22.8 psf on the total wall height. Alternatively, the soil pressure distribution may be trapezoidal where the maximum pressure "p" may consist of a trapezoid with the soil pressure increasing from zero at the top to a maximum soil pressure "p" located at 0.2H and then decreases to zero at the bottom of the shotcrete face from a distance equal to 0.2H from the bottom. The maximum soil pressure "p" is calculated using the equation p=25H.

The recommended design tieback bond strength is 6,000 plf. A higher bond strength may be used if performance tests are completed in advance to the production testing and final design. The critical failure surface is estimated to occur at 28 degrees, with the vertical plane passing behind the tieback wall. The recommended unbonded length of tiebacks should be 15 feet for strand tendons and 10 feet for bar tendons. In addition, the unbonded length should extend a minimum distance of 5 feet behind the critical potential failure plane. The structural designer should specify the pull tests following the Post-tensioning Institute guidelines.

33. *Slope Face Drainage*: MiraDrain 6200 drainage board should be placed on the slope face as chimney drains between the tiebacks, and discharge through weep holes at the base of the shotcrete wall. The weep holes should consist of 3-inch-diameter PVC pipe rising at 15 degrees from horizontal into the wall and spaced no farther than 5 feet apart. The subdrain outlet pipes should connect to a 6-inch-diameter perforated collector pipe in an envelope of crushed rock gravel wrapped with 140N Mirafi fabric that "daylights" at location specified by the structural designer.

34. *Reinforcement*: Reinforcement of steel tendons and bars for the shotcrete face should be specified by the structural engineer. The same is applicable for the shotcrete face.

35. *Soldier Pile and Lagging Shoring:* For cantilever soldier pile and lagging shoring walls, the active soil pressure for wall design is 35 pcf. The shaft frictional resistance of the soldier piles is an average of 600 psf. End bearing at the bottom of the pile excavation is not recommended due to difficulty in cleaning the bottom of the excavation. Recommended passive resistance of soldier piles is 1,000 x diameter of pile x depth of embedment below cut surface in front of pile, in pounds. Any surcharge loads should be considered

by multiplying the load by a conversion coefficient of 0.27 for cantilever shoring.

36. *Slope Top Structure Performance:* Rigid improvements such as top-of-slope walls, columns, decorative planters, concrete flatwork, swimming pools, 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.

Typical and to-be-expected separations can range from minimal to up to 1 inch or greater in width. In order to reduce the effect of slope-top lateral soil deformation, we recommend that the top-of-slope improvements be designed with flexible connections and joints in rigid structures so that the separations do not result in visually apparent cracking damage and/or can be cosmetically dressed as part of the ongoing property maintenance. These flexible connections may include "slip joints" in wrought iron fencing, evenly spaced vertical joints in block walls or fences, control joints with flexible caulking in exterior flatwork improvements, etc.

In addition, use of planters to provide separation between top-of-slope hardscape such as patio slabs and decking from top-of-slope walls can aid greatly in reducing cosmetic cracking and separations in exterior improvements. Actual materials and techniques would need to be determined by the project architect or the landscape architect for individual units. Steel dowels placed in flatwork may prevent noticeable vertical differentials, but if provided with a slip-end they may still allow some lateral displacement.

F. *Retaining Wall Design Criteria*

37. *Static Soil Design Parameters:* Retaining walls must be designed to resist lateral earth pressures and any additional lateral pressures caused by surcharge loads on the adjoining retained surface. We recommend that restrained retaining walls with level backfill be designed for an equivalent fluid pressure of 56 pcf for low expansive import or on-site soils. Wherever restrained walls will be subjected to surcharge loads, they should also be designed for an additional uniform lateral pressure equal to 0.43 times the anticipated surcharge pressure. Unrestrained (cantilever) walls may be designed using an equivalent fluid weight of 35 pcf when using low expansive backfill soils and a level backfill surface, and 52 pcf for 2.0:1.0 (H:V) sloping backfill. Restrained walls with sloping backfill should use 77 pcf.

    Backfill placed behind the 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 and designed to support the higher soil pressure produced by the equipment.

38. *Seismic Earth Pressures*: If seismic loading is to be considered for retaining walls more than 6 feet in height, they should be designed for seismic earth pressures in addition to the normal static pressures. The soil seismic increment is an equivalent fluid weight of 14 pcf. A Kh value of 0.16 may be used in a computer program such as "*Retaining Wall Pro*" or a similar program for wall design. The soil pressures described above may also be used for the

design of shoring structures. The seismic soil pressure increment may be applied at the resultant of triangular pressure distribution (inverted traditional way or at the newly accepted location similar to the static soil pressure). According to recent research, the seismic soil increment may be waived on restrained designed retaining walls.

39. *Design Parameters – Unrestrained:* The active earth pressure to be utilized in the design of any cantilever retaining walls (utilizing on-site or imported very low- to low-expansive soils [EI less than 50] as backfill) should be based on an Equivalent Fluid Weight of 35 pounds per cubic foot (for level backfill only). In the event that an unrestrained retaining wall is surcharged by sloping backfill, the design active earth pressure should be based on the appropriate Equivalent Fluid Weight presented in the following table.

	Height of Slope/Height of Wall*			
Slope Ratio	**0.25**	**0.50**	**0.75**	**1.00(+)**
2.0:1.0 (existing slope)	42	48	50	52

*To determine design active earth pressures for ratios intermediate to those presented, interpolate between the stated values.

Backfill soils should consist of low-expansive soils with EI less than 50, and should be placed from the heel of the foundation to the ground surface within the wedge formed by a plane at 30° from vertical, and passing by the heel of the foundation and the back face of the retaining wall.

40. *Surcharge Loads:* Any surcharge loads placed on the active wedge behind a cantilever unrestrained wall should be included in the design by multiplying

the vertical load by a factor of 0.27. This factor converts the vertical load to a horizontal load.

41. *Wall Drainage:* Proper subdrains and free-draining backwall material or board drains (such as J-drain or Miradrain) should be installed behind all retaining walls (in addition to proper waterproofing) on the subject project (see Figure No. VII for Retaining Wall Backdrain and Waterproofing Schematic). ***Geotechnical Exploration, Inc.*** will assume no liability for damage to structures or improvements that is attributable to poor drainage.

Architectural plans should clearly indicate that subdrains for any lower-level walls be placed at an elevation at least 1 foot below the top of the outer face of the footing, not on top of the footing. At least 0.5-percent gradient should be provided to the subdrain.

The subdrain should be placed in an envelope of crushed rock gravel up to 1 inch in maximum diameter, and be wrapped with Mirafi 140N filter fabric or equivalent. The subdrain should consist of AmeriDrain, QuickDrain (rectangular section boards), or equivalent products. A sump pump may be required if project elevations and discharge points do not allow for outlet via gravity flow. The collected water should be taken to an approved drainage facility. Open head joint subdrain discharge ***is not*** considered acceptable for retaining walls. All subdrain systems should be provided with access risers for periodic cleanout.

42. *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.

G. Site Drainage Considerations

43. *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, ponding on finished building pad areas or causing erosion on soil surfaces.

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

    Failure to observe this recommendation could result in undermining and possible differential settlement of the structures or other improvements or cause other moisture-related problems. Currently, the CBC requires a minimum 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.

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

H. General Recommendations

46. *Project Start Up Notification:* In order to reduce any work delays during site development, this firm should be contacted at least 48 hours prior to any need for observation of footing excavations or field density testing of compacted fill soils. If possible, placement of formwork and steel reinforcement in footing excavations should not occur prior to observing the excavations; in the event that our observations reveal the need for deepening or redesigning foundation structures at any locations, any formwork or steel reinforcement in the affected footing excavation areas would have to be removed prior to correction of the observed problem (i.e., deepening the footing excavation, recompacting soil in the bottom of the excavation, etc.).

47. *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 storm water conveyance systems due to construction vehicles or any other construction activity. The contractor is responsible for cleaning any such debris that may be in the street at the end of each work day or after a storm event that causes breach in the installed construction BMPs.

All stockpiles of uncompacted soil and/or building materials that are intended to be left unprotected for a period greater than 7 days are to be provided with erosion and sediment controls. Such soil must be protected each day when the probability of rain is 40% or greater.

A concrete washout should be provided on all projects that propose the construction of any concrete improvements that are to be poured in place. All erosion/sediment control devices should be maintained in working order at all times. All slopes that are created or disturbed by construction activity must be protected against erosion and sediment transport at all times. The storage of all construction materials and equipment must be protected against any potential release of pollutants into the environment.

XII. GRADING NOTES

Geotechnical Exploration, Inc. recommends that we be retained to verify the actual soil conditions revealed during site construction/grading work and footing excavation to be as anticipated in this "*Report of Geotechnical Investigation Update*" 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 retaining wall and trench backfill should be properly compacted. ***Geotechnical Exploration, Inc.*** will assume no liability for damage occurring due to improperly or uncompacted backfill placed without our observations and testing.

XIII. LIMITATIONS

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

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.

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

It is the responsibility of the owner and/or developer to ensure that the recommendations summarized in this report are carried out in the field operations and that our recommendations for design of this project are incorporated in the structural plans. We should be retained to review the project plans once they are available, to verify that our recommendations are adequately incorporated in the plans. Additional or 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. 15-10805** will expedite a reply to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

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Job No. 15-10805
September 2018

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VICINITY MAP

Thomas Bros. Guide San Diego County pg. 1106-C2

Breeze Townhomes
1200 Block of Nevada Street
Oceanside, CA.

Figure No. I
Job No. 15-10805

LEGEND

T-8 — Approximate Location of Exploratory Trench

B-1 — Approximate Location of Exploratory Boring

B — B' — Cross Section Location

4 — 4' — Slope Stability Cross Section Location

N

Scale: 1" = 100'
(approximate)

REFERENCE: *This Plot Plan was prepared from an existing SITE PLAN by OPEN ARCHITECTS usa, Inc. dated 06/20/16 and from on-site field reconnaissance performed by GEI.*

NOTE: This Plot Plan is not to be used for legal purposes. Locationss 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.

PLOT PLAN

Breeze Townhomes
1200 Block of Nevada Street
Oceanside, CA.
Figure No. II
Job No. 15-10805

Geotechnical Exploration, Inc.

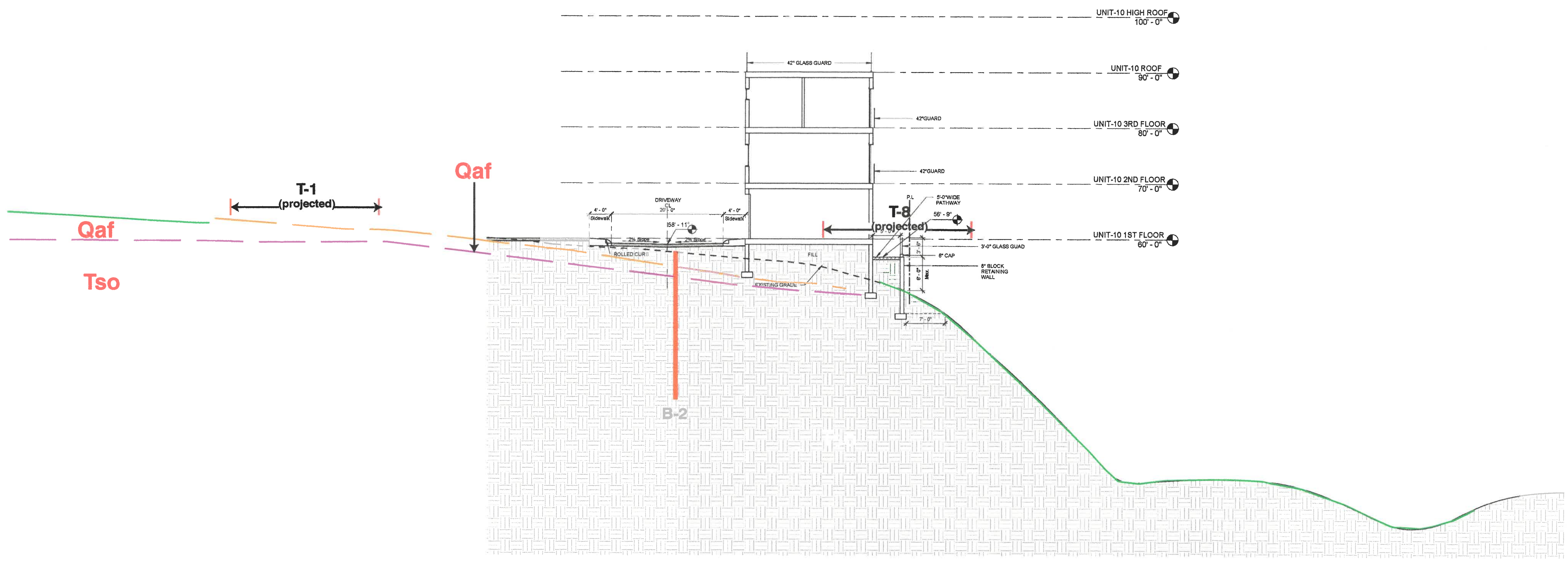
(revised August 2016)
(updated September 2018)

15-10805-p3.ai

GEOLOGIC CROSS SECTION A-A'

A

A'



Scale: 1" = 20'
(approximate)

LEGEND

Qaf	Artificial Fill
Qcol	Colluvium
Tso	San Onofre Breccia

REFERENCE: This Cross Section was prepared from an existing SECTION PLANS by THE LIGHTFOOT PLANNING GROUP dated 09/13/2018.

GEOLOGIC CROSS SECTION A-A'

Breeze Townhomes
1200 Block of Nevada Street
Oceanside, CA.
Figure No. IIb
Job No. 15-10805

Geotechnical Exploration, Inc.

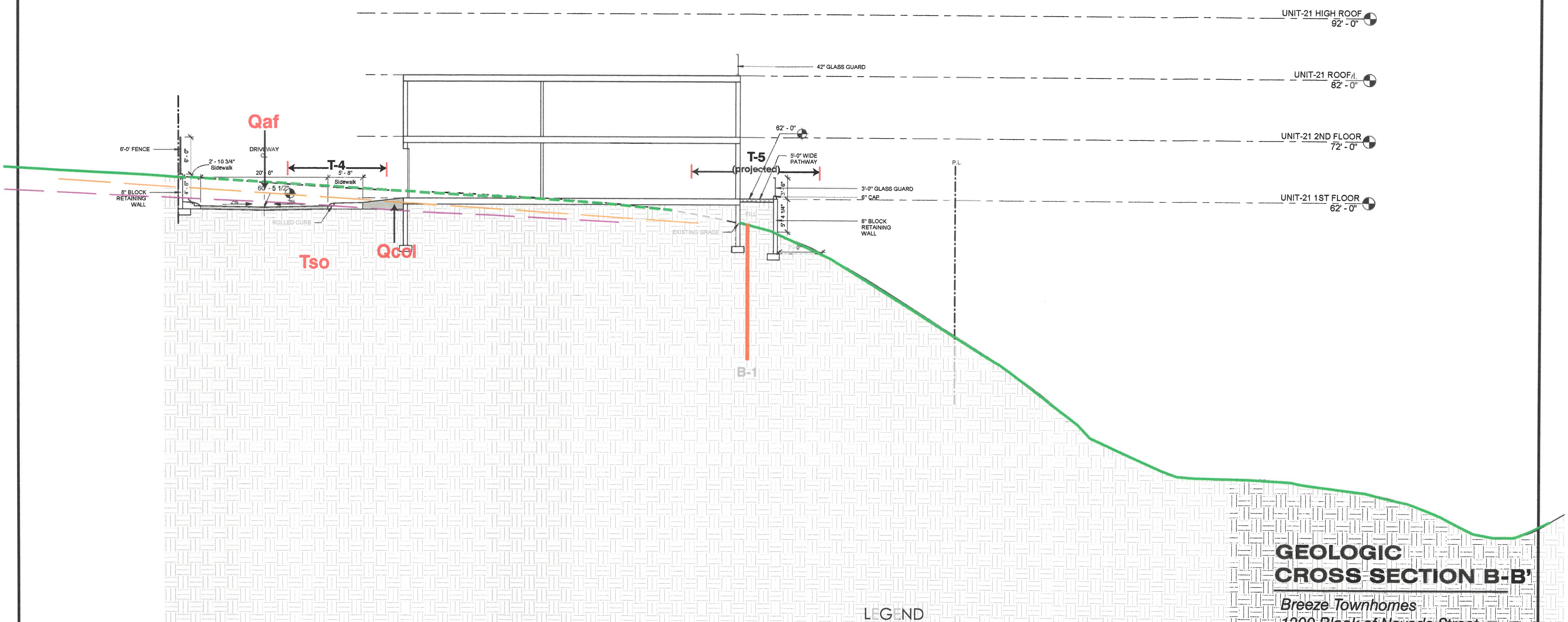
August 2016
(updated September 2018)

15-10505-AA-9-2018.ai

GEOLOGIC CROSS SECTION B-B'

B

B'



Scale: 1" = 20'
(approximate)

LEGEND

Qaf	Artificial Fill
Qcol	Colluvium
Tso	San Onofre Breccia

REFERENCE: This Cross Section was prepared from an existing SECTION PLANS by THE LIGHTFOOT PLANNING GROUP dated 09/13/2018.

GEOLOGIC CROSS SECTION B-B'

Breeze Townhomes
1200 Block of Nevada Street
Oceanside, CA.
Figure No. IIc
Job No. 15-10805

Geotechnical Exploration, Inc.
August 2016
(updated September 2018)

15-10505-B-B-9-2018.ai

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Track-mounted Backhoe	**2' X 10' X 4' Trench**	**7-1-15**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 63' Mean Sea Level	**Not Encountered**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION — DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–1			**SILTY SAND**, fine-grained, with rock fragments and concrete debris. Loose. Dry. Tan-brown. **FILL (Qaf)**	SM								
1–2.5			**SILTY SAND**, fine-grained; poorly cemented. Loose to medium dense. Dry. Brown. **TOPSOIL**	SM								
2.5–4			**SANDY CLAY**, with roots and some rock fragments. Very stiff. Dry. Dark gray-brown and orange. **TOPSOIL/ COLLUVIUM (Qcol)** -- very hard excavating for trackhoe.	CL								
4			Bottom @ 4'									

PERCHED WATER TABLE
BULK BAG SAMPLE
1 IN-PLACE SAMPLE
MODIFIED CALIFORNIA SAMPLE
S NUCLEAR FIELD DENSITY TEST
STANDARD PENETRATION TEST

JOB NAME		
Breeze Townhome Project		
SITE LOCATION		
1200 Block of Nevada Street, Oceanside, CA		
JOB NUMBER: **15-10805**	REVIEWED BY: **LDR/JAC**	LOG No. **T-1**
FIGURE NUMBER: **IIIa**	Geotechnical Exploration, Inc.	

EXPLORATION LOG 10805 OCEANSIDE APTS.GPJ GEO_EXPL.GDT 7/6/15

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Track-mounted Backhoe	**2' X 10' X 6' Trench**	**7-1-15**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 58' Mean Sea Level	**2 feet**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION — DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–2			**SILTY SAND**, fine-grained, with roots and some rock fragments. Loose to medium dense. Damp to moist. Gray-brown. **FILL/ TOPSOIL (Qaf)**	SM								
2			-- seepage on contact @ 2'.									
2–3.5			**SANDY CLAY**, with some roots and rock fragments. Soft to firm. Moist. Dark gray-brown and orange. **TOPSOIL/ COLLUVIUM (Qcol)**	CL								
3.5–6			**BEDROCK**, breccia with approximately 70% rock fragments (up to 8" in diameter) in **CLAYEY SAND** matrix. Dense. Damp. Yellow-brown. **SAN ONOFRE BRECCIA (Tso)**	SC								
6			Bottom @ 6'									

- PERCHED WATER TABLE
- BULK BAG SAMPLE
- 1 IN-PLACE SAMPLE
- MODIFIED CALIFORNIA SAMPLE
- S NUCLEAR FIELD DENSITY TEST
- STANDARD PENETRATION TEST

JOB NAME		
Breeze Townhome Project		
SITE LOCATION		
1200 Block of Nevada Street, Oceanside, CA		
JOB NUMBER **15-10805**	REVIEWED BY **LDR/JAC**	LOG No. **T-2**
FIGURE NUMBER **IIIb**	Geotechnical Exploration, Inc.	

EXPLORATION LOG 10805 OCEANSIDE APTS.GPJ GEO_EXPL.GDT 7/6/15

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Track-mounted Backhoe	**2' X 10' X 5.5' Trench**	**7-1-15**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 65' Mean Sea Level	**Not Encountered**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–1			**SILTY SAND**, fine-grained, with roots and wire debris. Loose. Dry. Gray-brown. **FILL (Qaf)**	SM								
1–2.5			**SILTY SAND**, fine-grained with some roots and rock fragments; poorly cemented. Loose to medium dense. Dry. Brown. **TOPSOIL**	SM								
2.5–4			**SANDY CLAY**, with some caliche. Very stiff. Damp. Dark gray-brown and orange. **TOPSOIL/ COLLUVIUM (Qcol)**	CL								
4–5.5			**BEDROCK**, breccia with approximately 75% rock fragments (to __" in diameter) in **SILTY SAND** matrix. Very dense. Damp. Red-brown and gray. **SAN ONOFRE BRECCIA (Tso)**	GM								
6			Bottom @ 5.5'									

▼	PERCHED WATER TABLE
☒	BULK BAG SAMPLE
1	IN-PLACE SAMPLE
■	MODIFIED CALIFORNIA SAMPLE
S	NUCLEAR FIELD DENSITY TEST
▨	STANDARD PENETRATION TEST

JOB NAME		
Breeze Townhome Project		
SITE LOCATION		
1200 Block of Nevada Street, Oceanside, CA		
JOB NUMBER **15-10805**	REVIEWED BY **LDR/JAC**	LOG No. **T-3**
FIGURE NUMBER **IIIc**	Geotechnical Exploration, Inc.	

EXPLORATION LOG 10805 OCEANSIDE APTS.GPJ GEO_EXPL.GDT 7/6/15

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Track-mounted Backhoe	**2' X 10' X 5' Trench**	**7-1-15**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 65' Mean Sea Level	**Not Encountered**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION — DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–1.5			**SILTY SAND**, fine-grained, with roots and rock fragments; poorly cemented. Loose to medium dense. Dry. Gray-brown. **TOPSOIL**	SM								
1.5–3			**SANDY CLAY**, with roots and rock fragments. Very stiff. Damp. Dark gray-brown and orange. **TOPSOIL/ COLLUVIUM (Qcol)**	CL								
3–5			**BEDROCK**, breccia with approximately 75%-80% rock fragments (to 12"- 14" in diameter) in **CLAYEY SAND** matrix. Very dense. Damp. Olive-gray and orange. **SAN ONOFRE BRECCIA (Tso)**	GC								
5			Bottom @ 5'									

Depth scale: 1, 2, 3, 4, 5, 6

EXPLORATION LOG 10805 OCEANSIDE APTS.GPJ GEO_EXPL.GDT 7/6/15

- PERCHED WATER TABLE
- BULK BAG SAMPLE
- 1 IN-PLACE SAMPLE
- MODIFIED CALIFORNIA SAMPLE
- S NUCLEAR FIELD DENSITY TEST
- STANDARD PENETRATION TEST

JOB NAME		
Breeze Townhome Project		
SITE LOCATION		
1200 Block of Nevada Street, Oceanside, CA		
JOB NUMBER **15-10805**	REVIEWED BY **LDR/JAC**	LOG No. **T-4**
FIGURE NUMBER **IIId**	Geotechnical Exploration, Inc.	

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Track-mounted Backhoe	**2' X 10' X 4' Trench**	**7-1-15**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 60' Mean Sea Level	**Not Encountered**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–1			**SILTY SAND**, fine-grained, with roots, rock fragments and some debris. Loose. Dry. Gray-brown. **FILL/ TOPSOIL (Qaf)**	SM								
1–2			**SANDY CLAY**, with some roots and rock fragments. Stiff. Damp. Dark gray-brown. **TOPSOIL**	CL								
2–4			**BEDROCK**, breccia with approximately 75% rock fragments (up to 8" in diameter) in **CLAYEY SAND** matrix. Very dense. Damp. Olive gray-green. **SAN ONOFRE BRECCIA (Tso)**	SC								
4–5			Backhoe refusal on very dense bedrock. Bottom @ 4'									

- PERCHED WATER TABLE
- BULK BAG SAMPLE
- 1 IN-PLACE SAMPLE
- MODIFIED CALIFORNIA SAMPLE
- S NUCLEAR FIELD DENSITY TEST
- STANDARD PENETRATION TEST

JOB NAME		
Breeze Townhome Project		
SITE LOCATION		
1200 Block of Nevada Street, Oceanside, CA		
JOB NUMBER **15-10805**	REVIEWED BY **LDR/JAC**	LOG No. **T-5**
FIGURE NUMBER **IIIe**	Geotechnical Exploration, Inc.	

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Track-mounted Backhoe	**2' X 10' X 4' Trench**	**7-1-15**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 65' Mean Sea Level	**Not Encountered**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–1.5			**SILTY SAND**, fine-grained, with roots and rock fragments. Loose to medium dense. Dry. Gray-brown. **TOPSOIL**	SM								
1.5–3			**SANDY CLAY**, with some roots and rock fragments. Very stiff. Damp. Dark gray-brown and orange. **TOPSOIL/ COLLUVIUM (Qcol)** -- large rock in middle of trench @ 2'.	CL								
3–4			**BEDROCK**, breccia with approximately 75%-80% rock fragments (up to 12" in diameter) in **CLAYEY SAND** matrix. Very dense. Damp. Olive gray-green and orange. **SAN ONOFRE BRECCIA (Tso)**	GM								
4			Backhoe refusal on very dense bedrock. Bottom @ 4'									

Depth scale: 1, 2, 3, 4, 5

- ▼ PERCHED WATER TABLE
- ☒ BULK BAG SAMPLE
- 1 IN-PLACE SAMPLE
- ■ MODIFIED CALIFORNIA SAMPLE
- S NUCLEAR FIELD DENSITY TEST
- ▨ STANDARD PENETRATION TEST

JOB NAME **Breeze Townhome Project**		
SITE LOCATION **1200 Block of Nevada Street, Oceanside, CA**		
JOB NUMBER **15-10805**	REVIEWED BY **LDR/JAC**	LOG No. **T-6**
FIGURE NUMBER **IIIf**	Geotechnical Exploration, Inc.	

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Track-mounted Backhoe	**2' X 10' X 4' Trench**	**7-1-15**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 62' Mean Sea Level	**Not Encountered**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION — DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–1.5			**SILTY SAND**, fine-grained, with roots and rock fragments. Loose to medium dense. Dry. Gray-brown. **TOPSOIL** -- large rock in middle of trench @ 1.5'.	SM								
1.5–3			**SANDY CLAY**, with some roots and rock fragments. Very stiff. Damp. Dark gray-brown and orange. **TOPSOIL/ COLLUVIUM (Qcol)**	CL								
3–4			**BEDROCK**, breccia with approximately 75%-80% rock fragments (to 12" in diameter) in **CLAYEY SAND** matrix. Very dense. Damp. Olive-gray and orange. **SAN ONOFRE BRECCIA (Tso)**	GC								
4			Backhoe refusal on very dense bedrock. Bottom @ 4'									
5												

PERCHED WATER TABLE
BULK BAG SAMPLE
1 IN-PLACE SAMPLE
MODIFIED CALIFORNIA SAMPLE
S NUCLEAR FIELD DENSITY TEST
STANDARD PENETRATION TEST

JOB NAME		
Breeze Townhome Project		
SITE LOCATION		
1200 Block of Nevada Street, Oceanside, CA		
JOB NUMBER: **15-10805**	REVIEWED BY: **LDR/JAC**	LOG No.
FIGURE NUMBER: **IIIg**	Geotechnical Exploration, Inc.	**T-7**

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Track-mounted Backhoe	2' X 10' X 3' Trench	7-1-15
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 55' Mean Sea Level	Not Encountered	JKH

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–1.5			**SILTY SAND**, fine-grained, with roots and rock fragments. Loose. Dry. Gray-brown. **FILL/ TOPSOIL (Qaf)**	SM								
1.5–3			**BEDROCK**, breccia with approximately 75%-80% rock fragments (up to __" in diameter) in **SILTY SAND** matrix. Very dense. Damp. Red-brown and gray. **SAN ONOFRE BRECCIA (Tso)**	GM								
3			Backhoe refusal on very dense bedrock. Bottom @ 3'									

Depth scale: 1, 2, 3, 4

PERCHED WATER TABLE
BULK BAG SAMPLE
1 IN-PLACE SAMPLE
MODIFIED CALIFORNIA SAMPLE
S NUCLEAR FIELD DENSITY TEST
STANDARD PENETRATION TEST

JOB NAME: **Breeze Townhome Project**		
SITE LOCATION: **1200 Block of Nevada Street, Oceanside, CA**		
JOB NUMBER: **15-10805**	REVIEWED BY: **LDR/JAC**	LOG No. **T-8**
FIGURE NUMBER: **IIIh**	Geotechnical Exploration, Inc.	

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Truck-mounted Auger Drill Rig	**8-inch diameter Boring**	**7-15-16**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 60' Mean Sea Level	**Not Encountered**	**JKH**

FIELD DESCRIPTION AND CLASSIFICATION

DEPTH (feet)	SYMBOL	SAMPLE	DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–3			**CLAYEY SAND**, with roots and rock fragments. Loose to medium dense. Dry to damp. Gray-brown.	SC								
			FILL/ TOPSOIL (Qaf)									
3			**BEDROCK**, gravelly sand with some clay. Very dense. Damp. Yellow-brown.	GM								
4–5		Modified California			4.2	131.0					94/ 8"	3"
5–7		Bulk bag	**SAN ONOFRE BRECCIA (Tso)** -- 35% passing #200 sieve.				7.5	142.5				
10.5		Modified California									50/ 2"	3"
15		Modified California / Standard Penetration	-- very dense; no sample recovery.								50/ 2" 50/ 2"	3" 2"
21		Modified California, 1									50/ 2.5"	3"
23			-- becomes very rocky.									
24			Bottom @ 24'									

PERCHED WATER TABLE	JOB NAME **Breeze Townhome Project**		
BULK BAG SAMPLE	SITE LOCATION **1200 Block of Nevada Street, Oceanside, CA**		
IN-PLACE SAMPLE	JOB NUMBER **15-10805**	REVIEWED BY **LDR/JAC**	LOG No. **B-1**
MODIFIED CALIFORNIA SAMPLE	FIGURE NUMBER **IIIi**	Geotechnical Exploration, Inc.	
NUCLEAR FIELD DENSITY TEST			
STANDARD PENETRATION TEST			

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Truck-mounted Auger Drill Rig	**8-inch diameter Boring**	**7-15-16**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 63' Mean Sea Level	**Not Encountered**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION — DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–2			**SILTY SAND**, fine- to edium-grained, with some roots. Loose to medium dense. Dry. Gray-brown.	SM									
3			**FILL/ TOPSOIL (Qaf)**	CL									
4			**SANDY CLAY**, with some rock fragments. Stiff. Damp. Dark brown.										
5–6		1	**TOPSOIL/ COLLUVIUM (Qcol)**	GM								110/ 2"	3"
7–8			**BEDROCK**, gravelly sand with some clay. Very dense. Damp. Yellow-brown.										
9			**SAN ONOFRE BRECCIA (Tso)**										
10		1	-- very dense (no ring sample recovery).								19	50/ 3"	3"
12			-- becomes very rocky from 12'- 14'.										
15												75/ 5.5"	2"
20												50/ 4"	3"
25			-- no sample recovery.									50/ 2"	3"
26			Bottom @ 25'										

PERCHED WATER TABLE

BULK BAG SAMPLE

1 IN-PLACE SAMPLE

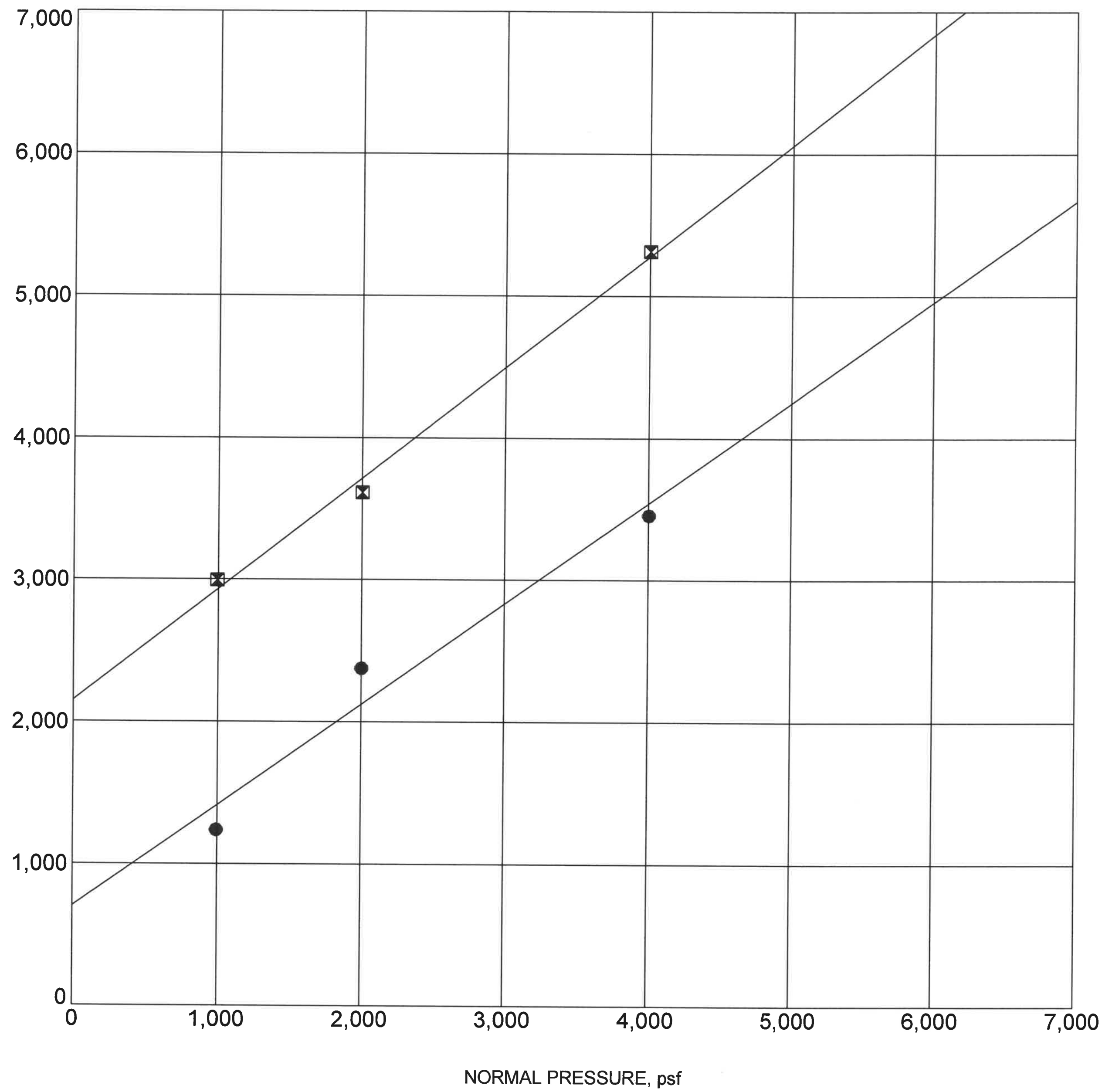
MODIFIED CALIFORNIA SAMPLE

S NUCLEAR FIELD DENSITY TEST

STANDARD PENETRATION TEST

JOB NAME	**Breeze Townhome Project**
SITE LOCATION	**1200 Block of Nevada Street, Oceanside, CA**
JOB NUMBER	**15-10805**
FIGURE NUMBER	**IIIj**
REVIEWED BY	**LDR/JAC**
LOG No.	**B-2**

Geotechnical Exploration, Inc.

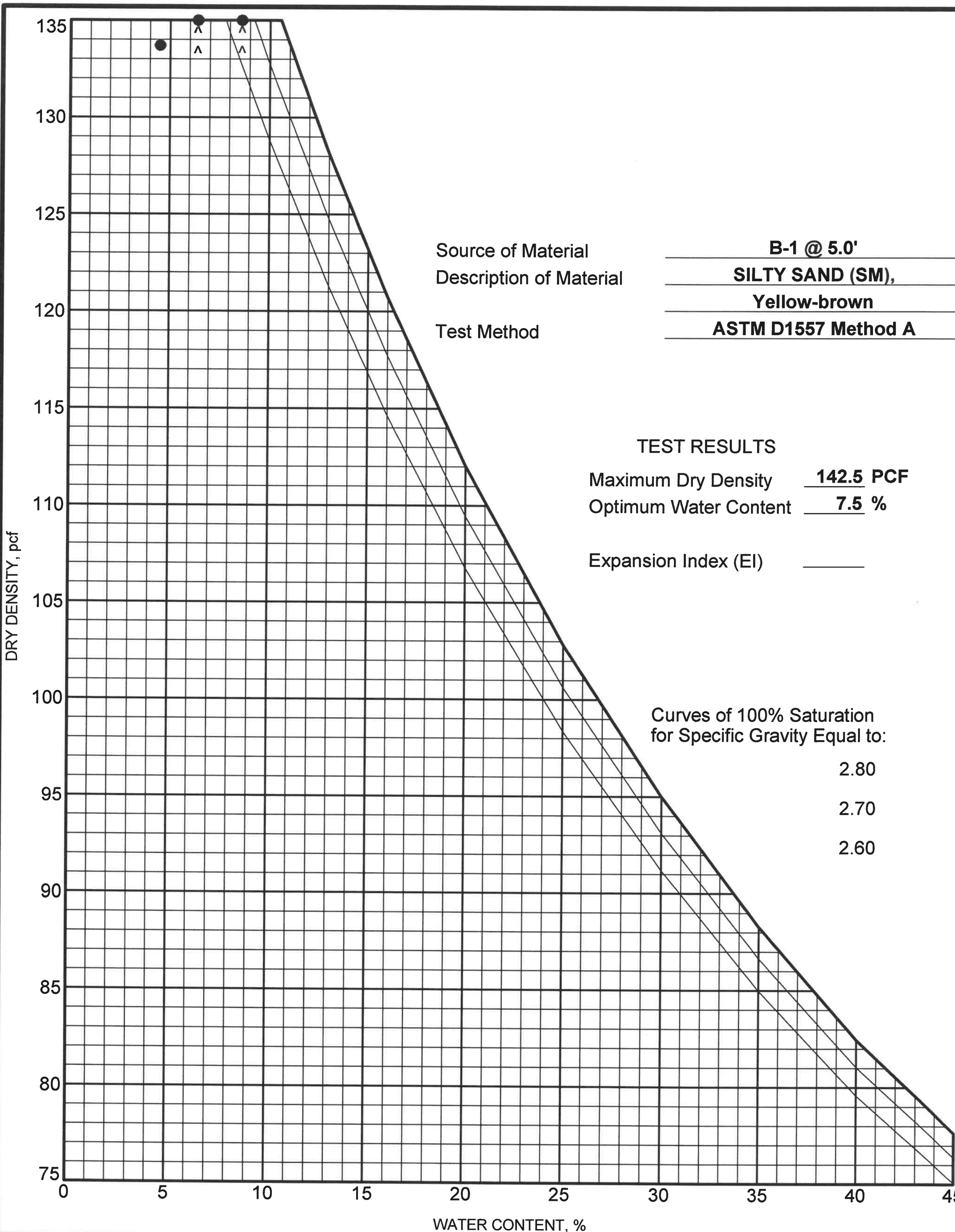


	Specimen Identification	Classification	γ_d	MC%	c	ϕ
●	**B-1 @ 5.0'**	**SILTY SAND (SM), Yellow-brown**			**702**	**35**
⊠	**B-2 @ 10.0'**	**SILTY SAND (SM), Yellow-brown**			**2151**	**38**

Geotechnical Exploration, Inc.

DIRECT SHEAR TEST

Figure Number: IVc
Job Name: Breeze Townhome Project
Site Location: 1200 Block of Nevada Street, Oceanside, CA
Job Number: 15-10805



Source of Material: B-1 @ 5.0'

Description of Material: SILTY SAND (SM), Yellow-brown

Test Method: ASTM D1557 Method A

TEST RESULTS

Maximum Dry Density 142.5 PCF

Optimum Water Content 7.5 %

Expansion Index (EI) ______

COMPACTION + EI DARK GRID 10805 OCEANSIDE APTS.GPJ GEI FEB06.GDT 8/8/16

Geotechnical Exploration, Inc.

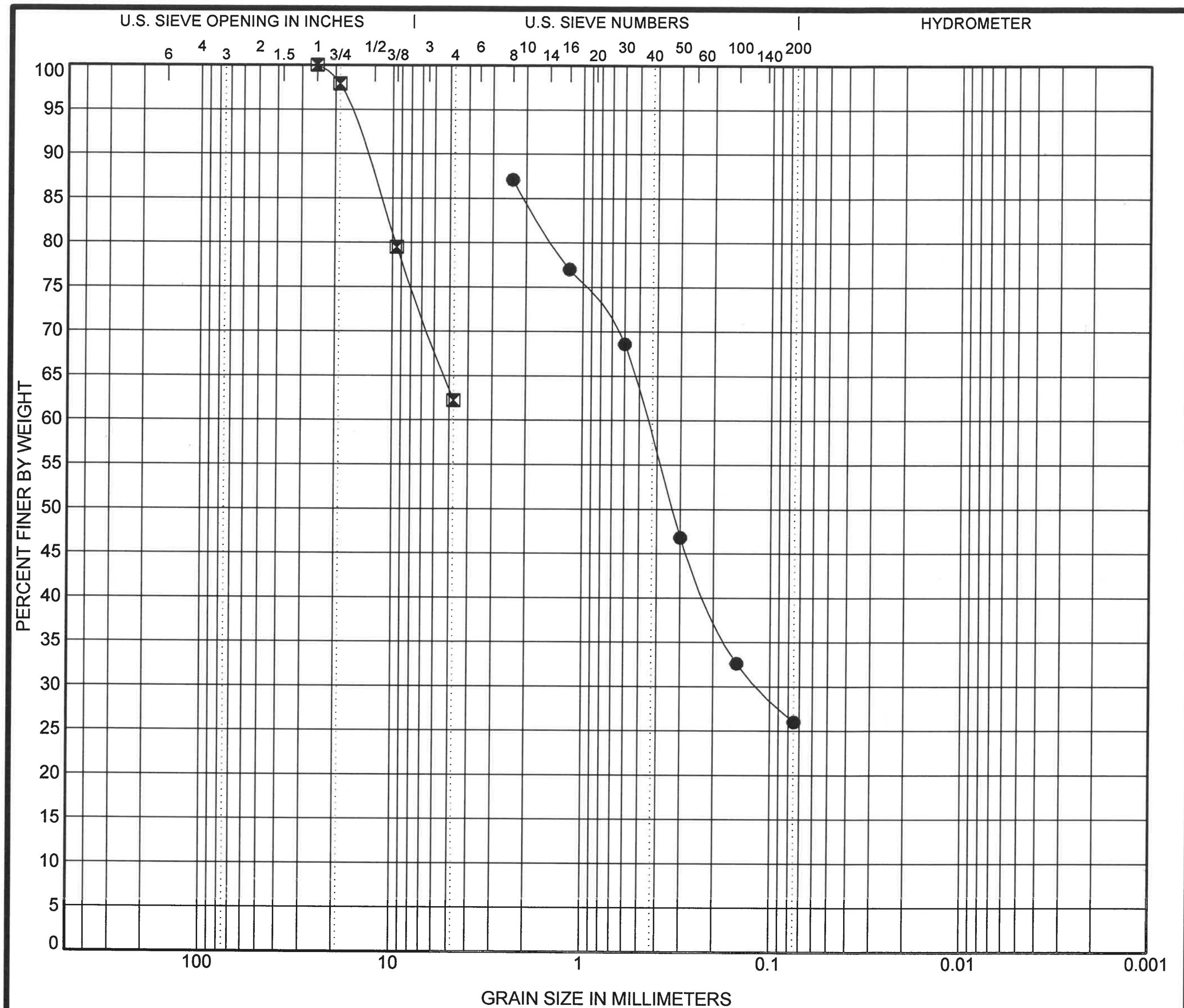
MOISTURE-DENSITY RELATIONSHIP

Figure Number: IVa

Job Name: Breeze Townhome Project

Site Location: 1200 Block of Nevada Street, Oceanside, CA

Job Number: 15-10805



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification	Classification	LL	PL	PI	Cc	Cu
●	**B-2 @ 10.0'**	**SILTY SAND (SM), Yellow-brown**					
⊠	**B-2 @ 10.0'**	**GRAVEL (SM), Yellow-brown**					

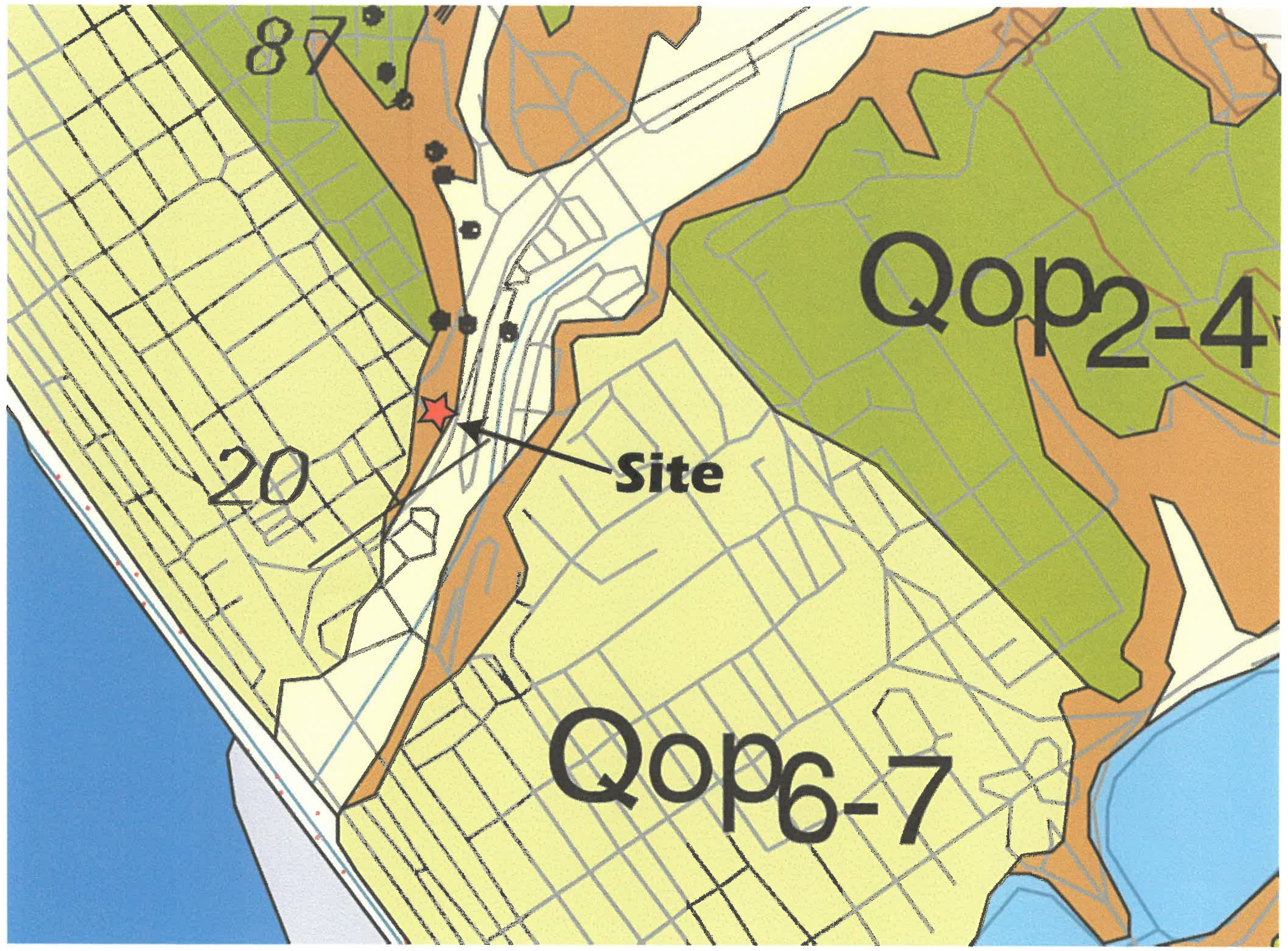
	Specimen Identification	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-2 @ 10.0'**	**2.36**	**0.456**	**0.114**		**0.0**	**61.1**	**26.0**	
⊠	**B-2 @ 10.1'**	**25**				**37.8**			

US GRAIN SIZE 10805 OCEANSIDE APTS.GPJ GEO EXPL.GDT 8/8/16

Geotechnical Exploration, Inc.

GRAIN SIZE DISTRIBUTION

Figure Number: IVb
Job Name: Breeze Townhome Project
Site Location: 1200 Block of Nevada Street. Oceanside, CA
Job Number: 15-10805



Breeze Townhomes
1200 Block Nevada Street
Oceanside, CA.

EXCERPT FROM GEOLOGIC MAP OF THE OCEANSIDE 30' X 60' QUADRANGLE, CALIFORNIA

Compiled by
Michael P. Kennedy and Siang S. Tan
2005
Digital Preparation by
Kelly R. Bovard[1], Rachel M. Alvarez[1] and Michael J. Watson[1]
1. U.S. Geological Survey, Department of Earth Sciences, University of California, Riverside

ONSHORE MAP SYMBOLS

Contact—Contact between geologic units; dotted where concealed.

70 U D **Fault**—Solid where accurately located; dashed where approximately located; dotted where concealed. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.

Anticline—Solid where accurately located; dotted where concealed.

Syncline—Solid where accurately located; dotted where concealed.

Kgd—granite pegmatite dike

Closed depression—Closed depression in Elsinore fault zone.

Qls? **Landslide**—Arrows indicate principal direction of movement. Queried where existence is questionable.

Strike and dip of beds

70 **Inclined**

70 **Overturned**

Vertical

Horizontal

Strike and dip of igneous foliation

45 **Inclined**

Vertical

Strike and dip of igneous joints

60 **Inclined**

Vertical

Strike and dip of metamorphic foliation

55 **Inclined**

Strike and dip of sedimentary joints

Vertical

DESCRIPTION OF MAP UNITS

Tso Tsoss **San Onofre Breccia (middle Miocene)**—Chiefly marine sedimentary breccia, conglomerate, and lithic sandstone. Named by Ellis and Lee (1919) for exposures in San Onofre Hills. Detailed descriptions of petrology and paleontology are given by Woodford (1925), who described unit as "San Onofre facies of the Temblor Formation" based on occurrence of Turritella ocoyana fauna in sandstone underlying breccia. Includes two parts: Tso (breccia)- green, greenish-gray, gray, brown, and white, massive to well bedded, mostly well indurated breccia with interbedded conglomerate, sandstone, siltstone, and mudstone; Tsoss (sandstone)- greenish-gray and brown lithic sandstone. The clasts of the breccia are large and angular and were derived from basement rock sources offshore to the west. They are characterized by blueschist and related rocks derived from Catalina Schist (Woodford, 1924). Unit is up to 900 m thick

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.

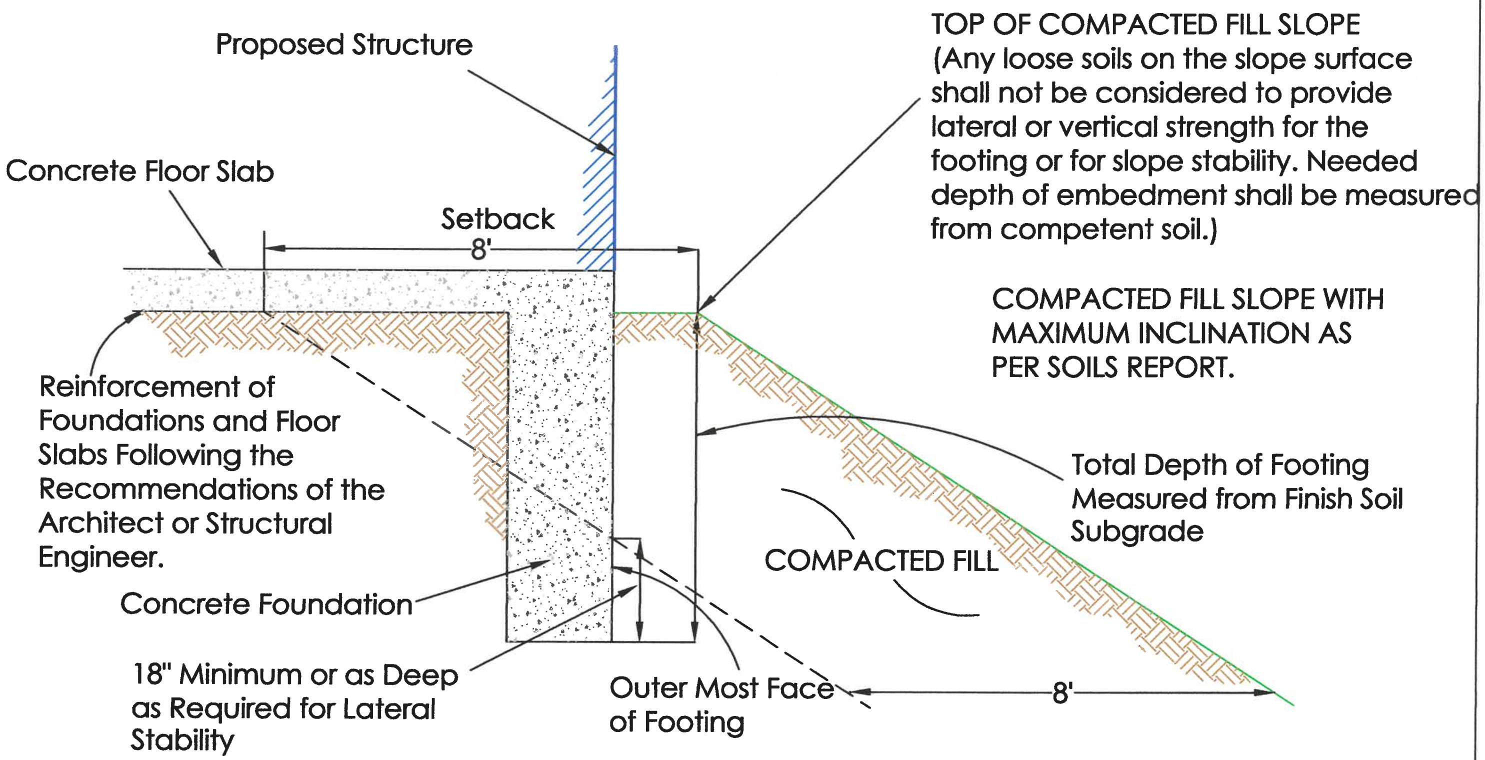
This map was funded in part by the U.S. Geological Survey National Cooperative Geologic Mapping Program, STATEMAP Award no. 98HQAG2049.

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

Breeze-2008-geo.ai

Figure No. V
Job No. 15-10805
Geotechnical Exploration, Inc.
August 2016

FOUNDATION REQUIREMENTS NEAR SLOPES



TYPICAL SECTION

(Showing Proposed Foundation Located Within 8 Feet of Top of Slope)

18" FOOTING / 8' SETBACK

Total Depth of Footing

Distance From Top of Slope	1.5:1.0 SLOPE *	2.0:1.0 SLOPE
0	82"	66"
2'	66"	54"
4'	51"	42"
6'	34"	30"
8'	18"	18"

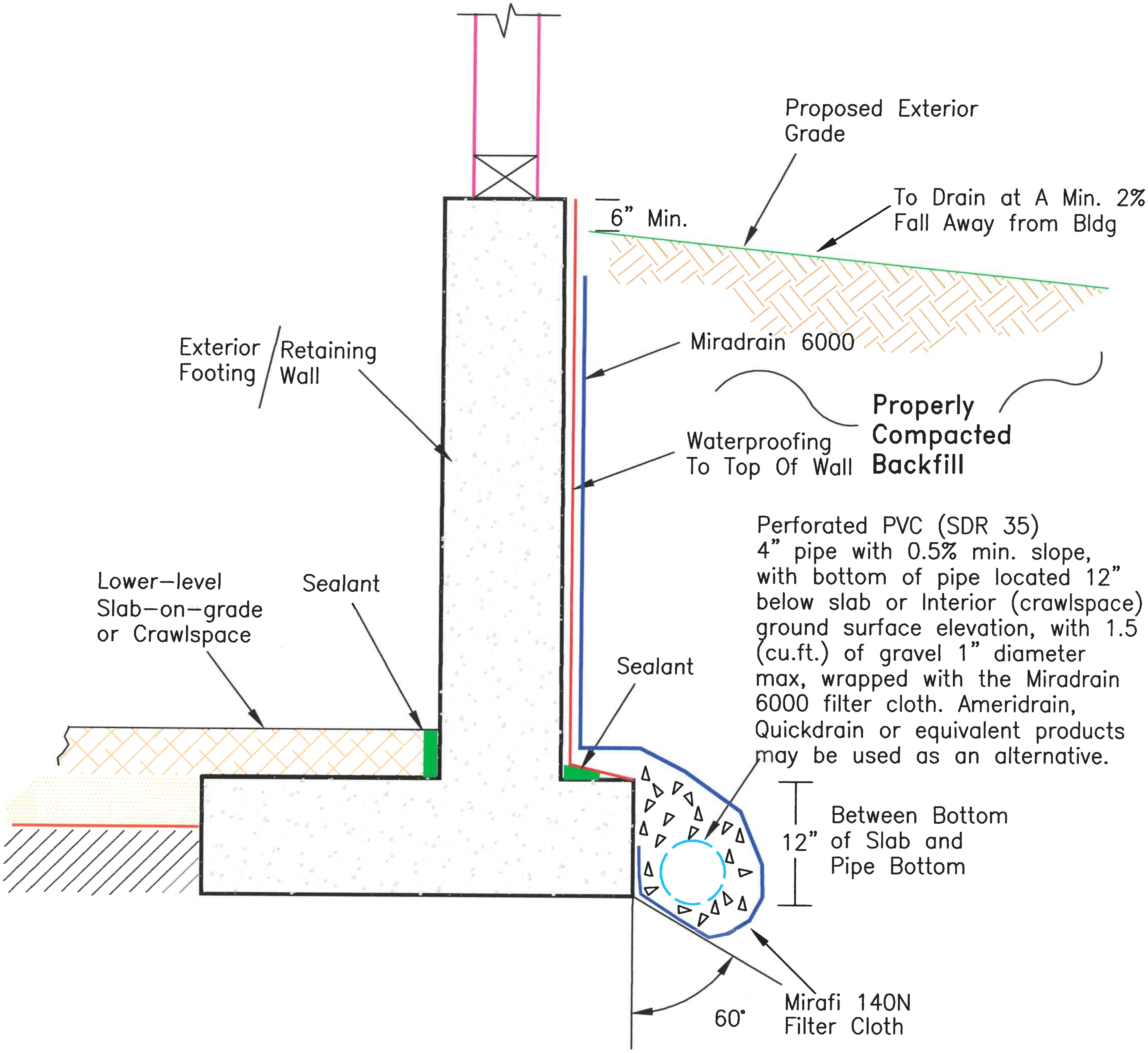
* when applicable

Figure No. VI
Job No. 15-10805

Geotechnical Exploration, Inc.

15-10805-VI

RECOMMENDED SUBGRADE RETAINING WALL DRAINAGE SCHEMATIC



NOT TO SCALE

Figure No. VII

Job No. 15-10805

Geotechnical Exploration, Inc.

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 interior face of the wall and 2.0 cu.ft./ft. of pipe gravel envelope.

15–10805–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, clean clays.
	OL	Organic silts and organic silty clays of low plasticity.
Liquid Limit Greater than 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
	CH	Inorganic clays of high plasticity, fat clays.
	OH	Organic clays of medium to high plasticity.
HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils

(rev. 6/05)

APPENDIX B
MODIFIED MERCALLI INTENSITY SCALE OF 1931
(Excerpted from the California Division of Conservation Division of Mines and Geology DMG Note 32)

The first scale to reflect earthquake intensities was developed by deRossi of Italy, and Forel of Switzerland, in the 1880s, and is known as the Rossi-Forel Scale. This scale, with values from I to X, was used for about two decades. A need for a more refined scale increased with the advancement of the science of seismology, and in 1902, the Italian seismologist Mercalli devised a new scale on a I to XII range. The Mercalli Scale was modified in 1931 by American seismologists Harry O. Wood and Frank Neumann to take into account modern structural features.

The Modified Mercalli Intensity Scale measures the intensity of an earthquake's effects in a given locality, and is perhaps much more meaningful to the layman because it is based on actual observations of earthquake effects at specific places. It should be noted that because the damage used for assigning intensities can be obtained only from direct firsthand reports, considerable time -- weeks or months -- is sometimes needed before an intensity map can be assembled for a particular earthquake.

On the Modified Mercalli Intensity Scale, values range from I to XII. The most commonly used adaptation covers the range of intensity from the conditions of *"I -- not felt except by very few, favorably situated,"* to *"XII -- damage total, lines of sight disturbed, objects thrown into the air."* While an earthquake has only one magnitude, it can have many intensities, which decrease with distance from the epicenter.

It is difficult to compare magnitude and intensity because intensity is linked with the particular ground and structural conditions of a given area, as well as distance from the earthquake epicenter, while magnitude depends on the energy released at the focus of the earthquake.

I	Not felt except by a very few under especially favorable circumstances.
II	Felt only by a few persons at rest, especially on upper floors of buildings. Delicately suspended objects may swing.
III	Felt quite noticeably indoors, especially on upper floors of buildings, but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing of truck. Duration estimated.
IV	During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, windows, doors disturbed; walls make cracking sound. Sensation like heavy truck striking building. Standing motor cars rocked noticeably.
V	Felt by nearly everyone, many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbances of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop.
VI	Felt by all, many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.
VII	Everybody runs outdoors. Damage negligible in building of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motor cars.
VIII	Damage slight in specially designed structures; considerable in ordinary substantial buildings, with partial collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Persons driving motor cars disturbed.
IX	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; great in substantial buildings with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.
X	Some well-built wooden structures destroyed; most masonry and frame structures destroyed with foundations; ground badly cracked. Rails bent. Landslides considerable from riverbanks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks.
XI	Few, if any, masonry structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.
XII	Damage total. Practically all works of construction are damaged greatly or destroyed. Waves seen on ground surface. Lines of sight and level are distorted. Objects thrown upward into the air.

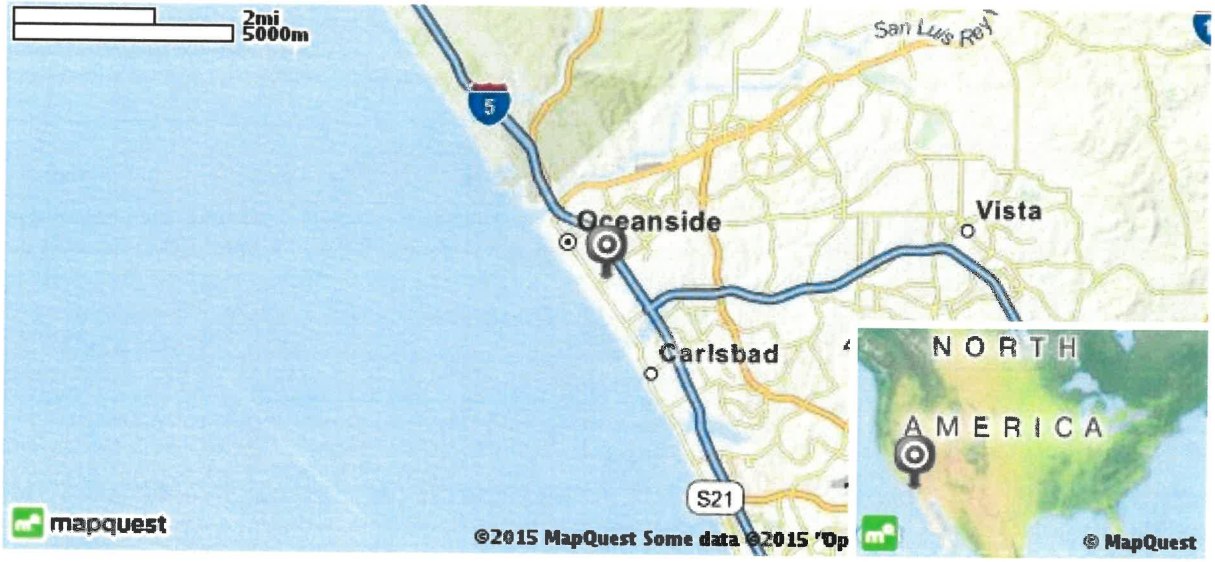
APPENDIX C

USGS DESIGN MAPS SUMMARY REPORT

USGS Design Maps Summary Report

User–Specified Input

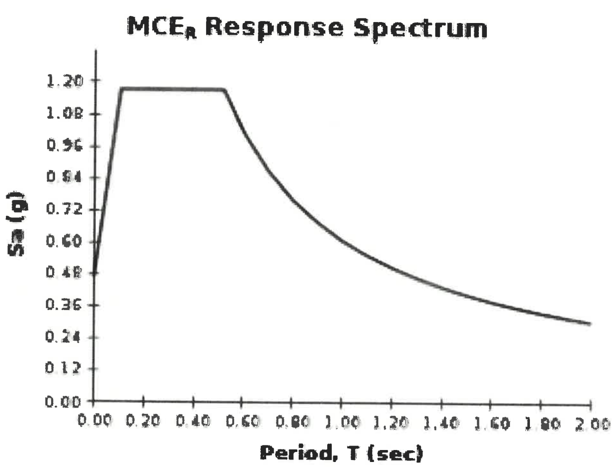
Report Title	1200 Block of Nevada St, Ocenside, CA Tue July 28, 2015 18:25:32 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	33.1849°N, 117.3659°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Risk Category	I/II/III

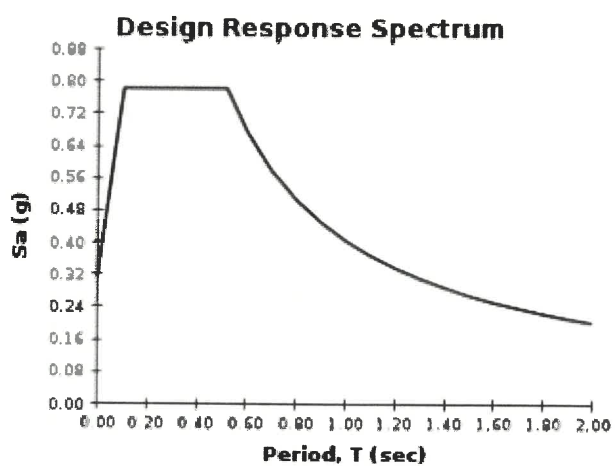


USGS–Provided Output

S_S =	1.173 g	S_{MS} =	1.173 g	S_{DS} =	0.782 g
S_1 =	0.451 g	S_{M1} =	0.608 g	S_{D1} =	0.406 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

APPENDIX D

SLOPE STABILITY ANALYSIS

SLOPE STABILITY CALCULATIONS WITH SLIDE 6 COMPUTER PROGRAM

BREEZE OCEANSIDE APARTMENTS

GEI Job No. 15-10805

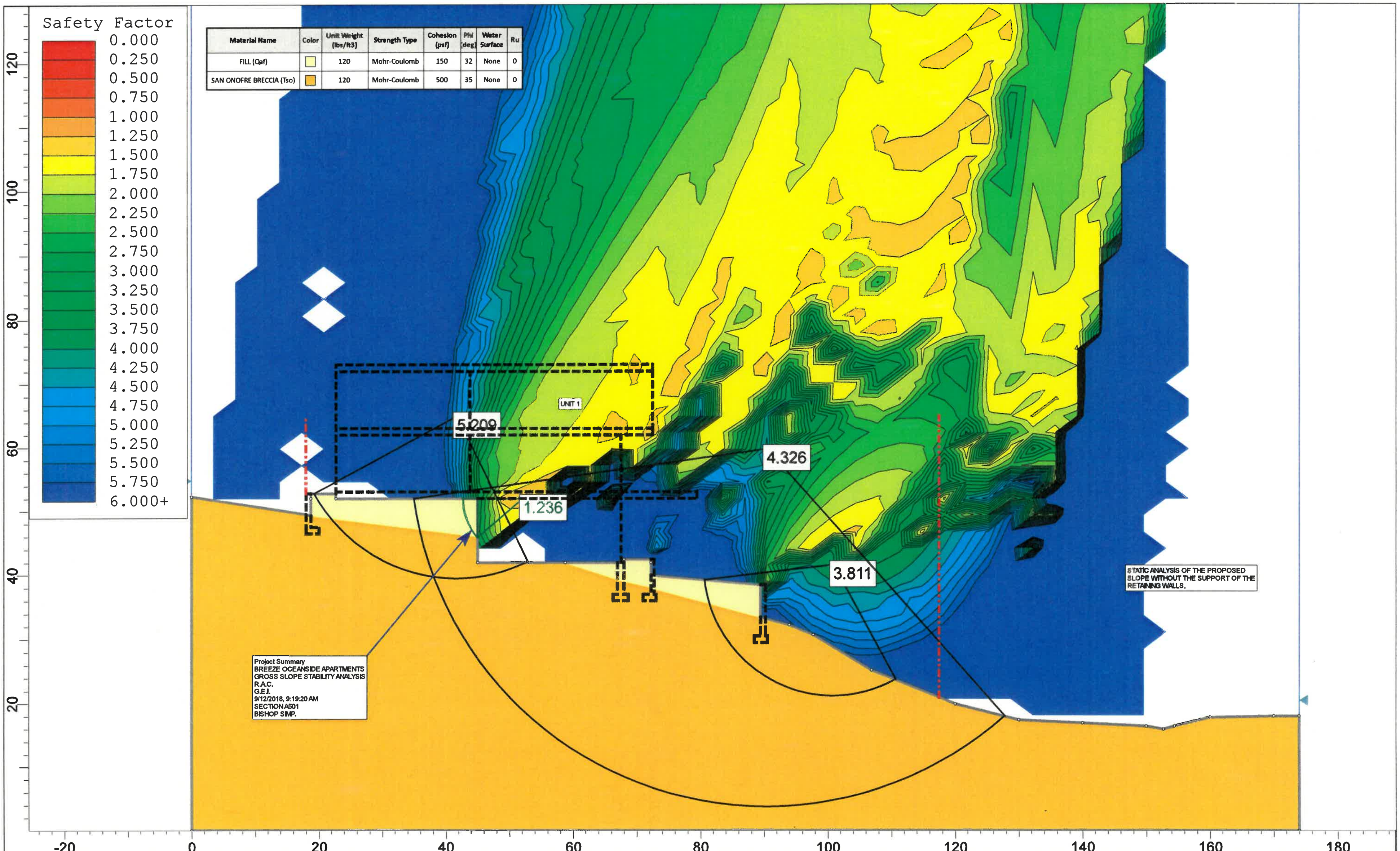
We performed gross slope stability calculations using the *SLIDE 6* program by Roc Science. The program is a limit equilibrium slope stability program that allows the use of several slope stability methods to calculate the factors of safety against shear failure. On this project, we used the Bishop Simplified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

The program calculates the factor of safety against failure of potential slide surfaces for a selected range. We chose the range of slide surfaces where failures are most likely to occur. The printout shows a block with contours of different colors and shades that correspond to the different factors of safety calculated that can be obtained for the analyzed range of slide surfaces for Sections 1, 2, 4, 5, 7, 14, 27, 28 in our report (see attached printouts). The green circular surface displayed is the lowest possible factor of safety located within the specified search range. Soil strength values, geometry, and water conditions (water was not encountered) used in the program were based on geological information at the site obtained by our project geologist. Direct shear test results from soils obtained at the site, as well as slopes in the vicinity of the site, were used for the gross slope stability analysis. Shear strength values have been conservatively adjusted.

The proposed slopes were evaluated without and with the support of the retaining walls. An equivalent lateral fluid pressure of 45-pcf (pounds per cubic feet) was used for the basement retaining walls and 38-pcf for the slope and exterior retaining walls.

Once the static gross stability of different slide planes was calculated, we analyzed the same sections including a seismic lateral force of 0.15g to obtain the factor of safety for seismic conditions. The calculated factors of safety for both static and seismic analysis yielded values that are considered acceptable, i.e., 1.5 or higher for static load analysis, and 1.15 for seismic analysis.

The surficial slope stability calculations were performed on the different slope segments measured on the slope faces of sections along the different slopes by using a geotechnical accepted equation for infinite slopes with a saturated upper layer. The calculations were performed by assuming that the upper 3 feet of those soils were saturated and the slope segment analyzed had infinite length. The calculations yielded the factor of safety against shear failure of a sliding block 3 feet high against the soil shear strength frictional and cohesion strength opposing the driving force. The calculated factors of safety also yielded factors of safety that are equal or higher than the minimum acceptable of 1.5. The soil strength values used for the shallow stability calculations were conservatively adjusted.



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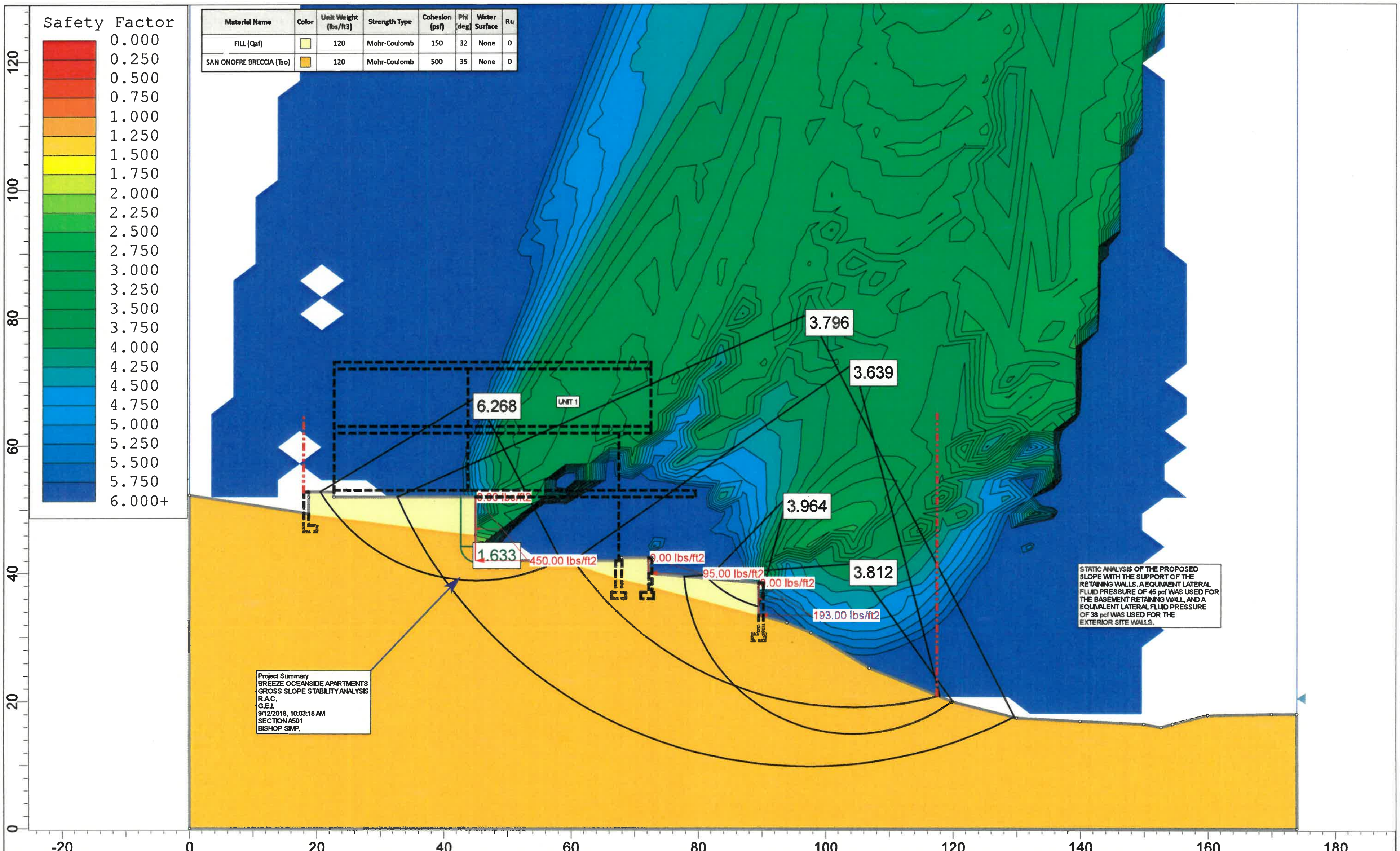
FILL (Qaf)		120	Mohr-Coulomb	150	32	None	0
SAN ONOFRE BRECCIA (Tso)		120	Mohr-Coulomb	500	35	None	0

Geotechnical Exploration, Inc.

Project	BREEZE OCEANSIDE APARTMENTS
Analysis Description	GROSS SLOPE STABILITY ANALYSIS
Drawn By	R.A.C.
Scale	1:250
Company	G.E.I.
Date	9/12/2018, 9:19:20 AM
File Name	JOB NO. 15-10805_S1_01.slim

SLIDEINTERPRET 6.039



Geotechnical Exploration, Inc.

Project	BREEZE OCEANSIDE APARTMENTS		
Analysis Description	GROSS SLOPE STABILITY ANALYSIS		
Drawn By	R.A.C.	Scale	1:250
Company	G.E.I.		
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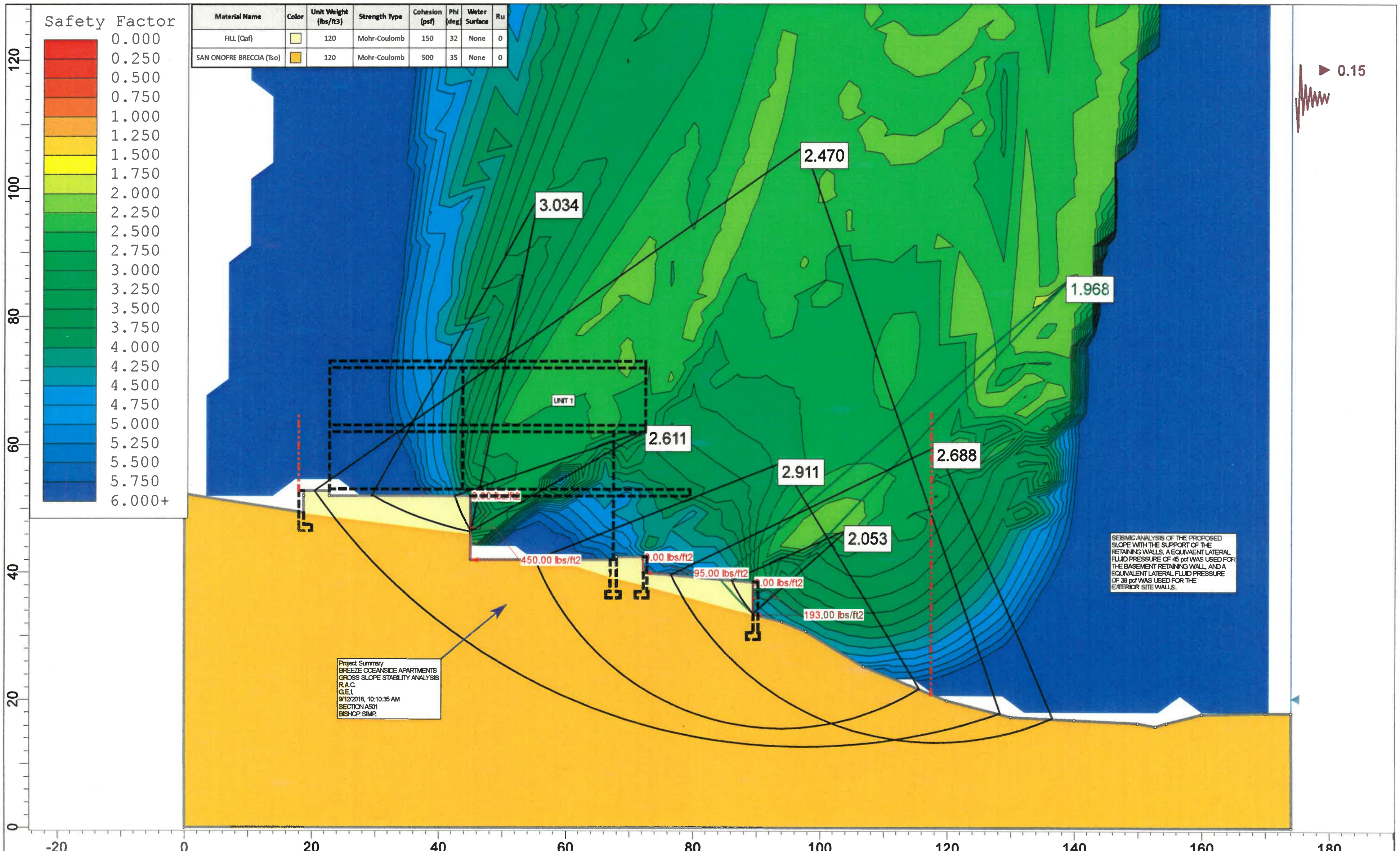

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
FILL (Qaf)		120	Mohr-Coulomb	150	32	None	0
SAN ONOFRE BRECCIA (Tso)		120	Mohr-Coulomb	500	35	None	0

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	BREEZE OCEANSIDE APARTMENTS		
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Drawn By	R.A.C.	Scale	1:250
Company	G.E.I.		
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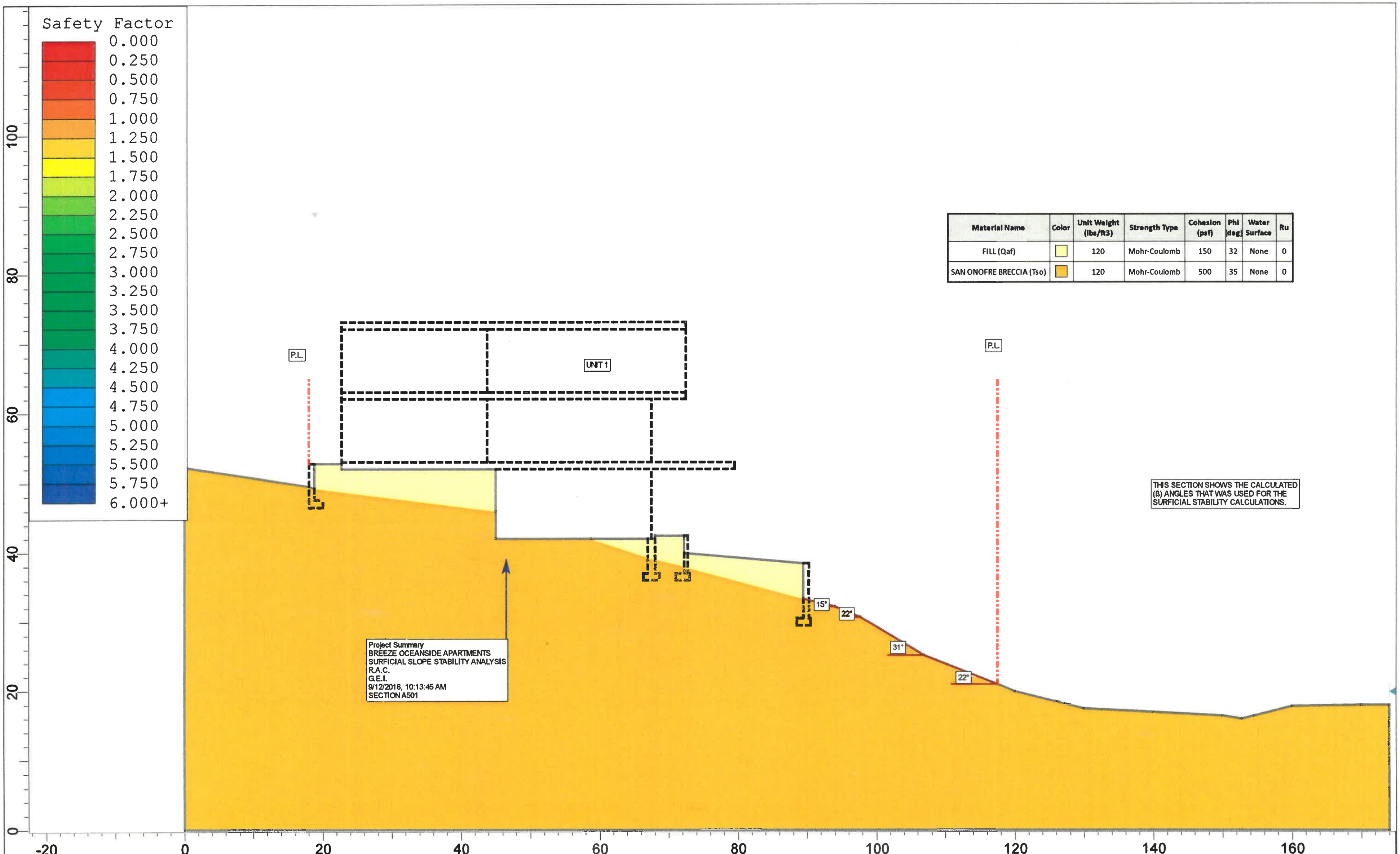

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FILL (Qaf)		120	Mohr-Coulomb	150	32	None	0
SAN ONOFRE BRECCIA (Tso)		120	Mohr-Coulomb	500	35	None	0

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	BREEZE OCEANSIDE APARTMENTS
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Company	G.E.I.
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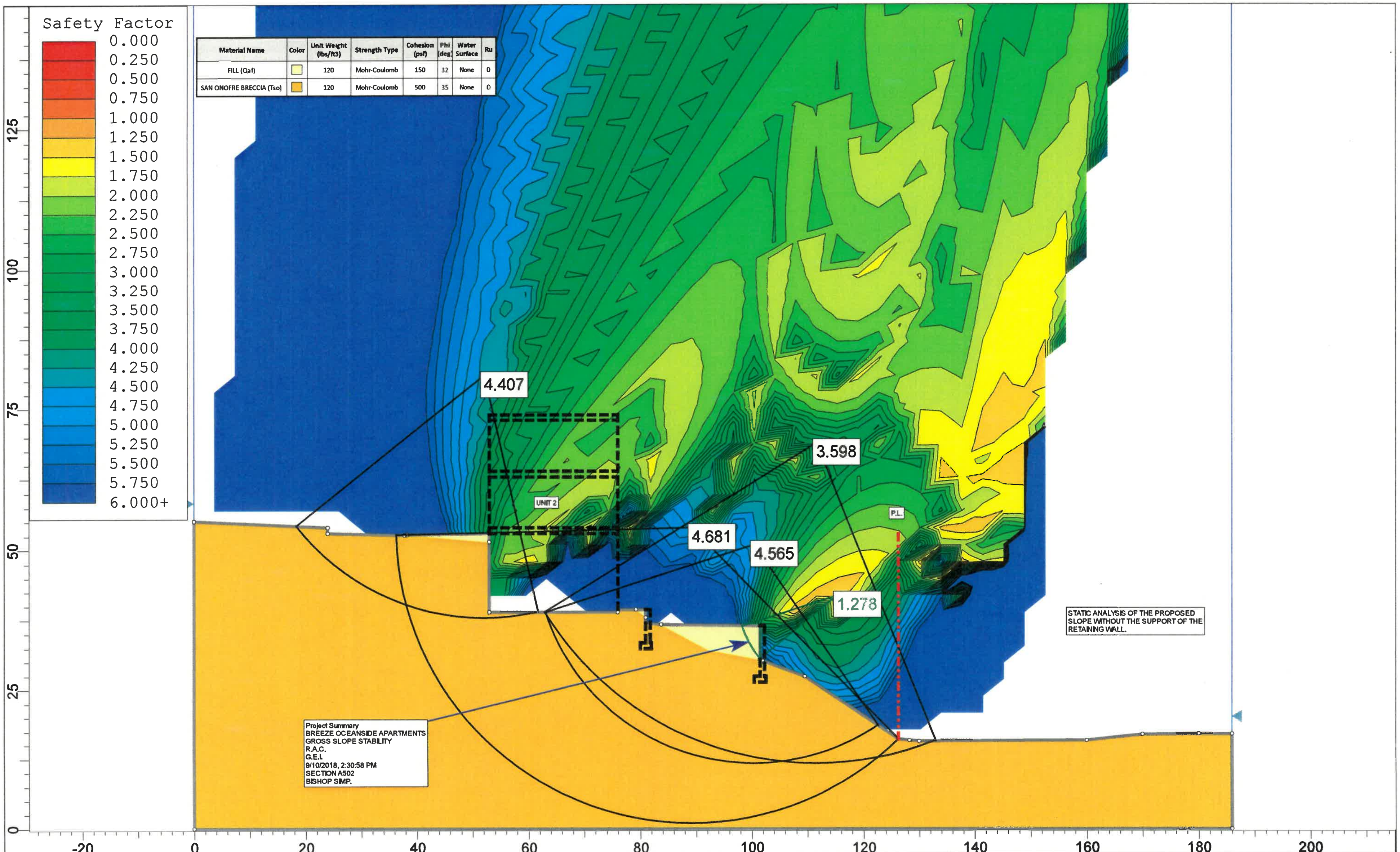

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FILL (Qaf)		120	Mohr-Coulomb	150	32	None	0
SAN ONOFRE BRECCIA (Tso)		120	Mohr-Coulomb	500	35	None	0

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	BREEZE OCEANSIDE APARTMENTS
Analysis Description	GROSS SLOPE STABILITY
Drawn By	R.A.C.
Scale	1:285
Company	G.E.I.
Date	9/10/2018, 2:30:58 PM
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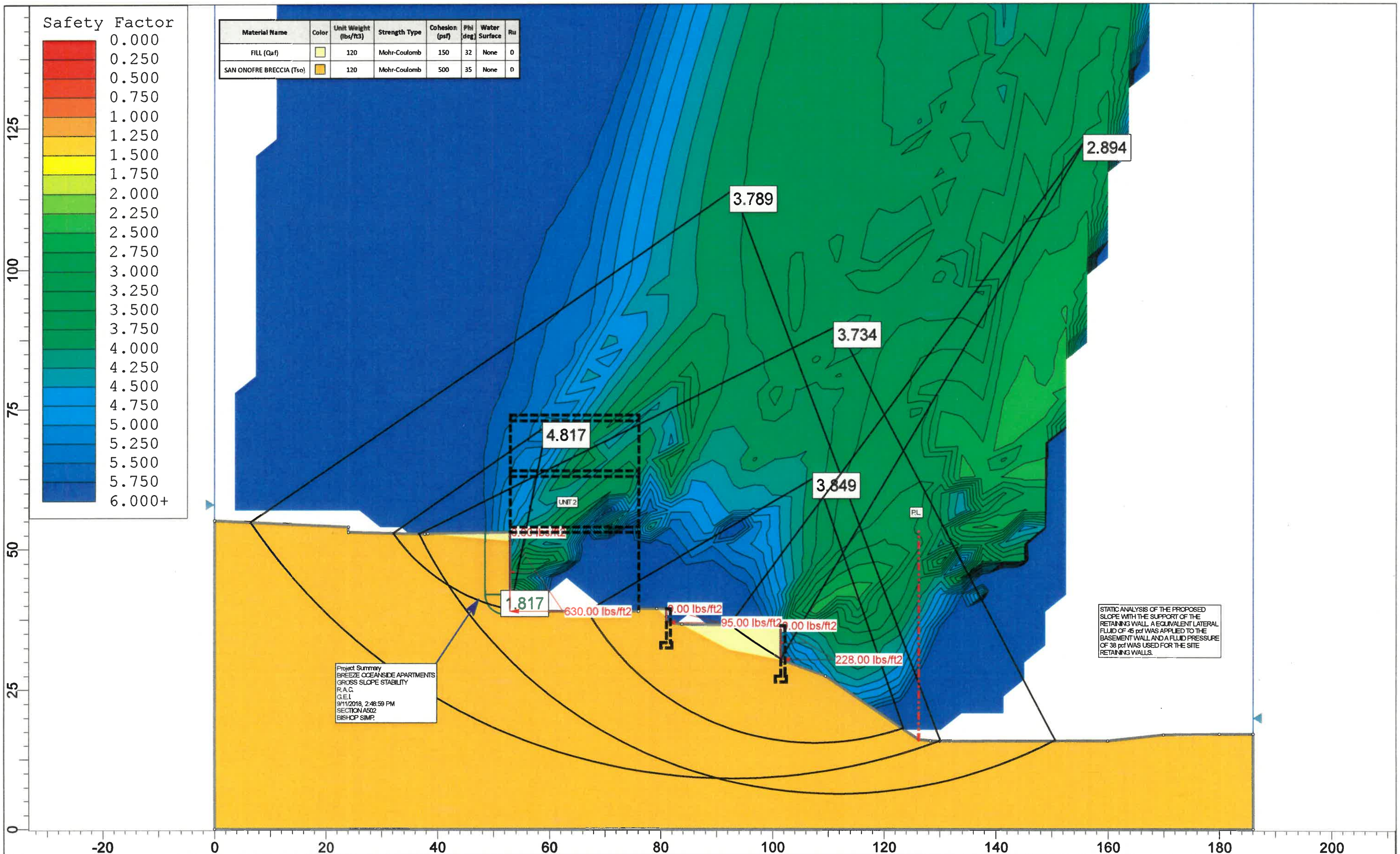

Geotechnical Exploration, Inc.

Project	BREEZE OCEANSIDE APARTMENTS		
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Drawn By	R.A.C.	Scale	1:285
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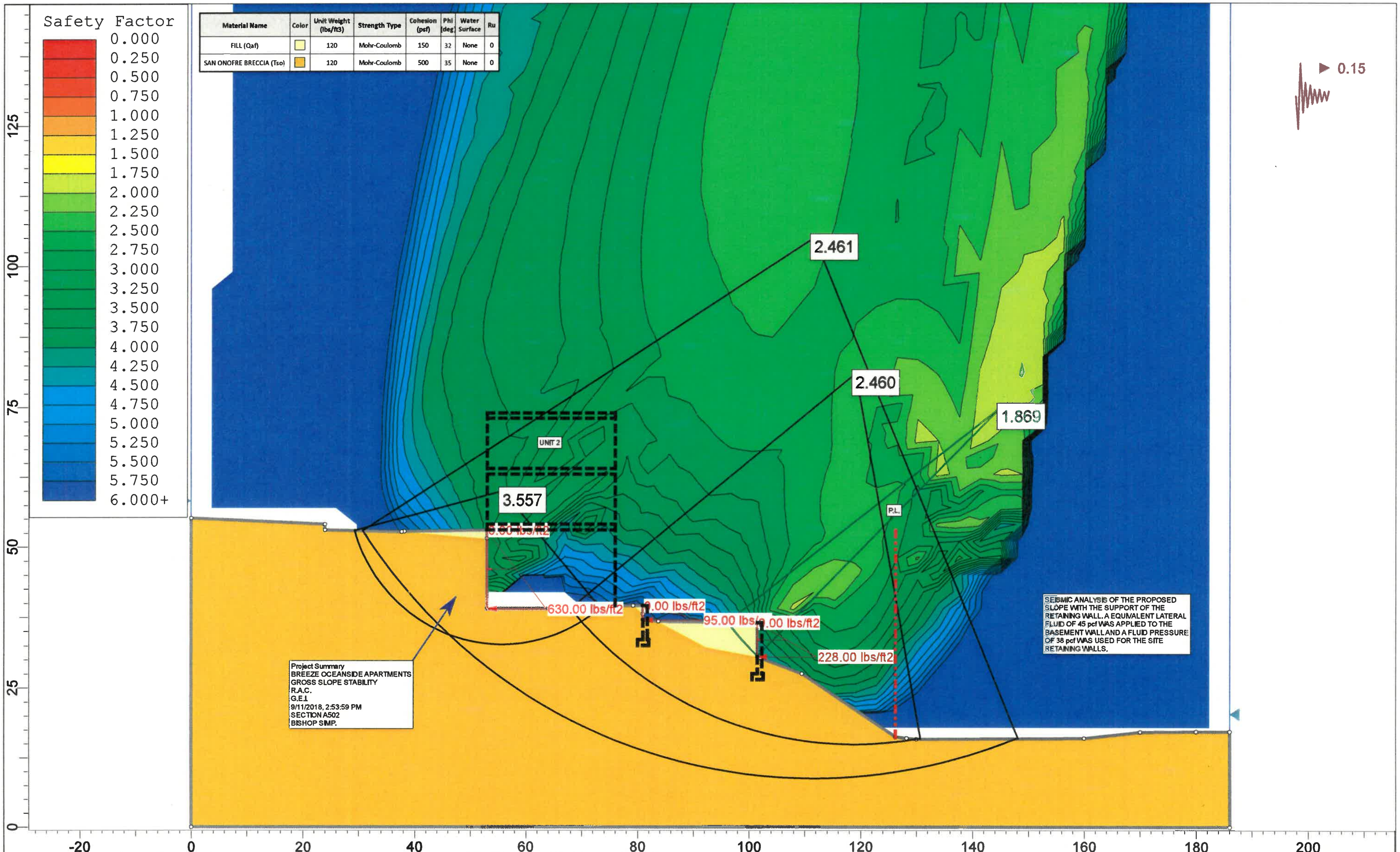
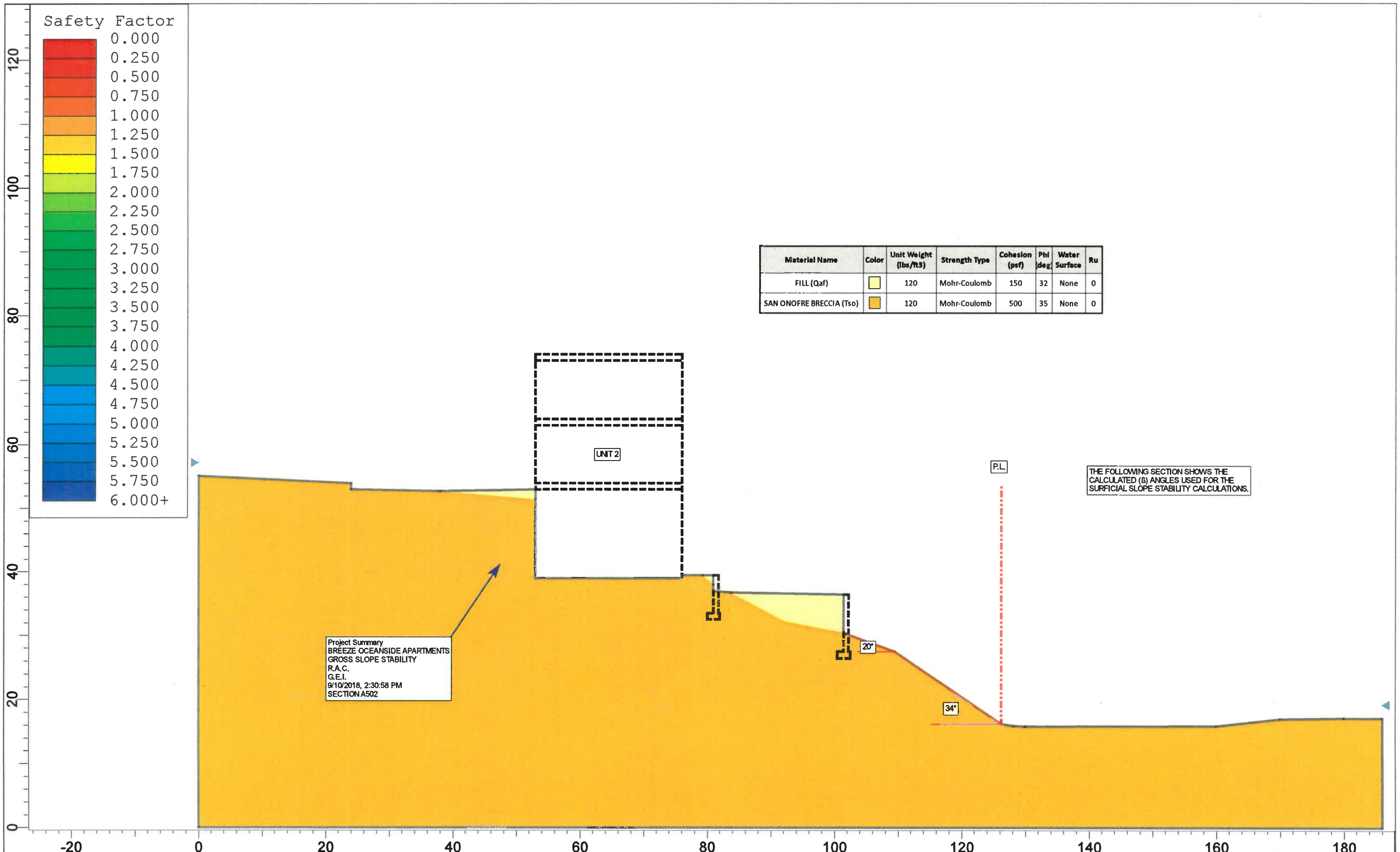


Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	BREEZE OCEANSIDE APARTMENTS		
Analysis Description	GROSS SLOPE STABILITY		
Drawn By	R.A.C.	Scale	1:285
Company	G.E.I.		
Date	9/11/2018, 2:53:59 PM	File Name	JOB NO. 15-10805_S2_02_0.15gSHAKE.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
FILL (Qaf)		120	Mohr-Coulomb	150	32	None	0
SAN ONOFRE BRECCIA (Tso)		120	Mohr-Coulomb	500	35	None	0

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	BREEZE OCEANSIDE APARTMENTS
Analysis Description	GROSS SLOPE STABILITY
Drawn By	R.A.C.
Scale	1:250
Company	G.E.I.
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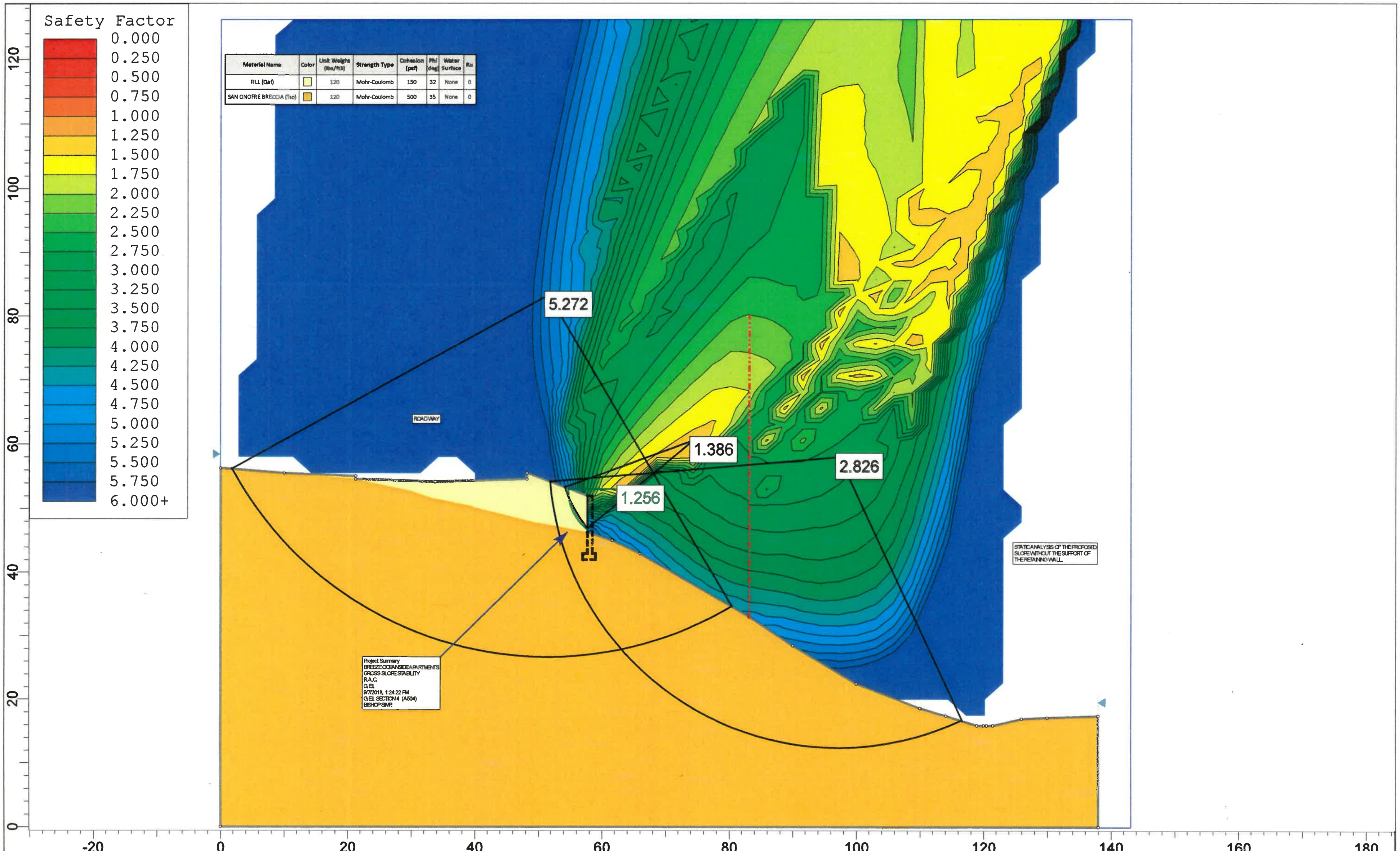

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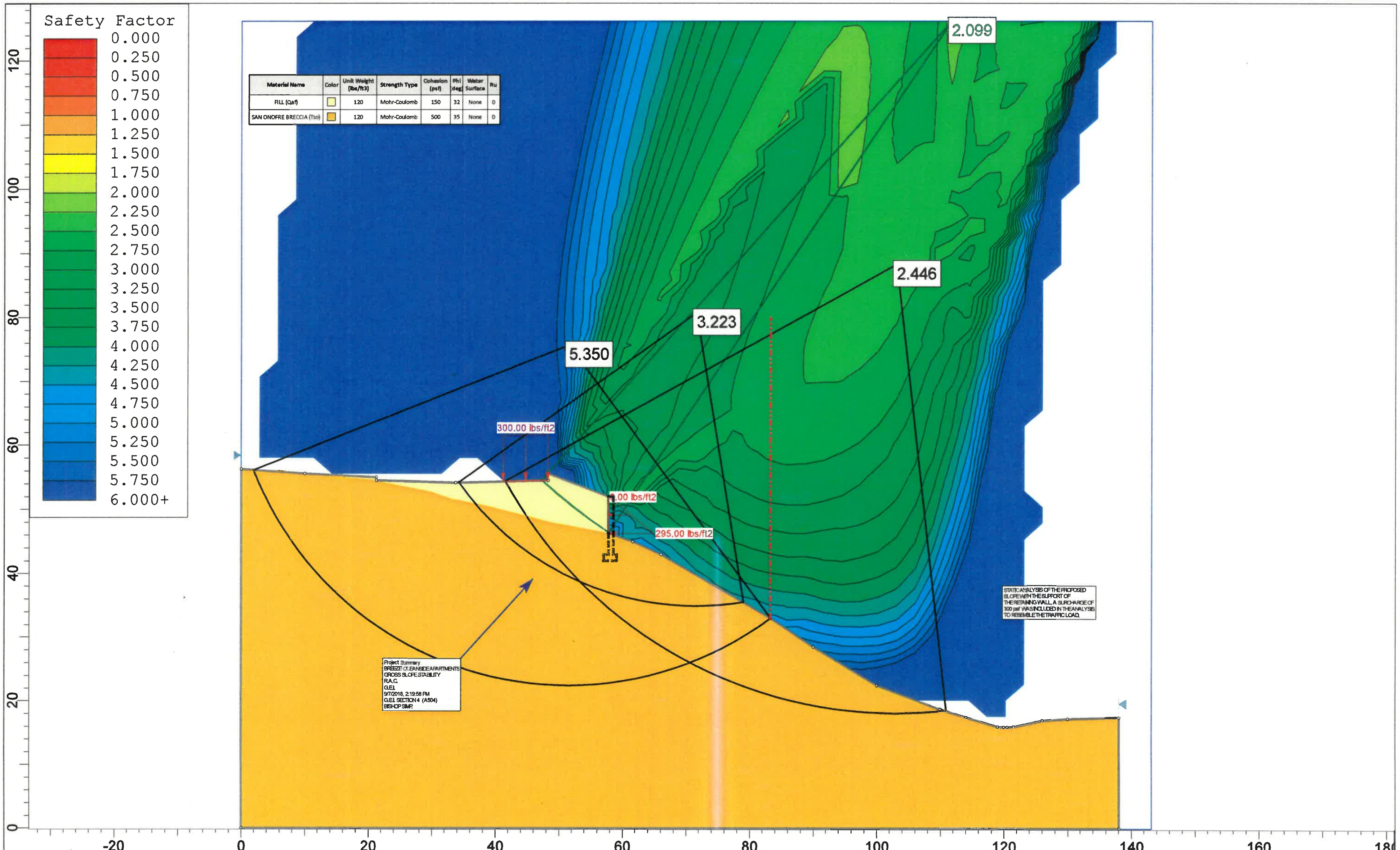

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FILL (Qaf)		120	Mohr-Coulomb	150	32	None	0
SAN ONOFRE BRECCIA (Tso)		120	Mohr-Coulomb	500	35	None	0

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	BREEZE OCEANSIDE APARTMENTS
Analysis Description	GROSS SLOPE STABILITY
Drawn By R.A.C.	*Scale* 1:250
Company	G.E.I.
Date	9/7/2018, 2:19:58 PM
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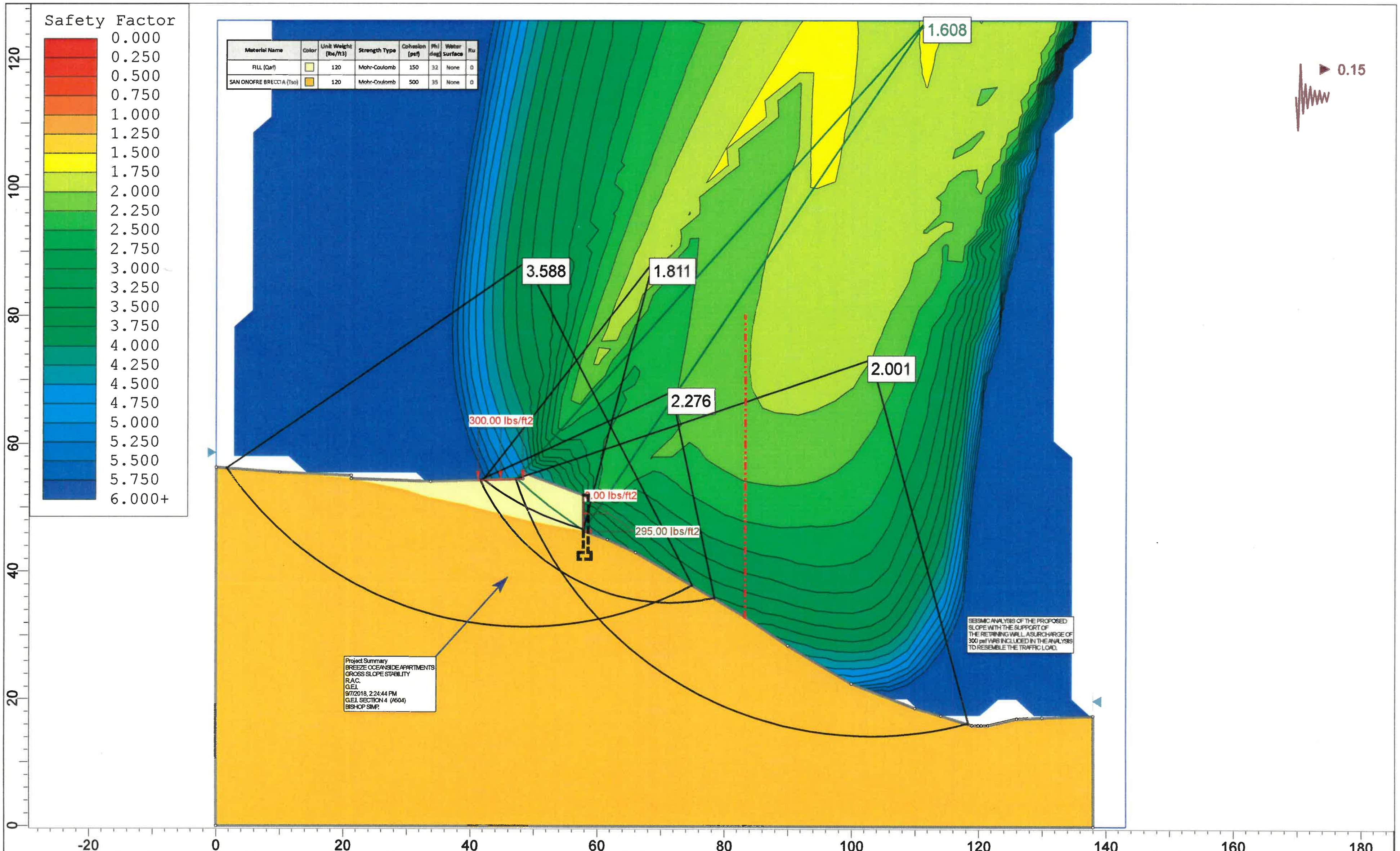

Geotechnical Exploration, Inc.

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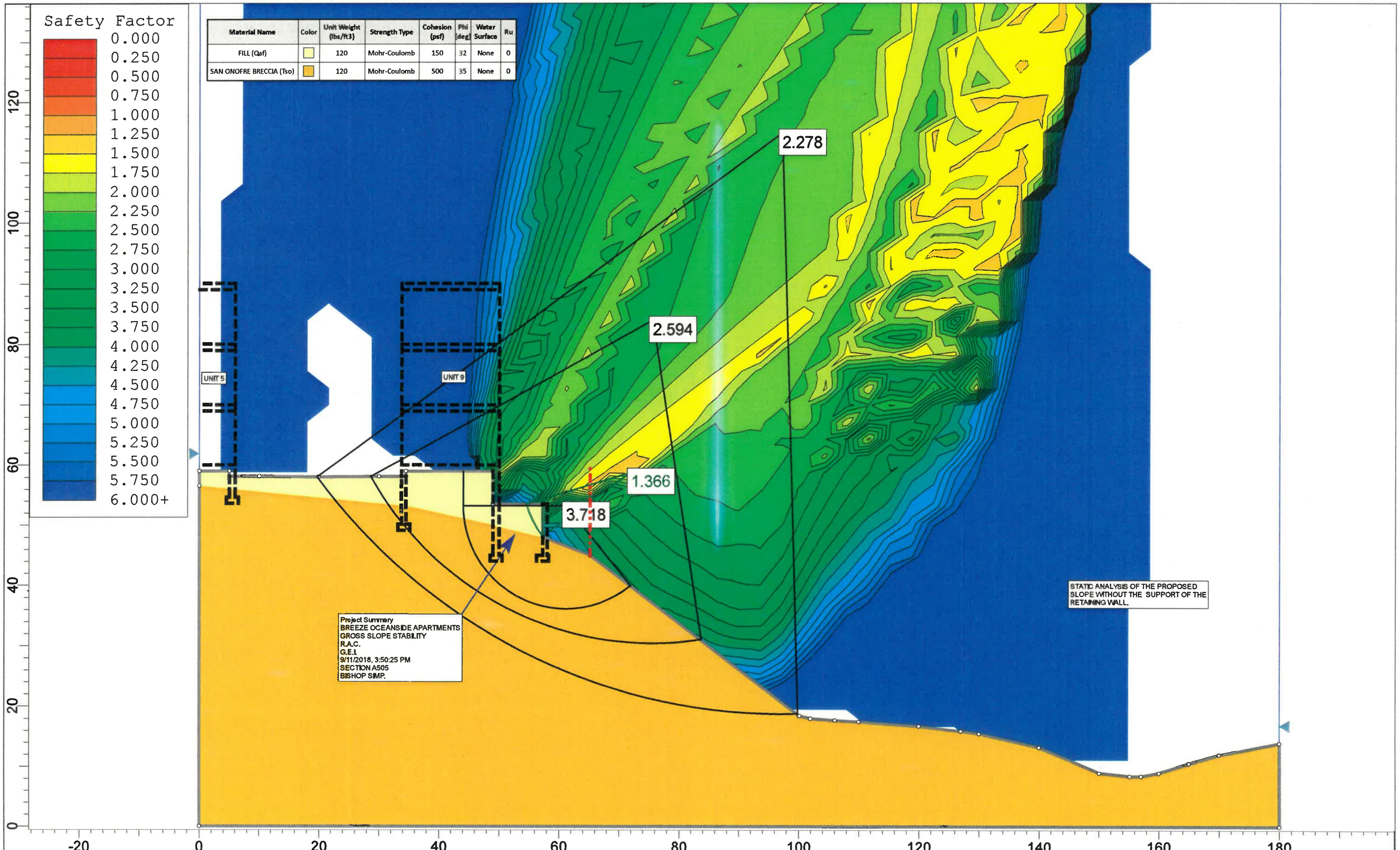

Geotechnical Exploration, Inc.

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Company	G.E.I.		
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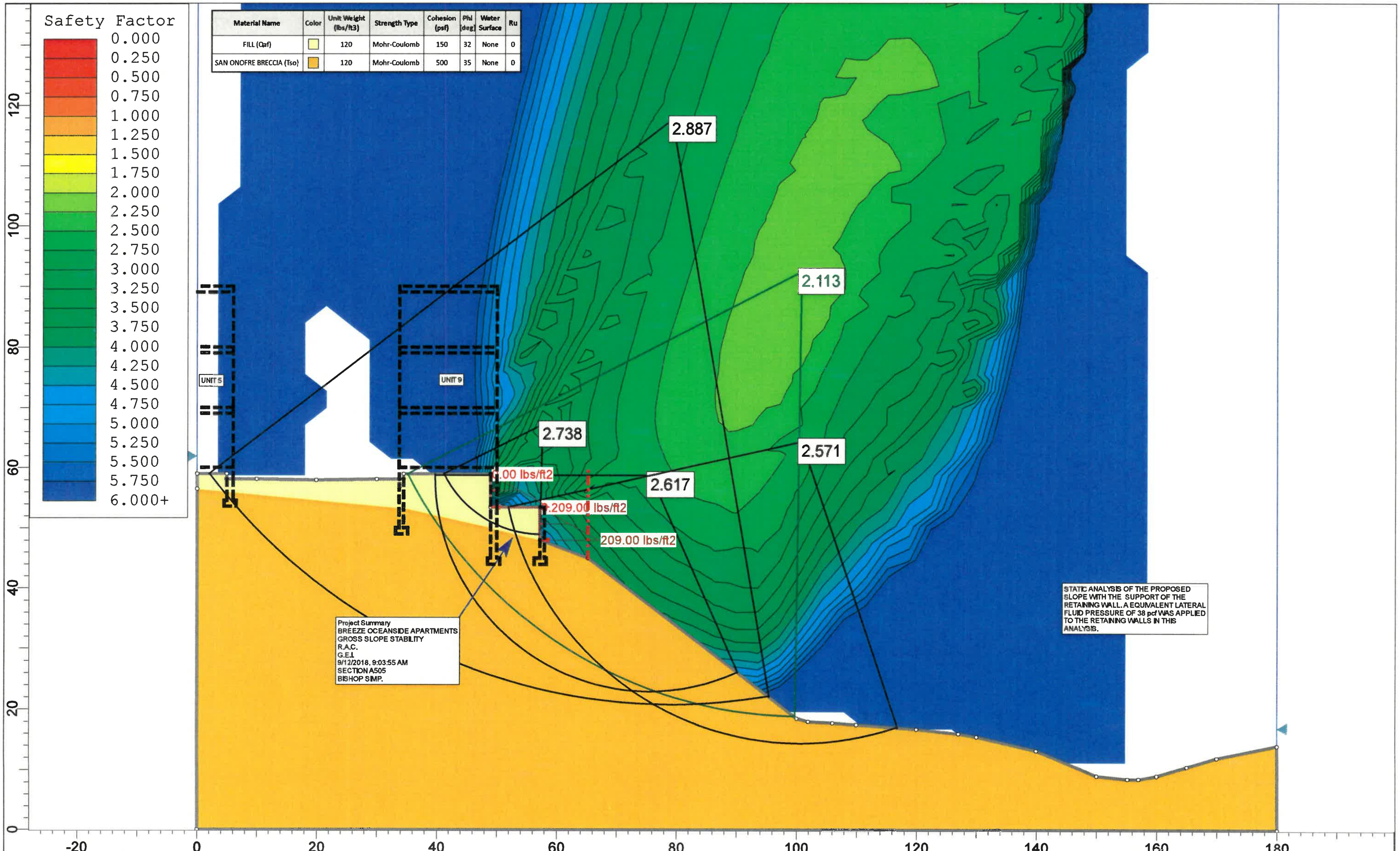

Geotechnical Exploration, Inc.

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Analysis Description	GROSS SLOPE STABILITY		
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Date	9/12/2018, 9:03:55 AM	*File Name*	JOB NO. 15-10805_S5_02.slim

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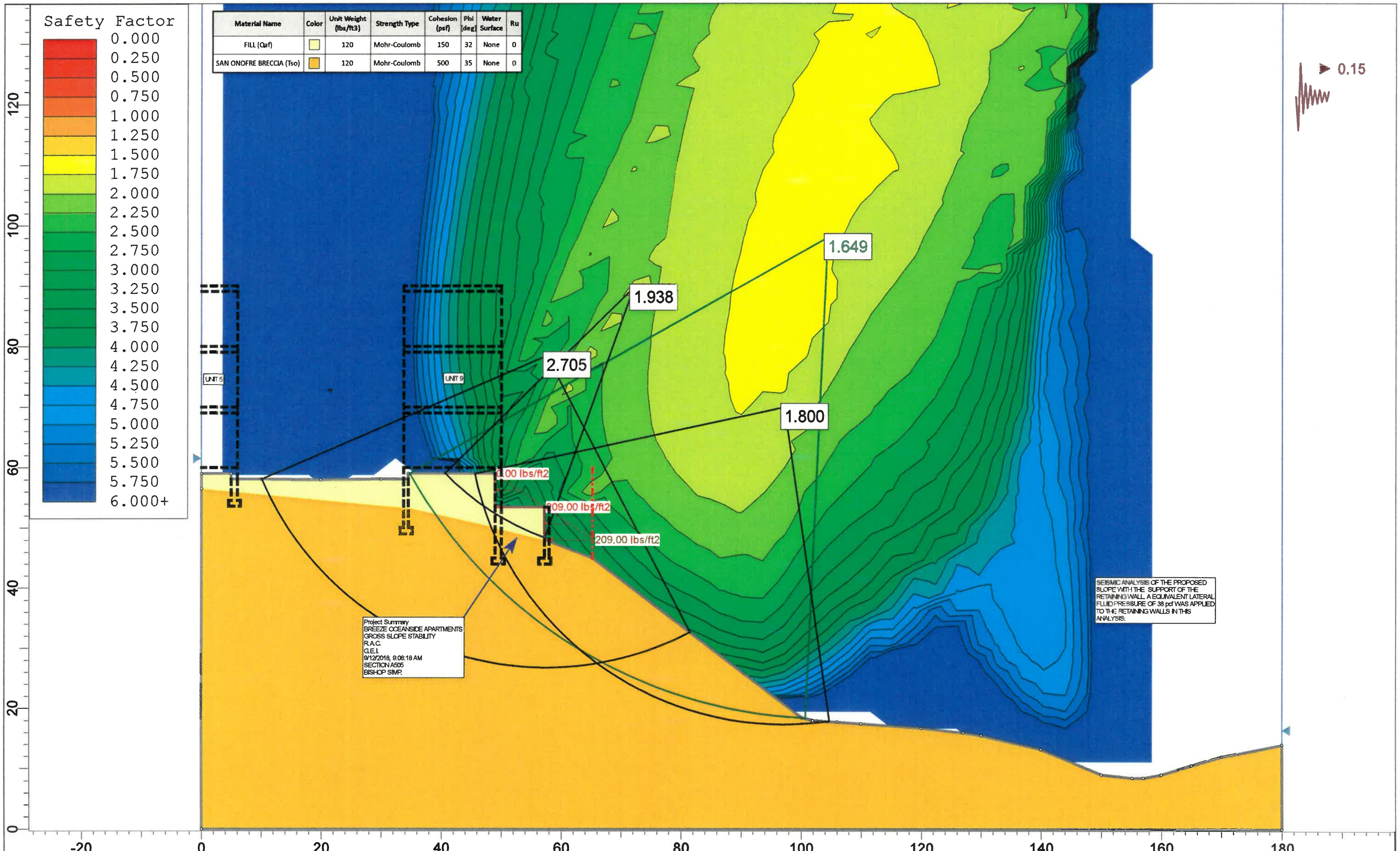

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FILL (Qaf)		120	Mohr-Coulomb	150	32	None	0
SAN ONOFRE BRECCIA (Tso)		120	Mohr-Coulomb	500	35	None	0

Geotechnical Exploration, Inc.

Project	BREEZE OCEANSIDE APARTMENTS		
Analysis Description	GROSS SLOPE STABILITY		
Drawn By	R.A.C.	Scale	1:265
Company	G.E.I.		
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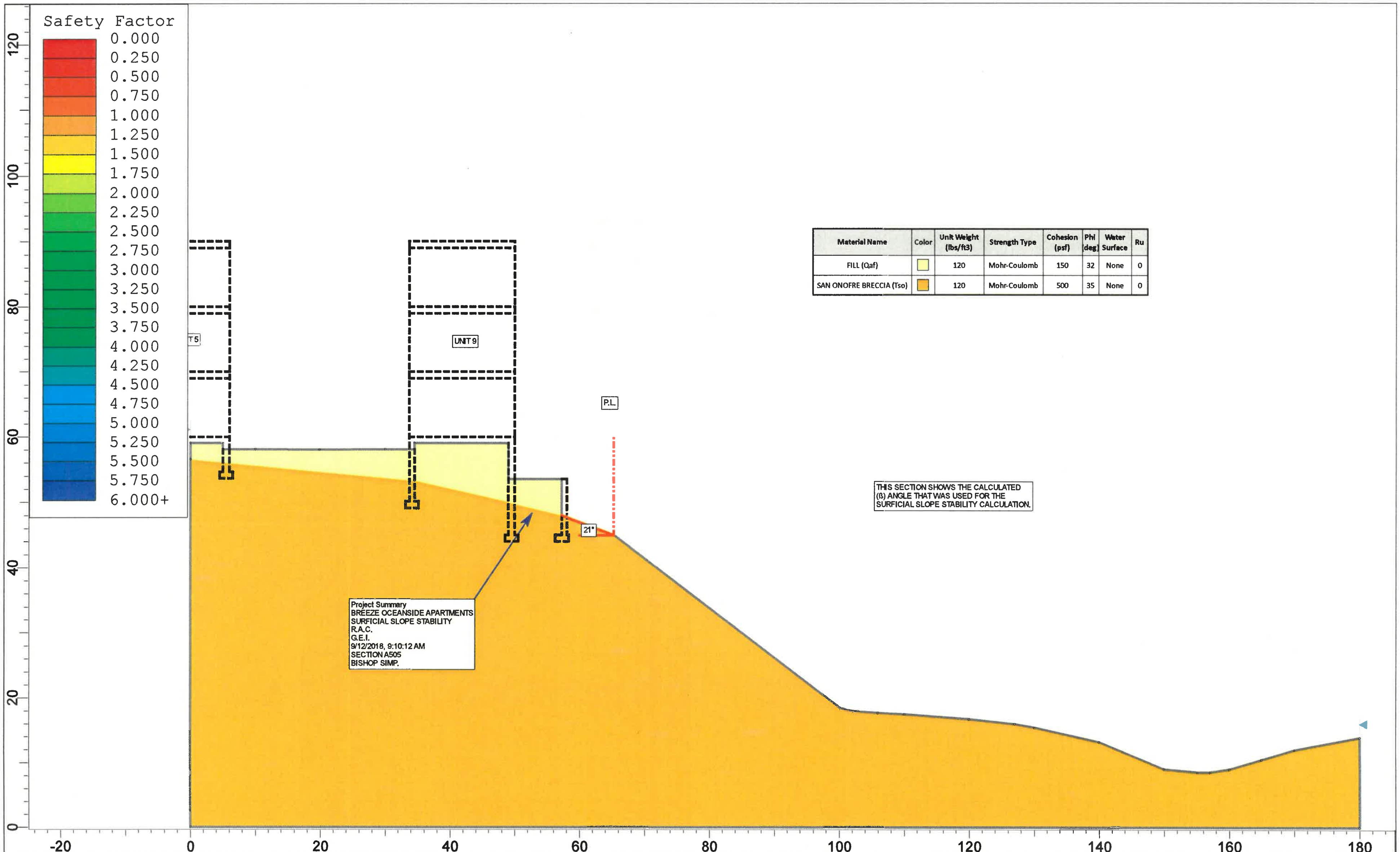

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FILL (Qaf)		120	Mohr-Coulomb	150	32	None	0
SAN ONOFRE BRECCIA (Tso)		120	Mohr-Coulomb	500	35	None	0

Geotechnical Exploration, Inc.

Project	BREEZE OCEANSIDE APARTMENTS
Analysis Description	SURFICIAL SLOPE STABILITY
Drawn By	R.A.C.
Scale	1:245
Company	G.E.I.
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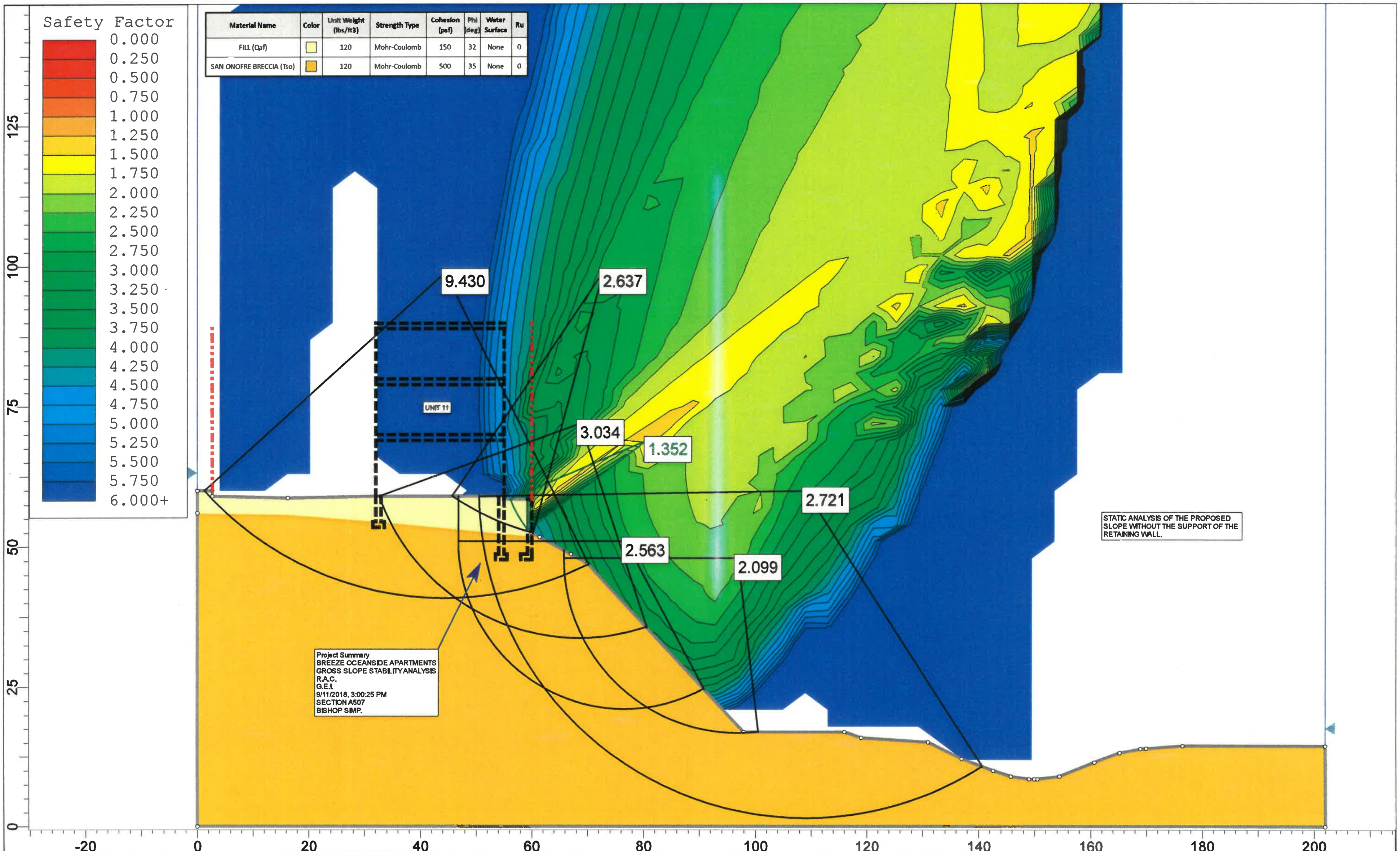
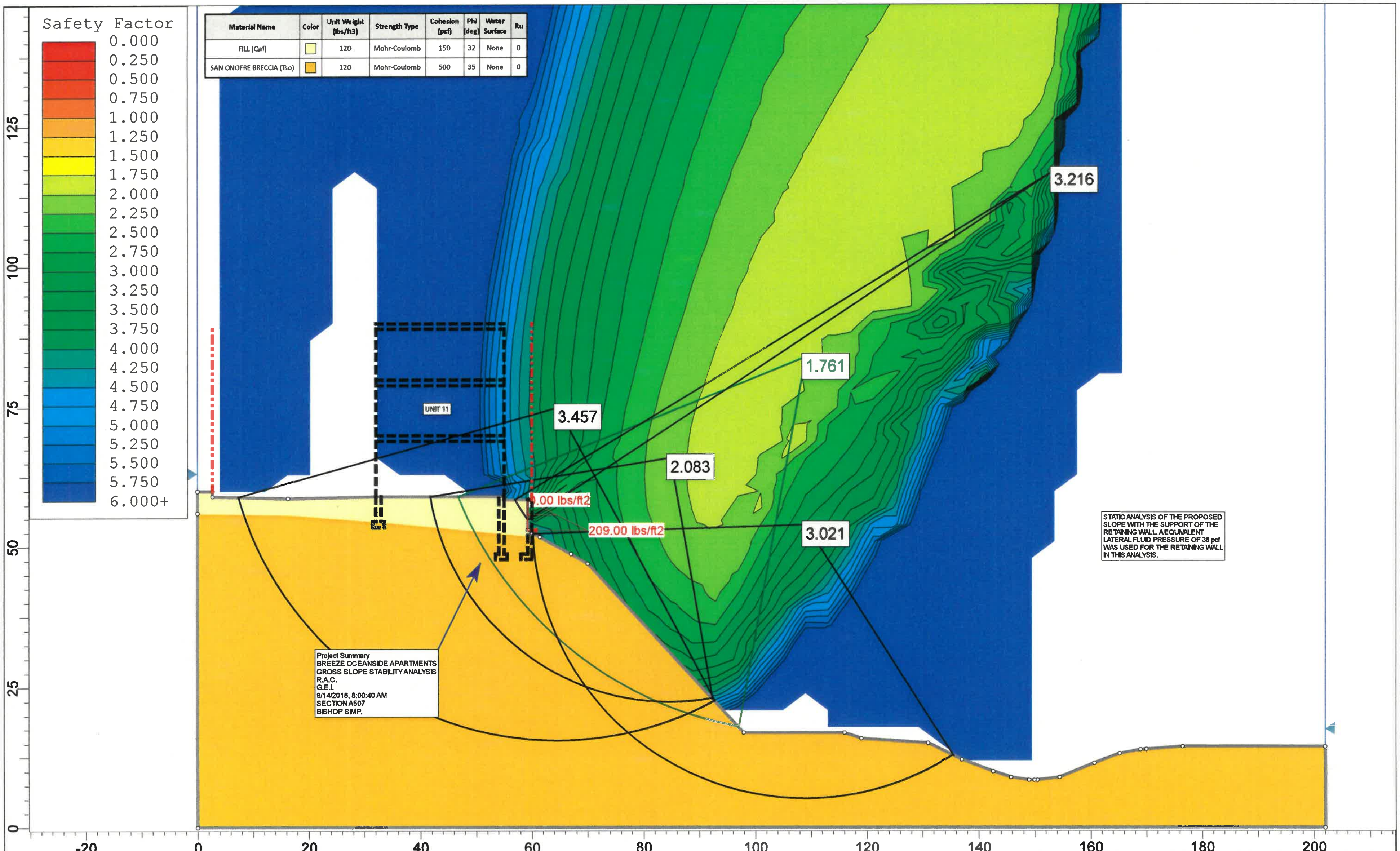


Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	BREEZE OCEANSIDE APARTMENTS
Analysis Description	GROSS SLOPE STABILITY ANALYSIS
Drawn By R.A.C.	*Scale* 1:285	*Company* G.E.I.
Date 9/11/2018, 3:00:25 PM	*File Name* JOB NO. 15-10805_S7_01.slim



Geotechnical Exploration, Inc.

Project	BREEZE OCEANSIDE APARTMENTS		
Analysis Description	GROSS SLOPE STABILITY ANALYSIS		
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Company	G.E.I.		
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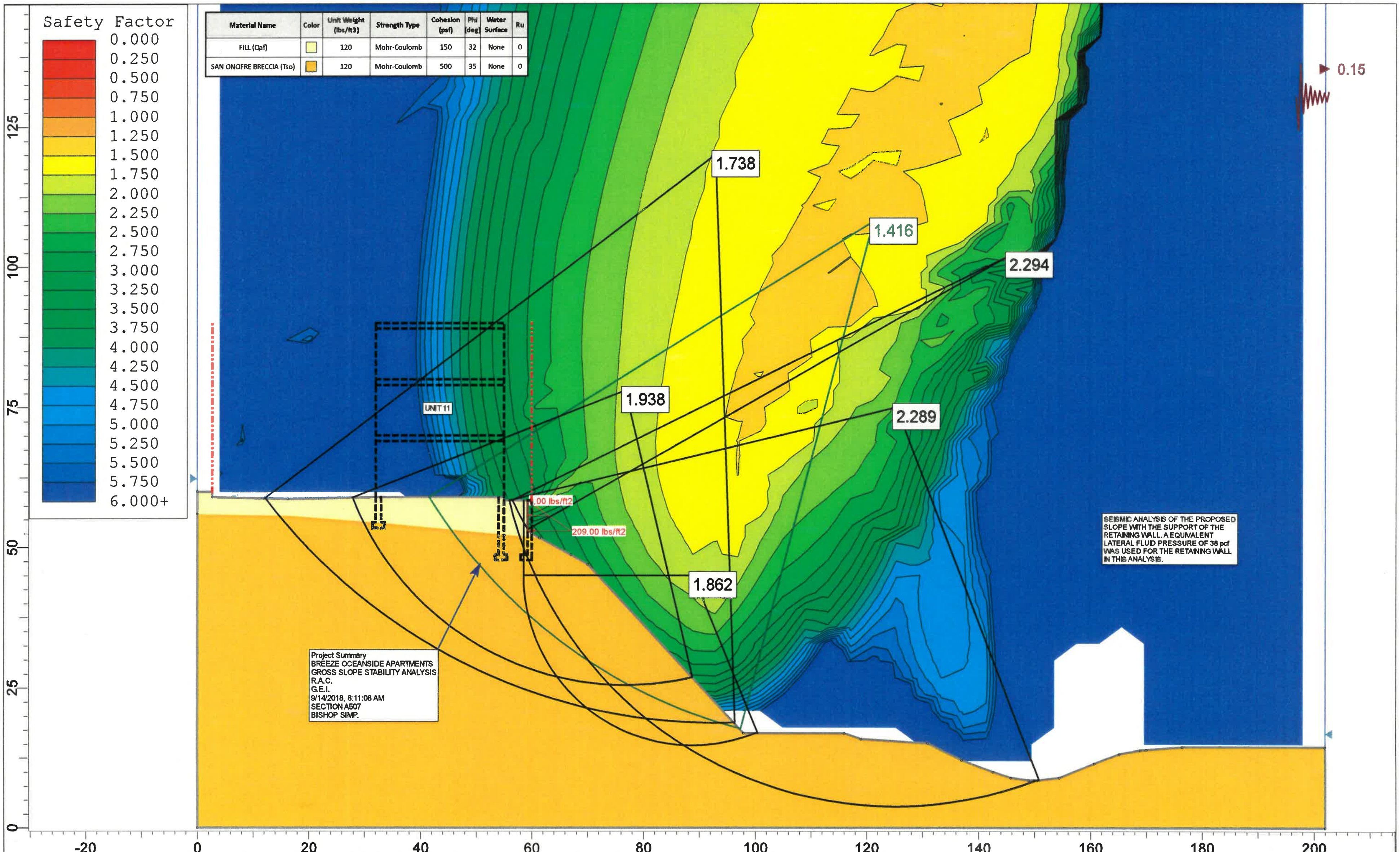

Geotechnical Exploration, Inc.

Project	BREEZE OCEANSIDE APARTMENTS		
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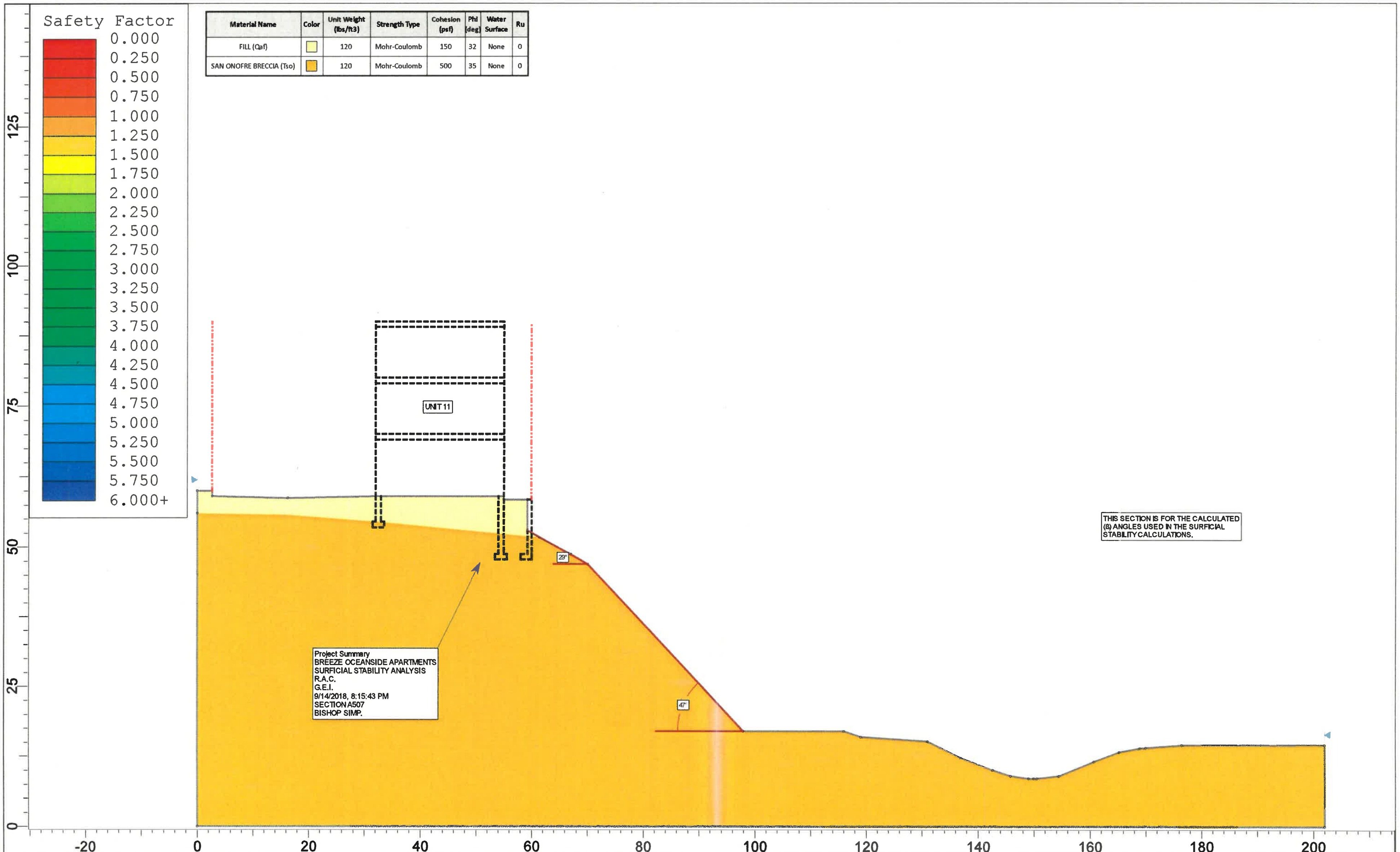
SLIDEINTERPRET 6.039



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SAN ONOFRE BRECCIA (Tso)		120	Mohr-Coulomb	500	35	None	0

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	BREEZE OCEANSIDE APARTMENTS		
Analysis Description	SURFICIAL STABILITY ANALYSIS		
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Company	G.E.I.		
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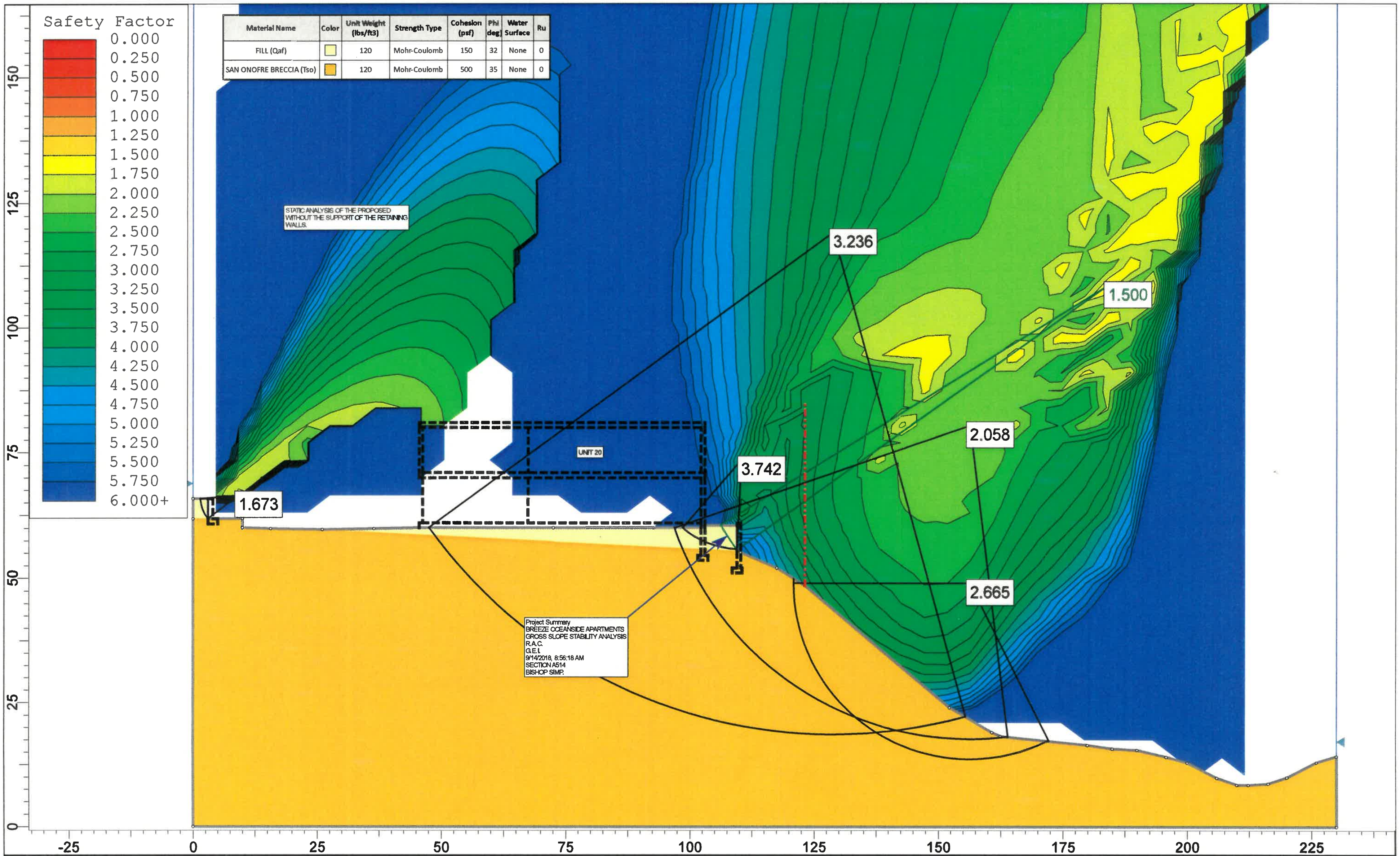

Geotechnical Exploration, Inc.	*Project*	BREEZE OCEANSIDE APARTMENTS	
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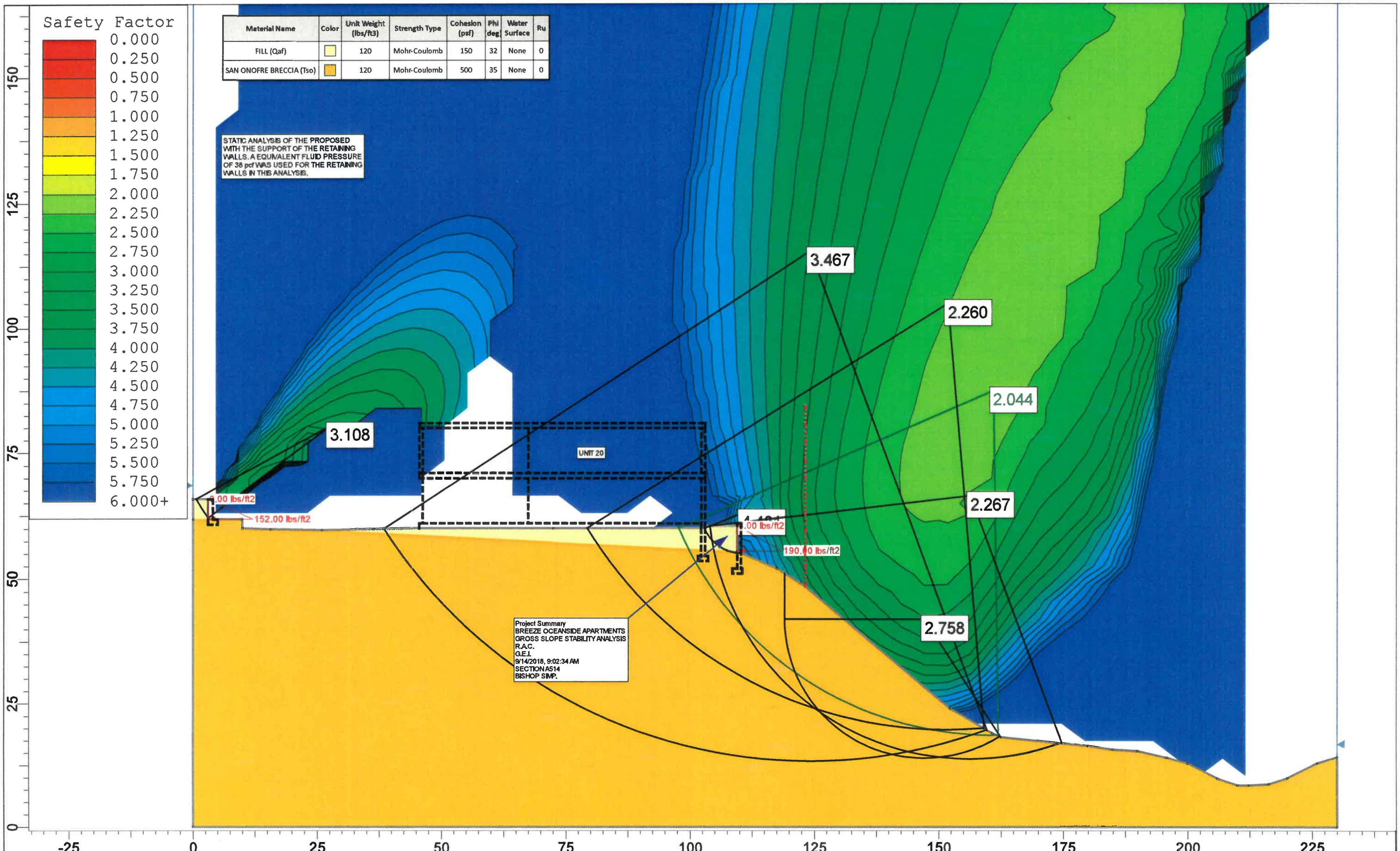

Geotechnical Exploration, Inc.

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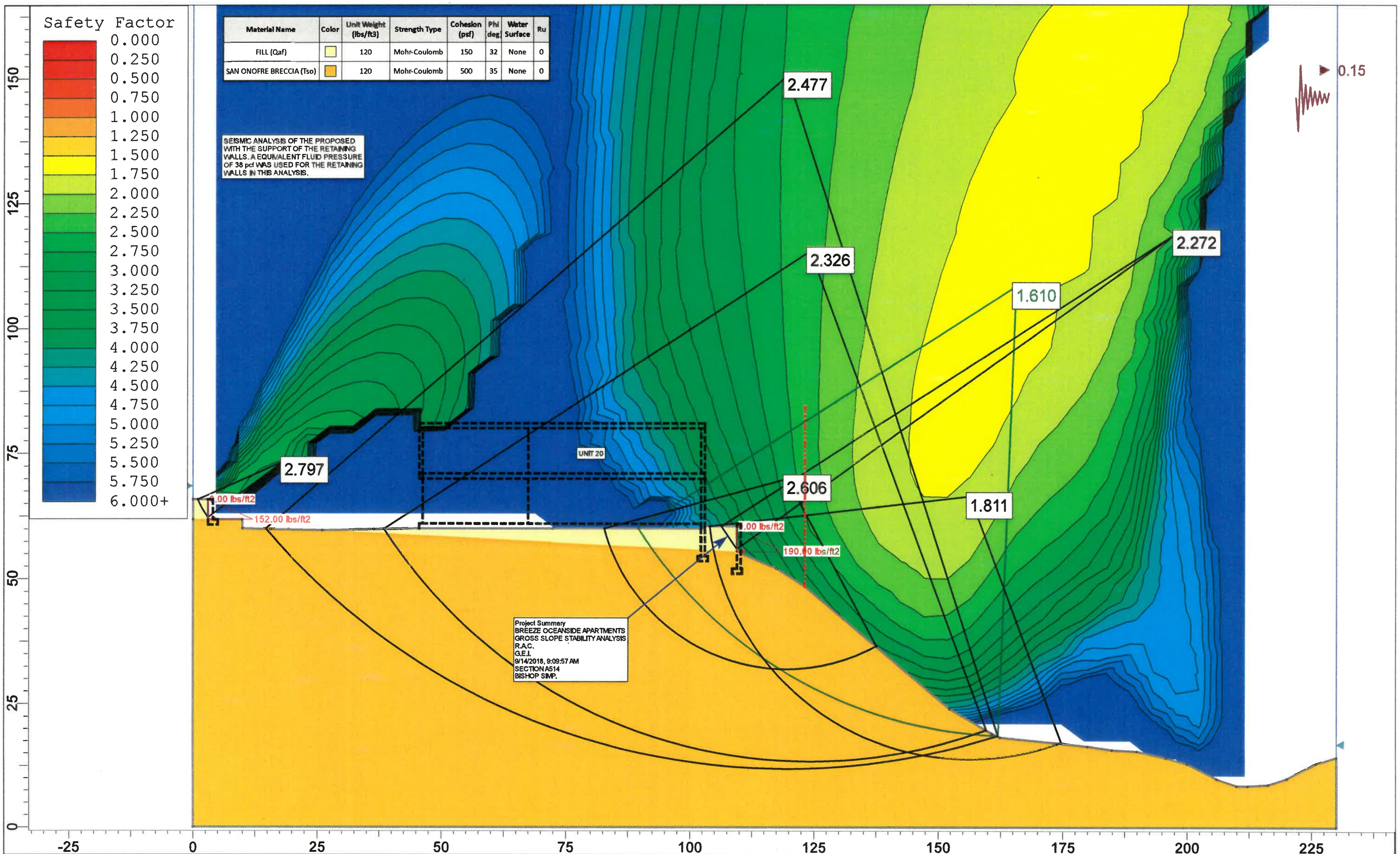
SLIDEINTERPRET 6.039



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SAN ONOFRE BRECCIA (Tso)		120	Mohr-Coulomb	500	35	None	0

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	BREEZE OCEANSIDE APARTMENTS		
Analysis Description	GROSS SLOPE STABILITY ANALYSIS		
Drawn By	R.A.C.	Scale	1:320
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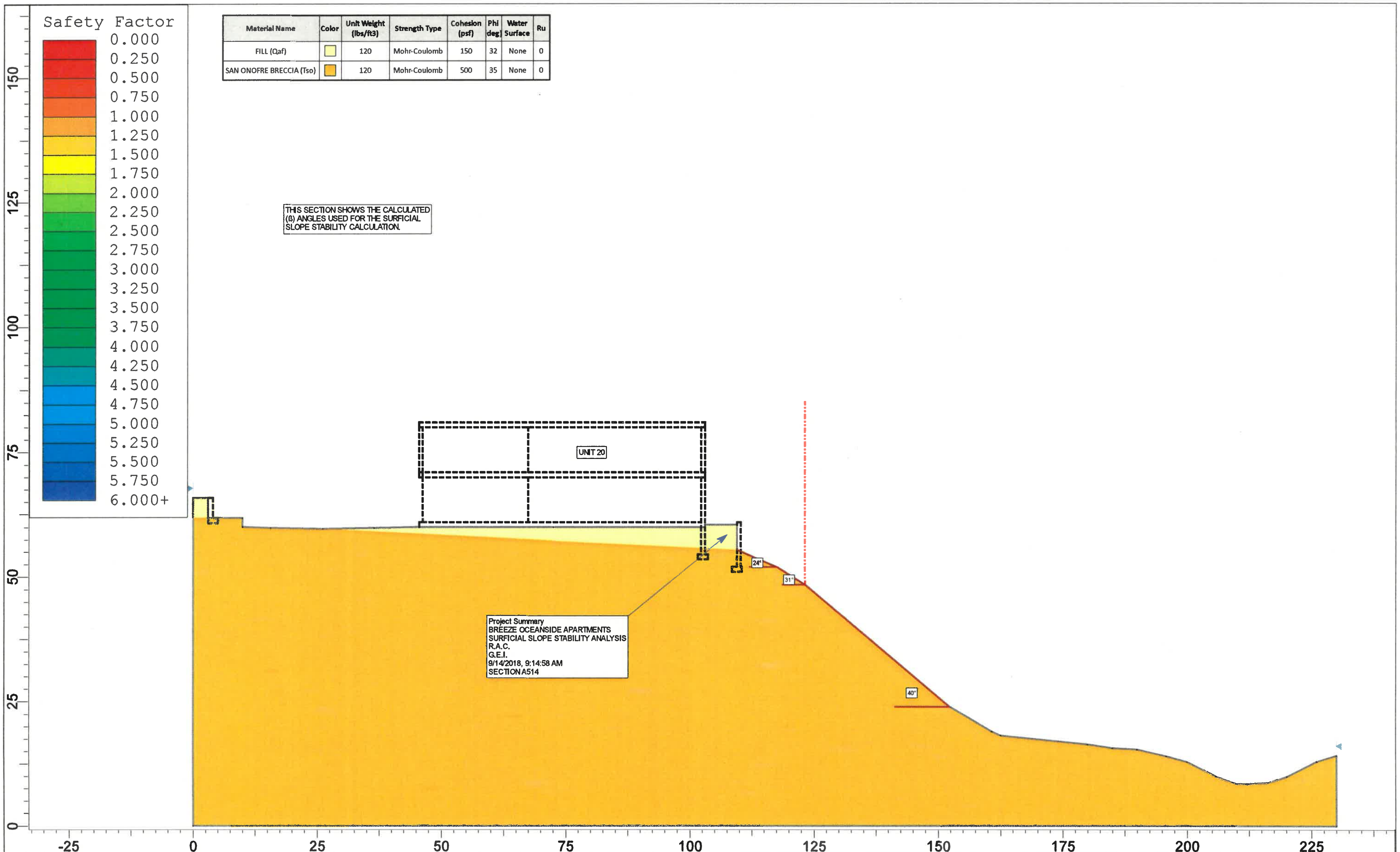
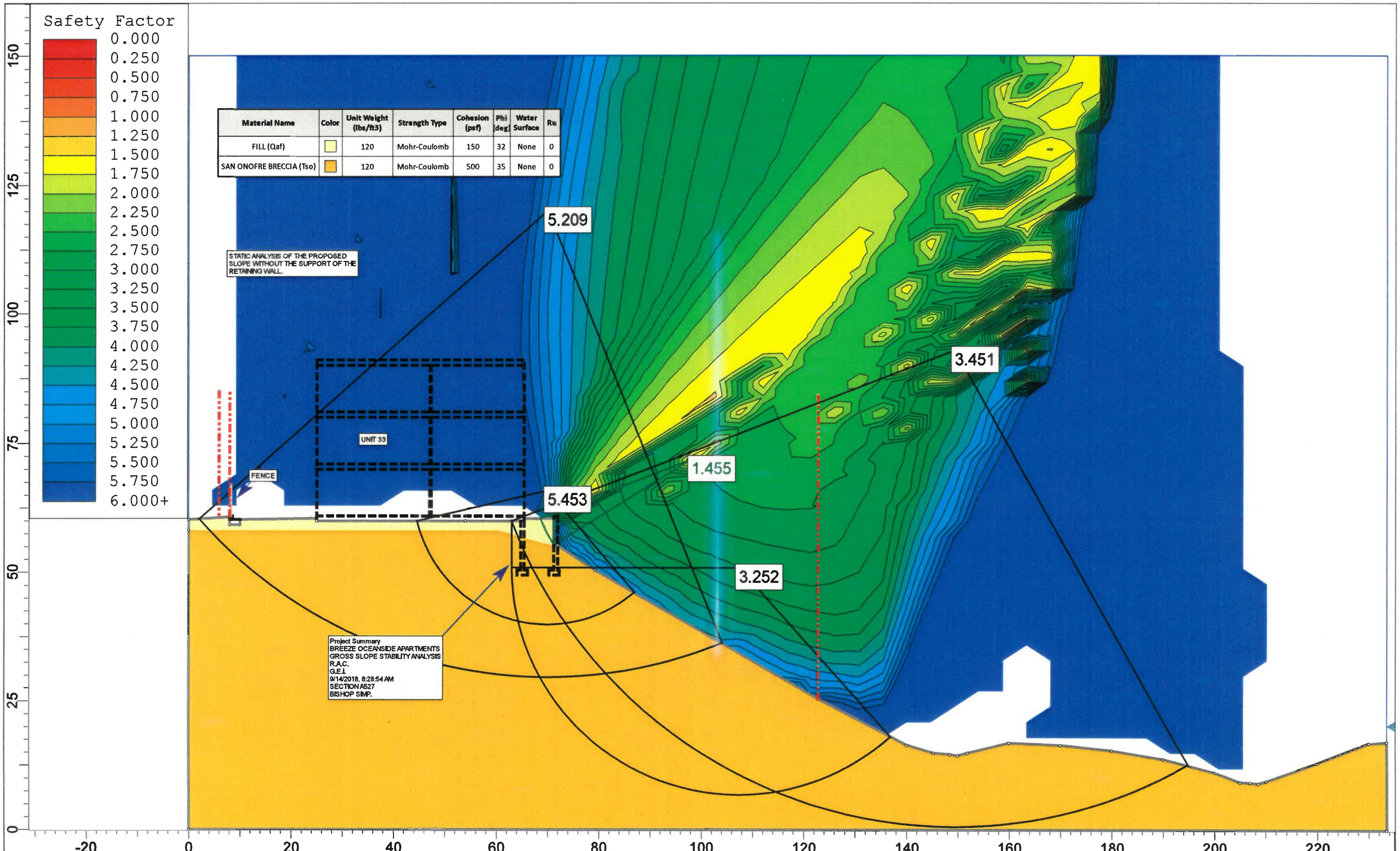


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi deg	Water Surface	Ru
FILL (Qaf)		120	Mohr-Coulomb	150	32	None	0
SAN ONOFRE BRECCIA (Tso)		120	Mohr-Coulomb	500	35	None	0

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

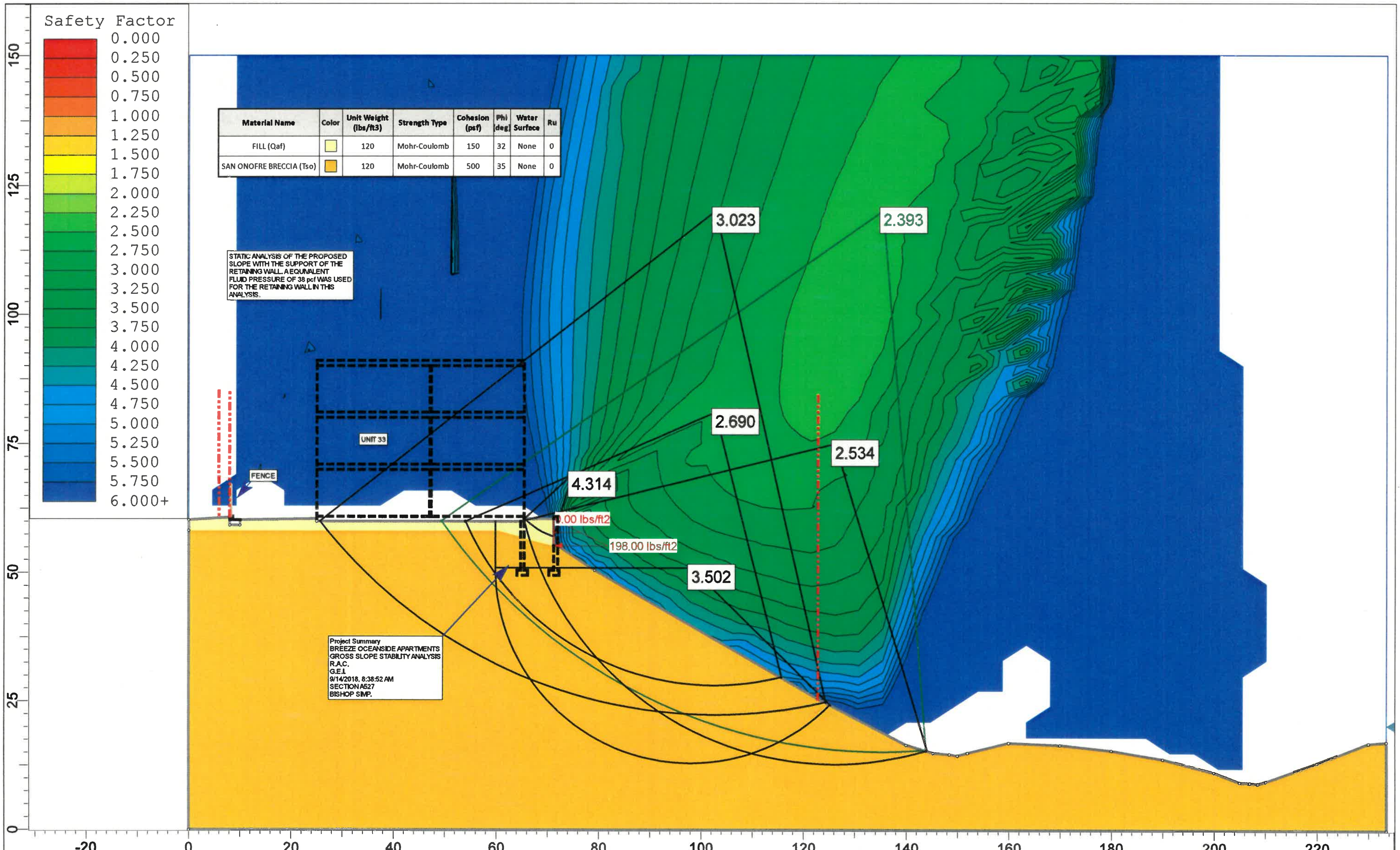
Project	BREEZE OCEANSIDE APARTMENTS
Analysis Description	SURFICIAL SLOPE STABILITY ANALYSIS
Drawn By	R.A.C.
Scale	1:320
Company	G.E.I.
Date	9/14/2018, 9:14:58 AM
File Name	JOB NO. 15-10805_S14_SURFICIAL.slim



Geotechnical Exploration, Inc.

Project	BREEZE OCEANSIDE APARTMENTS		
Analysis Description	GROSS SLOPE STABILITY ANALYSIS		
Drawn By R.A.C.	*Scale* 1:310	*Company*	G.E.I.
Date	9/14/2018, 8:28:54 AM	*File Name*	JOB NO. 15-10805_S27_01.slim

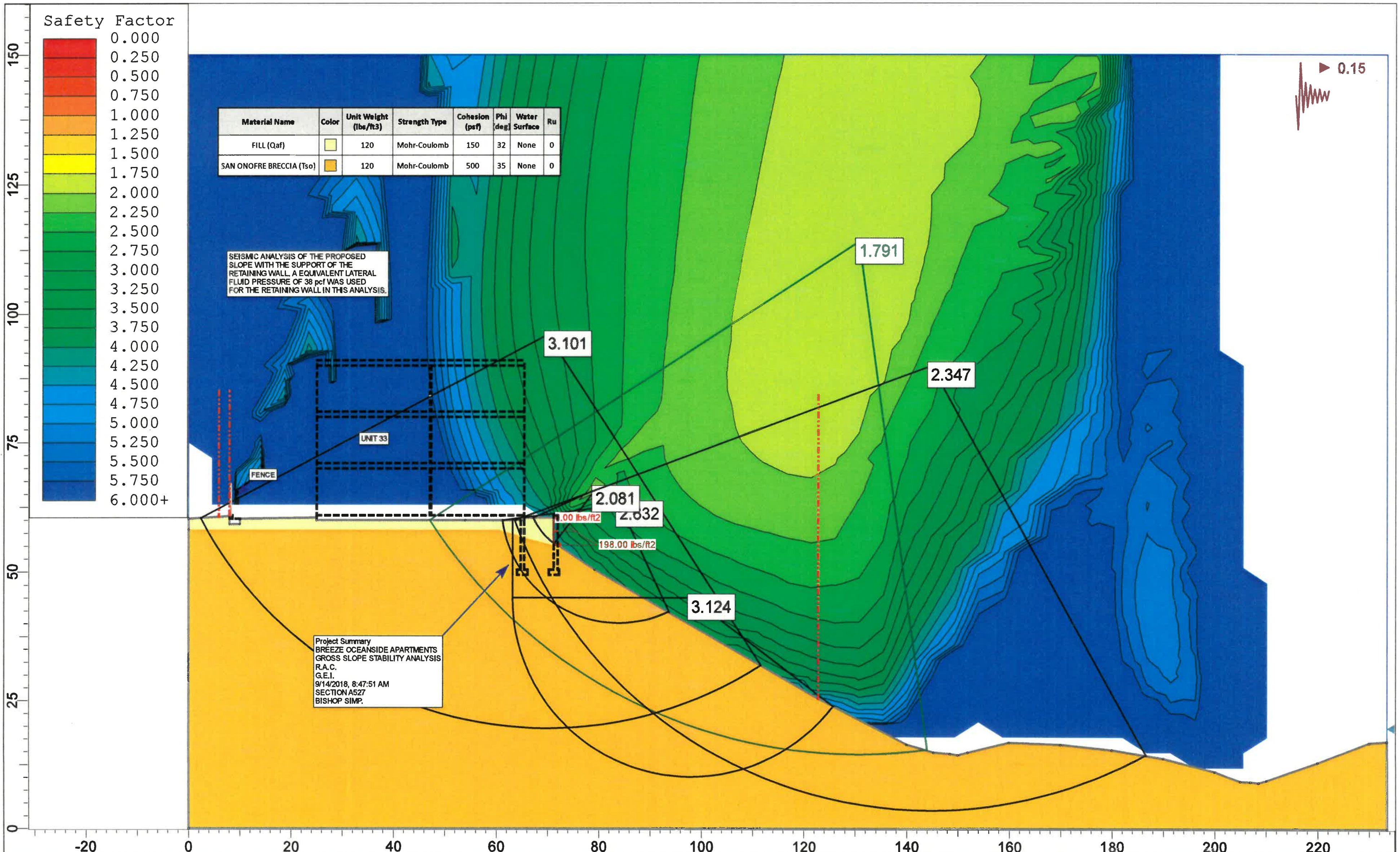
SLIDEINTERPRET 6.039



Geotechnical Exploration, Inc.

Project	BREEZE OCEANSIDE APARTMENTS		
Analysis Description	GROSS SLOPE STABILITY ANALYSIS		
Drawn By	R.A.C.	*Scale*	1:310
Company	G.E.I.		
Date	9/14/2018, 8:38:52 AM	*File Name*	JOB NO. 15-10805_S27_02.slim

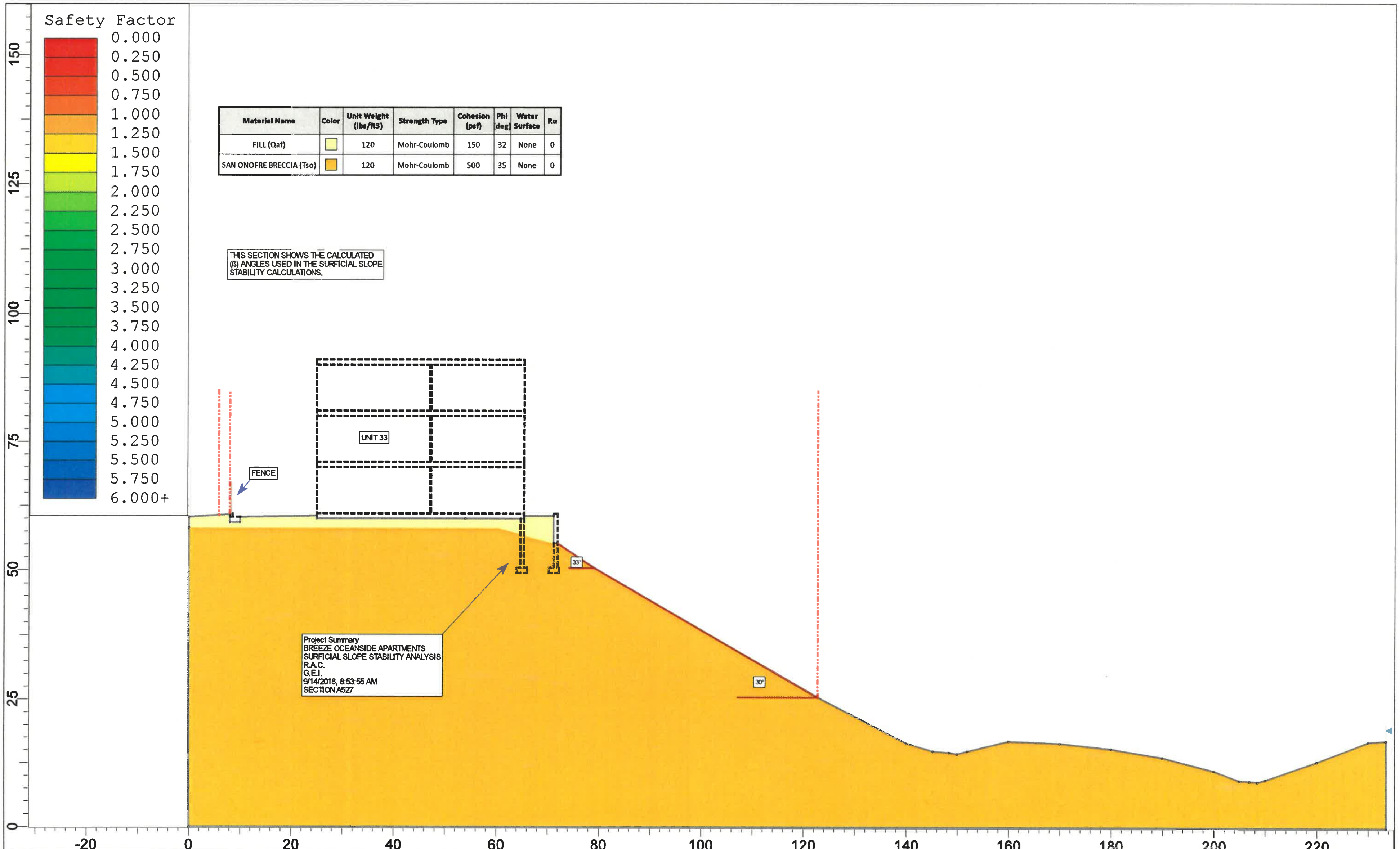
SLIDEINTERPRET 6.039



Geotechnical Exploration, Inc.

Project	BREEZE OCEANSIDE APARTMENTS		
Analysis Description	GROSS SLOPE STABILITY ANALYSIS		
Drawn By R.A.C.	*Scale* 1:310	*Company*	G.E.I.
Date	9/14/2018, 8:47:51 AM	*File Name*	JOB NO. 15-10805_S27_02w_0.15gSHAKE.slim

SLIDEINTERPRET 6.039

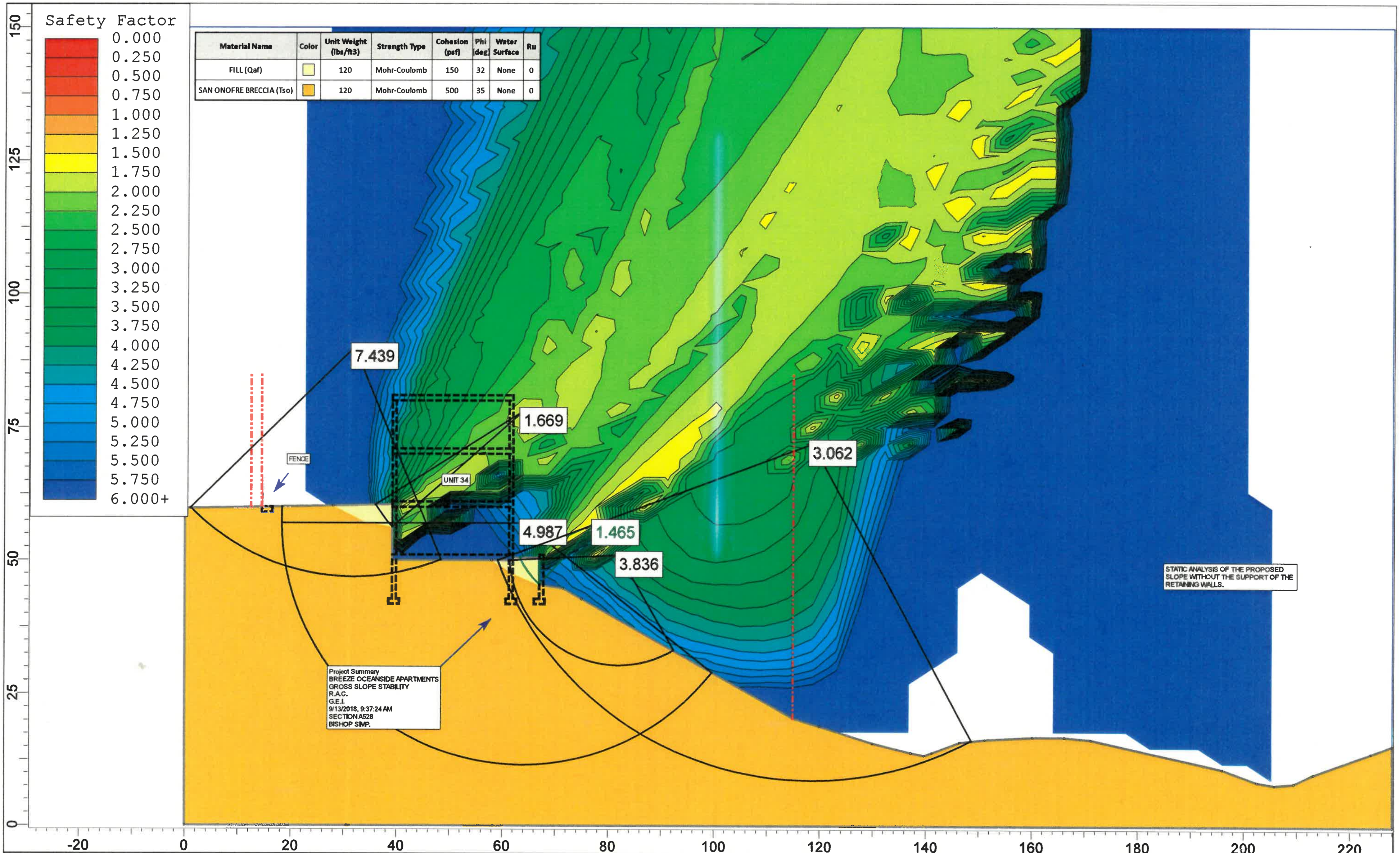


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
FILL (Qaf)		120	Mohr-Coulomb	150	32	None	0
SAN ONOFRE BRECCIA (Tso)		120	Mohr-Coulomb	500	35	None	0

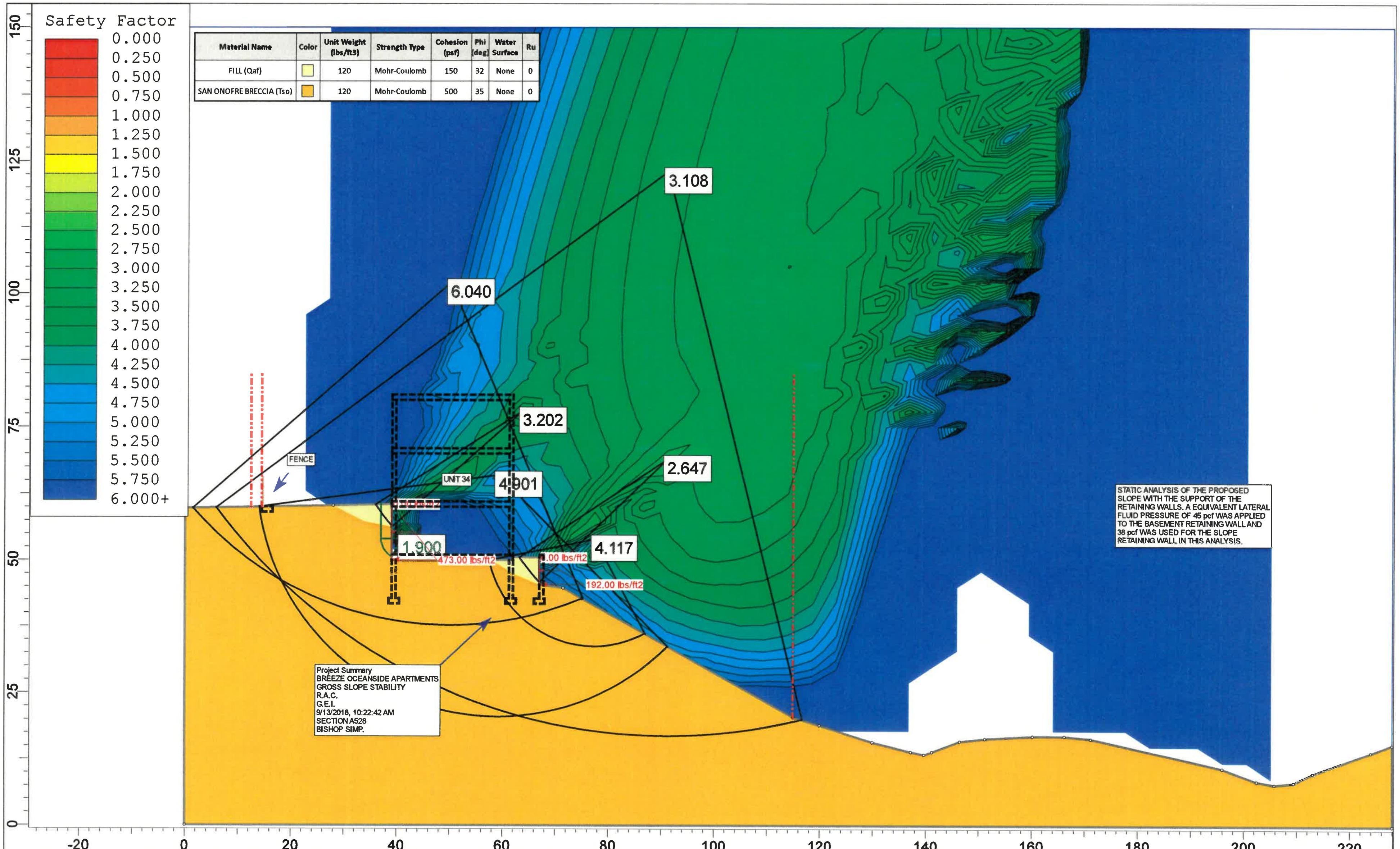
Geotechnical Exploration, Inc.

Project	BREEZE OCEANSIDE APARTMENTS		
Analysis Description	SURFICIAL SLOPE STABILITY ANALYSIS		
Drawn By	R.A.C.	Scale	1:310
Company	G.E.I.		
Date	9/14/2018, 8:53:55 AM	File Name	JOB NO. 15-10805_S27_SURFICIAL.slim

SLIDEINTERPRET 6.039



Geotechnical Exploration, Inc.	Project	BREEZE OCEANSIDE APARTMENTS	
	Analysis Description	GROSS SLOPE STABILITY	
	Drawn By: R.A.C.	Scale: 1:300	Company: G.E.I.
SLIDEINTERPRET 6.039	Date: 9/13/2018, 9:37:24 AM		File Name: JOB NO. 15-10805_S28_01.slim

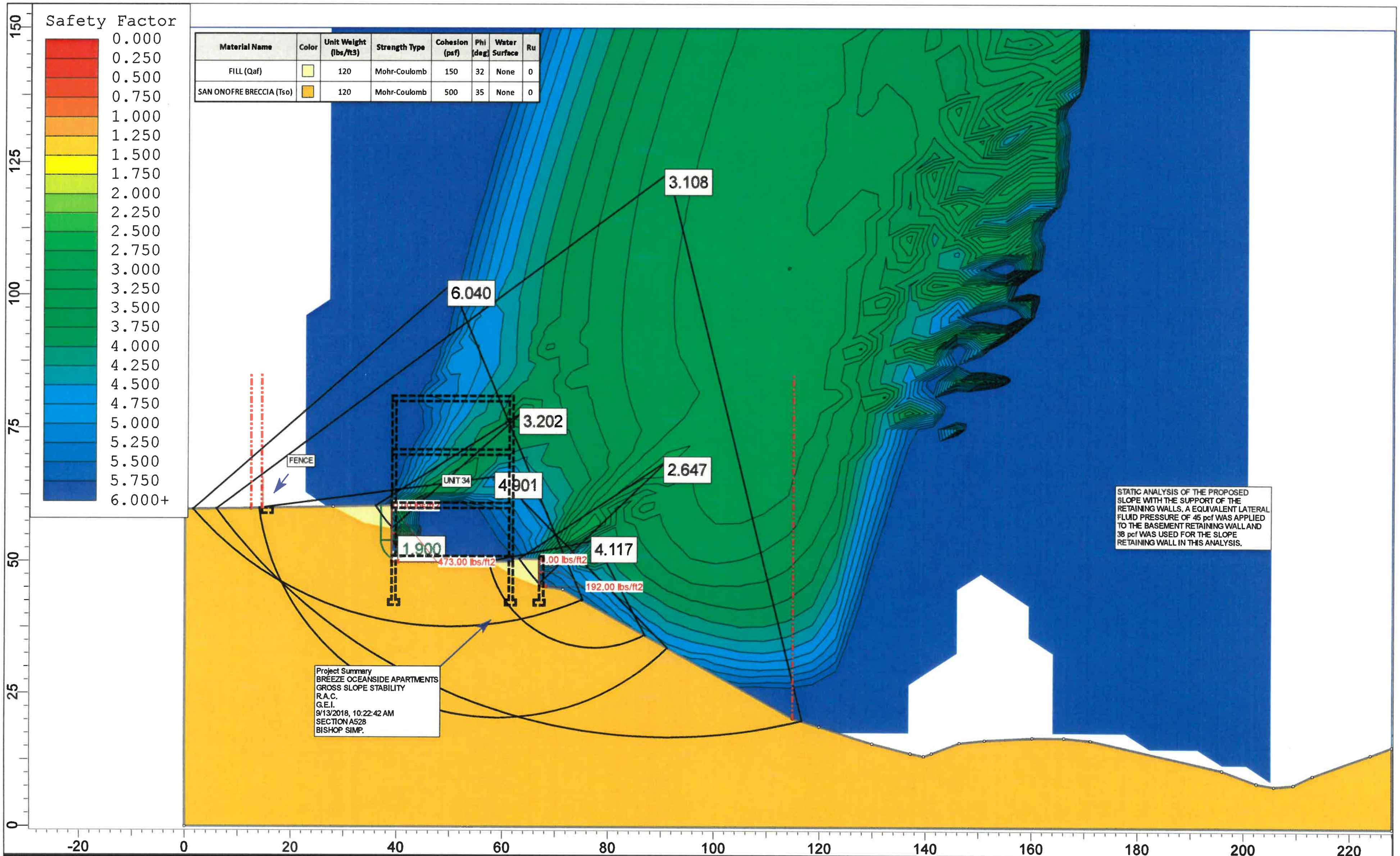


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
FILL (Qaf)		120	Mohr-Coulomb	150	32	None	0
SAN ONOFRE BRECCIA (Tso)		120	Mohr-Coulomb	500	35	None	0

Geotechnical Exploration, Inc.

Project	BREEZE OCEANSIDE APARTMENTS		
Analysis Description	GROSS SLOPE STABILITY		
Drawn By R.A.C.	*Scale* 1:300	*Company*	G.E.I.
Date	9/13/2018, 10:22:42 AM	*File Name*	JOB NO. 15-10805_S28_02.slim

SLIDEINTERPRET 6.039

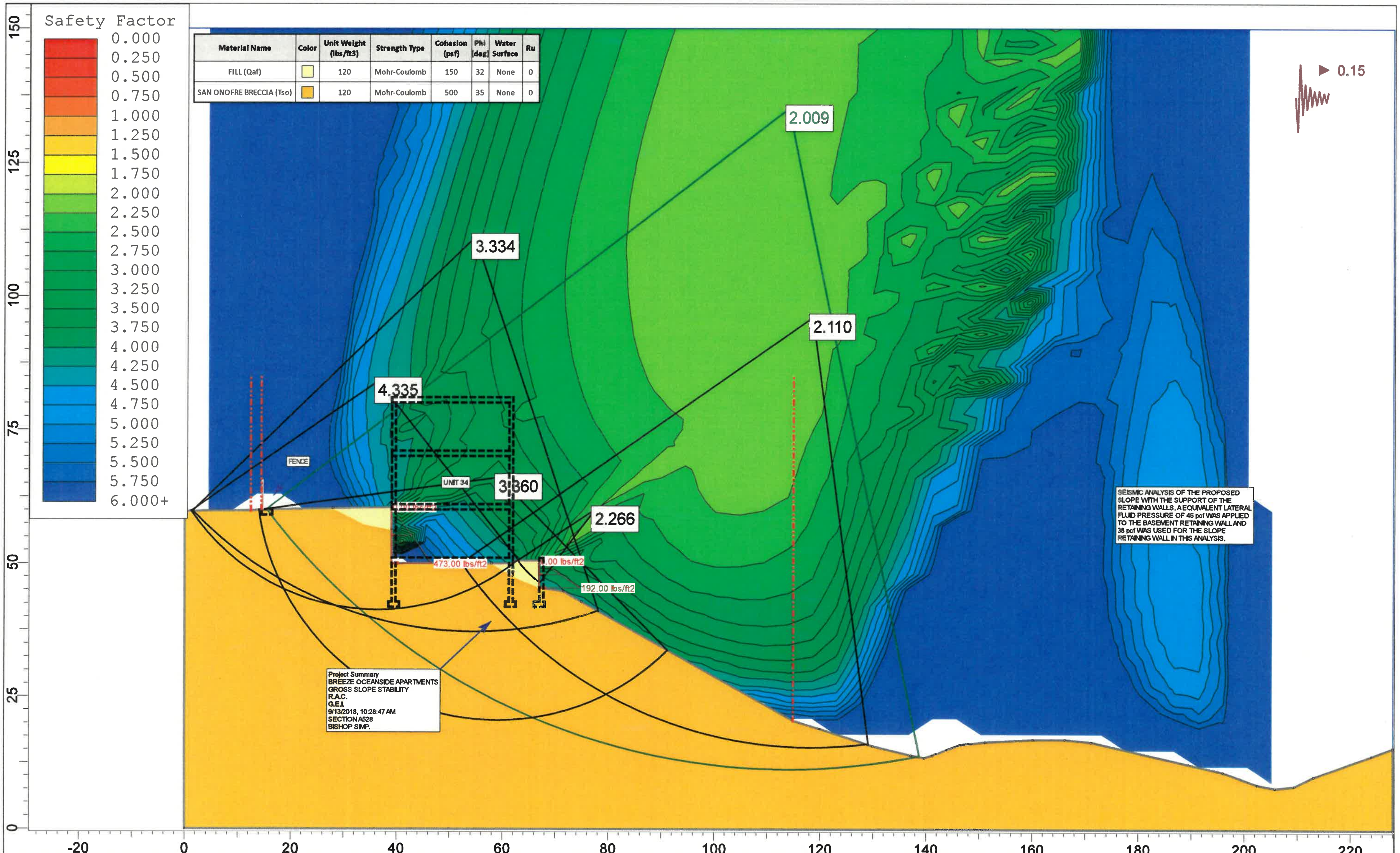


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi [deg]	Water Surface	Ru
FILL (Qaf)		120	Mohr-Coulomb	150	32	None	0
SAN ONOFRE BRECCIA (Tso)		120	Mohr-Coulomb	500	35	None	0

Geotechnical Exploration, Inc.

Project	BREEZE OCEANSIDE APARTMENTS
Analysis Description	GROSS SLOPE STABILITY
Drawn By	R.A.C.
Scale	1:300
Company	G.E.I.
Date	9/13/2018, 10:22:42 AM
File Name	JOB NO. 15-10805_S28_02.slim

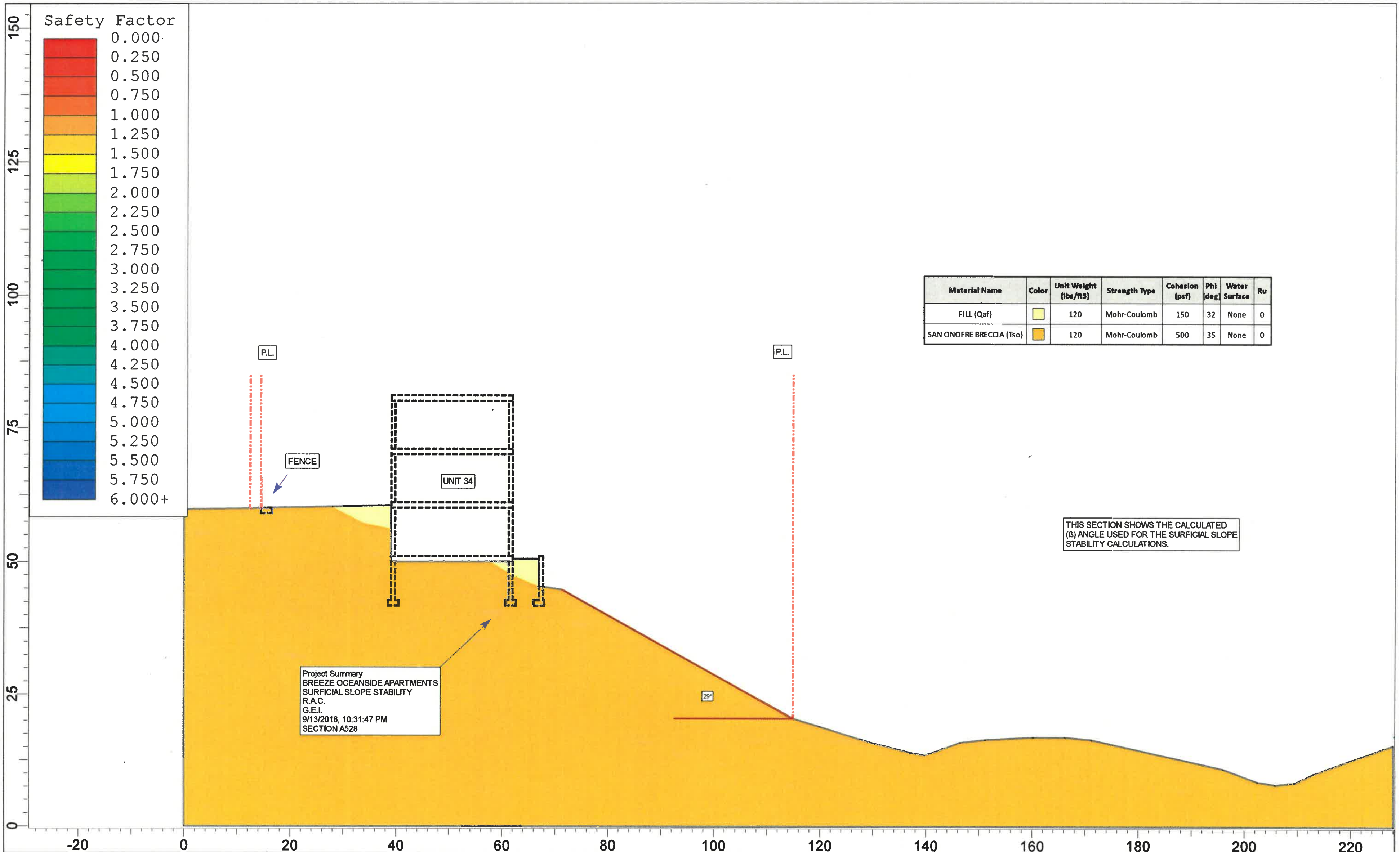
SLIDEINTERPRET 6.039



Geotechnical Exploration, Inc.

Project	BREEZE OCEANSIDE APARTMENTS		
Analysis Description	GROSS SLOPE STABILITY		
Drawn By R.A.C.	*Scale* 1:300	*Company*	G.E.I.
Date	9/13/2018, 10:28:47 AM	*File Name*	JOB NO. 15-10805_S28_02w_0.15gSHAKE.slim

SLIDEINTERPRET 6.039



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
FILL (Qaf)		120	Mohr-Coulomb	150	32	None	0
SAN ONOFRE BRECCIA (Tso)		120	Mohr-Coulomb	500	35	None	0

Géotechnical Exploration, Inc.

Project	BREEZE OCEANSIDE APARTMENTS
Analysis Description	SURFICIAL SLOPE STABILITY
Drawn By	R.A.C.
Scale	1:300
Company	G.E.I.
Date	9/13/2018, 10:31:47 PM
File Name	JOB NO. 15-10805_S28_SURFICIAL.slim

SLIDEINTERPRET 6.039

SURFICIAL FAILURE

γ_{sat}	γ_{water}	γ'	H
pcf	pcf	pcf	ft
130	62.4	67.6	3

EQUATION 1

$$F.S. = \left(\frac{C}{\gamma_{sat} \times H \times \cos(\beta) \times \sin(\beta)}\right) + \left(\frac{\gamma'}{\gamma_{sat}} * \frac{\tan(\varphi)}{\tan(\beta)}\right)$$

SURFICIAL SLOPE STABILITY ANALYSIS IS BASED ON EQUATION (1) FOR THE CALCULATED VALUES.

SECTION 1 (A501)				
SOIL TYPE	C (psf)	ϕ(°)	β(°)	F.S.
SAN ONOFRE BRECCIA (T_{so})	500	35	15	6.487
SAN ONOFRE BRECCIA (T_{so})	500	35	22	4.592
SAN ONOFRE BRECCIA (T_{so})	500	35	31	3.510
SAN ONOFRE BRECCIA (T_{so})	500	35	22	4.592

SECTION 2 (A502)				
SOIL TYPE	C (psf)	ϕ(°)	β(°)	F.S.
SAN ONOFRE BRECCIA (T_{so})	500	35	20	4.989
SAN ONOFRE BRECCIA (T_{so})	500	35	34	3.305

SECTION 5 (A505)				
SOIL TYPE	C (psf)	ϕ(°)	β(°)	F.S.
SAN ONOFRE BRECCIA (T_{so})	500	35	21	4.781

β	Slope inclination with respect to the horizontal plane
ϕ	Friction angle of the soil
C	Cohesion of the soil
γ_{sat}	Saturated unit weight of the soil
γ'	Submerged unit weight of the soil
H	Thickness of the saturated soil layer
F.S.	Factor of Safety

Factors of Safety **ABOVE** 1.5 are adequate.

γ_{sat}	γ_{water}	γ'	H
pcf	pcf	pcf	ft
130	62.4	67.6	3

SECTION 7 (A507)

SOIL TYPE	C (psf)	ϕ(°)	β(°)	F.S.
SAN ONOFRE BRECCIA (T_{so})	500	35	29	3.680
SAN ONOFRE BRECCIA (T_{so})	500	35	47	2.910

SECTION 14 (A514)

SOIL TYPE	C (psf)	ϕ(°)	β(°)	F.S.
SAN ONOFRE BRECCIA (T_{so})	500	35	22	4.592
SAN ONOFRE BRECCIA (T_{so})	500	35	31	3.510
SAN ONOFRE BRECCIA (T_{so})	500	35	40	3.038

SECTION 27 (A527)

SOIL TYPE	C (psf)	ϕ(°)	β(°)	F.S.
SAN ONOFRE BRECCIA (T_{so})	500	35	33	3.367
SAN ONOFRE BRECCIA (T_{so})	500	35	30	3.591

SECTION 28 (A528)

SOIL TYPE	C (psf)	ϕ(°)	β(°)	F.S.
SAN ONOFRE BRECCIA (T_{so})	500	35	29	3.680

β	Slope inclination with respect to the horizontal plane
ϕ	Friction angle of the soil
C	Cohesion of the soil
γ_{sat}	Saturated unit weight of the soil
γ'	Submerged unit weight of the soil
H	Thickness of the saturated soil layer
F.S.	Factor of Safety

Factors of Safety **ABOVE** 1.5 are adequate.