# REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION

Proposed Goldfinch Street Residence
4285 Goldfinch Street
San Diego, California

**JOB NO. 17-11683**
09 November 2018

Prepared for:

***TyCo Construction***

09 November 2017

TyCo Construction
3636 Fifth Avenue #101
San Diego, CA 92103
Attn: Mr. Ty Creamer

**Job No. 17-11683**

Subject: **Report of Preliminary Geotechnical Investigation**
Proposed Goldfinch Street Residence
4285 Goldfinch Street
San Diego, California

Dear Mr. Creamer:

In accordance with your request ***Geotechnical Exploration, Inc***. has performed a preliminary geotechnical investigation for the subject project in San Diego, California. The fieldwork was performed on November 8 and November 17, 2017.

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

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. 17-11683** will expedite a response to your inquiries.

Respectfully submitted,

**GEOTECHNICAL EXPLORATION, INC.**

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

## I. PROJECT SUMMARY AND SCOPE OF SERVICES

It is our understanding, based on communications with you and review of preliminary plans, that the existing undeveloped lot will be developed to receive a two-story-over-basement, single-family residential structure with an upper level garage, a driveway, decks and associated improvements. The proposed structure will be constructed of standard-type building materials utilizing foundation retaining walls and slab on-grade floors.

Based on review of preliminary plans provided to us, grading to achieve the desired elevations will require cuts up to approximately 15 feet deep for the foundation retaining walls. Final construction plans have not been provided to us during the preparation of this report. When completed, they should be made available for our review.

Based on the preceding, the scope of work performed for this investigation included a site reconnaissance and subsurface exploration program, laboratory testing, geotechnical engineering analysis of the field and laboratory data, and the preparation of this report. The data obtained and the analyses performed were for the purpose of providing design and construction criteria for the project earthwork, building foundations, slab on-grade floors, and retaining walls.

## II. SITE DESCRIPTION AND HISTORY

The property is known as Assessor's Parcel No. 444-272-09-00, Block 7, Lots 3 and 4 per Recorded Map 334, in the Mission Hills area of the City of San Diego, County of San Diego, State of California. Refer to the Vicinity Map, Figure No. I, for site location.

The undeveloped lot has a plan area of approximately 5,000 square feet. The "rectangular-shaped" lot is located on the east side of the undeveloped portion of Goldfinch Street, in the Mission Hills area of the City of San Diego. The property is bordered on the north by an existing single-family residence at a higher elevation; on the east by an existing single-family residence at a lower elevation; on the south by an undeveloped, natural, southeasterly descending hillside lower in elevation; and on the west by the undeveloped portion of Goldfinch Street slightly higher in elevation. Vegetation across the site consists primarily of weeds, shrubs and a few mature trees.

Elevations across the property range from approximately 189 feet above Mean Sea Level (MSL) at the southeast corner of the property to approximately 250 feet above MSL at the northwest corner of the property. Elevations were obtained from a topographic site plan prepared by Di Donato Associates dated May 1, 2018.

## III. FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program utilizing a limited access, continuous-flight auger drill rig and hand tools. Two exploratory borings and three exploratory handpits were excavated to depths of 3 to 13½ feet in the area of the proposed residence on November 8 and 17, 2017. The soils encountered in the exploratory excavations were continuously logged in the field by our geologist and described in accordance with the Unified Soil Classification System (refer to Appendix A). The approximate locations of the exploratory excavations are shown on Figure No. II.

Representative samples were obtained from the exploratory excavations at selected depths appropriate to the investigation. All samples were returned to our laboratory for evaluation and testing. Standard penetration resistance blow counts were obtained by driving a 2-inch O.D. split spoon sampler with a 140-pound hammer dropping through a 30-inch free fall. The sampler was driven a maximum of 18 inches and the number of blows for each 6-inch interval was recorded. The blows per foot indicated on the boring logs represent the accumulated number of blows that were required to drive the last 12 inches or portion thereof. Samples contained in liners were recovered by driving a 3.0-inch O.D. modified California sampler 18 inches into the soil using a 140-pound hammer.

Boring and handpit logs have been prepared on the basis of our observations and laboratory test results. Logs of the borings and handpits are attached as Figure Nos. IIIa-e. The following chart provides an in-house correlation between the number of blows and the relative density of the soil for the Standard Penetration Test and the 3-inch sampler.

| ***SOIL*** | ***DENSITY DESIGNATION*** | ***2-INCH O.D. SAMPLER BLOWS/FOOT*** | ***3-INCH O.D. SAMPLER BLOWS/FOOT*** |
|---|---|---|---|
| Sand and Nonplastic Silt | Very loose | 0-4 | 0-7 |
| | Loose | 5-10 | 8-20 |
| | Medium | 11-30 | 21-53 |
| | Dense | 31-50 | 54-98 |
| | Very Dense | Over 50 | Over 98 |

| SOIL | DENSITY DESIGNATION | 2-INCH O.D. SAMPLER BLOWS/FOOT | 3-INCH O.D. SAMPLER BLOWS/FOOT |
|---|---|---|---|
| Clay and Plastic Silt | Very soft | 0-2 | 0-2 |
| | Soft | 3-4 | 3-4 |
| | Firm | 5-8 | 5-9 |
| | Stiff | 9-15 | 10-18 |
| | Very stiff | 16-30 | 19-45 |
| | Hard | 31-60 | 46-90 |
| | Very Hard | Over 60 | Over 90 |

Based on the field and laboratory test data, our observations of the primary soil types, and our previous experience with laboratory testing of similar soils, our Geotechnical Engineer has assigned values for friction angle, coefficient of friction, and cohesion for those soils that will have significant lateral support or load bearing functions on the project. These values have been utilized in determining the recommended bearing value as well as active and passive earth pressure design criteria and slope stability calculations. A list of soil shear strength values used in our slope stability calculations is included in Appendix B.

## IV. LABORATORY TESTS

Laboratory tests were performed on relatively undisturbed and disturbed bulk soil samples encountered in order to evaluate their index, strength, expansion, and compressibility properties. The test results are presented on the excavation logs at the appropriate sample depths, Figure Nos. III and IV. The following tests were conducted on the sampled soils:

| | |
|---|---|
| *1.* | *Laboratory Compaction Characteristics (ASTM D1557-12)* |
| *2.* | *Determination of Percentage of Particles Passing No. 200 Sieve (ASTM D1140-14)* |
| *3.* | *Expansion Index (ASTM D4829-11)* |
| *4.* | *Direct Shear Test (ASTM D3080-11)* |

Laboratory compaction tests establish the laboratory maximum dry density and optimum moisture content of the tested soils and are also used to aid in evaluating the strength characteristics of the soils. The test results are shown on Figure No. IVa.

The particle size smaller than a No. 200 sieve analysis aids in classifying the tested soils in accordance with the Unified Soil Classification System and provides qualitative information related to engineering characteristics such as expansion potential, permeability, and shear strength.

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

Based on the test results, and our experience with similar materials in the San Diego area, the sampled existing clayey sand fill soils have a borderline "low- to- medium" expansion potential, with a maximum measured expansion index of 51.

One laboratory direct shear test was performed to aid in evaluating the strength properties of the on-site soils. The test was performed on a relatively undisturbed sample of the formational sandstone materials encountered. The specimens were run at in-situ moisture and density, unsaturated, and under drained conditions. The test results are shown on Figure No. IVb.

## V. REGIONAL GEOLOGIC DESCRIPTION

San Diego County has been divided into three major geomorphic provinces: the Coastal Plain, Peninsular Ranges and 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, 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 to the west. 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. The property is located in this physiographic province. 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 a depth of approximately 5 miles since the movement began in the early Miocene, 24 million years ago. The source of these sediments has been the local mountains as well as the ancestral and modern Colorado River (Demere, 1997).

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

During recent history, the San Diego County area has been relatively quiet seismically. No fault ruptures or major earthquakes have been experienced in historic time within the 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. A 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. The most recent widely felt earthquake in San Diego County occurred July 20, 2009. No significant damage was reported for the San Diego area.

On Sunday, April 4, 2010, a large earthquake occurred in Baja California, Mexico. It was widely felt throughout the southwest including southwestern Arizona and southern California. This M7.2 event, the Sierra El Mayor earthquake, occurred in northern Baja California approximately 40 miles south of the Mexico-USA border at shallow depth along the principal plate boundary between the North American and Pacific plates. According to the U. S. Geological Survey this is an area with a high level of historical seismicity, and it has recently also been seismically active, 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 20$^{th}$ century events (e.g., 1915 and 1934) in this region of northern Baja California.

This event's aftershock zone extends significantly to the northwest, overlapping with the portion of the fault system that is thought to have ruptured in 1892. 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.

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 historical earthquakes have occurred (about the last 210 years in California) are also considered to be active (Association of Engineering Geologists, 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).

## VI. SITE-SPECIFIC SOIL & GEOLOGIC DESCRIPTION

### *A. Stratigraphy*

Our field work, reconnaissance and review of the geologic map by Kennedy and Tan, 2008, "*Geologic Map of San Diego, 30'x60' Quadrangle, CA,*" indicate that the site is underlain by Tertiary-age Mission Valley formational materials (Tmv). During the course of our field investigation, these formational materials were encountered in all of our exploratory excavations. The encountered formational materials are, in general, overlain by approximately 1 to 6 feet of loose, young surficial soils consisting of artificial fill soils (Qaf) and/or slopewash (Qsw). Figure No. V presents a geologic map (Kennedy and Tan, 2008) of the general area of the site.

*Fill Soils (Qaf):* The northwestern area of the proposed residential structure is underlain by approximately 6 feet of fill soils as encountered in exploratory excavation B-1. The encountered fill soils consist of loose, fine-to-medium grained clayey sand. These relatively shallow, low-density surficial soils are generally dry to slightly moist, gray-brown, and are not considered suitable in their current condition to support loads from the proposed residential structure. Refer to Figure Nos. IIIa-e for details.

*Slopewash (Qsw):* The central and eastern portions of the proposed residential development are underlain by approximately 1 foot of slopewash materials as encountered in exploratory excavation locations B-2, HP-1 and HP-2. The encountered slopewash materials generally consist of soft/loose, fine-to-medium grained sandy clay/clayey sand. These relatively shallow, low-density surficial soils are dry, dark-brown, and are not considered suitable in their current condition to support loads from the proposed residential structure. Refer to Figure Nos. IIIa-e for details.

_Mission Valley (Tmv):_ The encountered formational materials consist of very dense, slightly moist, light grey, fine grained clayey sand and were encountered to the maximum depth of exploration in all exploratory excavations. Refer to Figure Nos. IIIa-e for details.

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

Although no obvious geologic structure was observed within our exploratory excavations of the Mission Valley (Tmv) formational materials, our review of the geologic map by Kennedy and Tan, 2008, *"Geologic Map of San Diego, 30'x60' Quadrangle, CA,"* indicates that the Mission Valley formation closest to the site shows a mapped bedding attitude of approximately N62°W with a dip of 2° to the southwest. The relatively gentle dip is considered favorable, with the dip direction perpendicular to the sloping hillside, and not out of slope.

The exploratory excavation logs and related information depict subsurface conditions only at the specific locations shown on the site plan and on the particular dates designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring and handpit locations. Also, the passage of time may result in changes in the subsurface conditions due to environmental changes.

## VII. GROUNDWATER

Free groundwater was not encountered in the exploratory excavations. It must be noted, however, that fluctuations in the level of groundwater may occur due to variations in ground surface topography, subsurface stratification, rainfall, and other possible factors that may not have been evident at the time of our field investigation.

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

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

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

## VIII. GEOLOGIC HAZARDS AND SEISMIC CONSIDERATIONS

Our review of some available published information including the City of San Diego Seismic Safety Study, Geologic Hazards and Faults Map Sheet No. 21, indicates that the site is located in a low risk geologic hazard area designated as Category 52.

Category 52 is defined as *"Other Level Areas, gently sloping to steep terrain, favorable geologic structure, low risk".* Based on the Geologic Map of San Diego and the City of San Diego Seismic Safety Study, Geologic Hazards Map Sheet No. 21, Figure Nos. V and VI, there are no faults mapped on the site.

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

### *A. __Local and Regional Faults__*

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

*__Rose Canyon Fault__*: The Rose Canyon Fault Zone (Mount Soledad and Rose Canyon Faults) is located approximately 2 miles southwest of the subject site. The Rose Canyon Fault 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 generating an M7.2 earthquake and is considered microseismically active, although no significant recent earthquakes are known to have occurred on the fault.

Investigative work on faults that are part of the Rose Canyon Fault Zone at the Police Administration and Technical Center in downtown San Diego, at the SDG&E facility in Rose Canyon, and within San Diego Bay and elsewhere within downtown San Diego, has encountered offsets in Holocene (geologically recent) sediments. These findings confirm Holocene displacement on the Rose Canyon Fault, which was designated an *"active"* fault in November 1991 (Hart, E.W. and W.A. Bryant, 2007, Fault-Rupture Hazard Zones in California, California Geological Survey Special Publication 42).

*Coronado Bank Fault*: The Coronado Bank Fault is located approximately 14 miles southwest of the site. Evidence for this fault is based upon geophysical data (acoustic profiles) and the general alignment of epicenters of recorded seismic activity (Greene, 1979). The Oceanside earthquake of M5.3 recorded July 13, 1986, is known to have been centered on the fault or within the Coronado Bank Fault Zone. Although this fault is considered active, due to the seismicity within the fault zone, it is significantly less active seismically than the Elsinore Fault (Hileman, 1973). It is postulated that the Coronado Bank Fault is capable of generating a M7.6 earthquake and is of great interest due to its close proximity to the greater San Diego metropolitan area.

*Newport-Inglewood Fault:* The Newport-Inglewood Fault Zone is located approximately 19 miles northwest of the site. A significant earthquake (M6.4) occurred along this fault on March 10, 1933. Since then no additional significant events have occurred. The fault is believed to have a slip rate of approximately 0.6 mm/yr with an unknown recurrence interval. This fault is believed capable of producing an earthquake of M6.0 to M7.4 (SCEC, 2004).

*Elsinore Fault*: The Elsinore Fault is located approximately 39 miles northeast of the site. The fault extends approximately 200 kilometers (125 miles) from the Mexican border to the northern end of the Santa Ana Mountains. The Elsinore Fault zone is a 1- to 4-mile-wide, northwest-southeast-trending zone of discontinuous and en echelon faults extending through portions of Orange, Riverside, San Diego, and Imperial Counties. Individual faults within the Elsinore Fault Zone range from less than 1 mile to 16 miles in length. The trend, length and geomorphic expression of the Elsinore Fault Zone identify it as being a part of the highly active San Andreas Fault system.

Like the other faults in the San Andreas system, the Elsinore Fault is a transverse fault showing predominantly right-lateral movement. According to Hart, et al. (1979), this movement averages less than 1 centimeter per year. Along most of its length, the Elsinore Fault Zone is marked by a bold topographic expression consisting of linearly aligned ridges, swales and hallows. Faulted Holocene alluvial deposits (believed to be less than 11,000 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 ranging from M6.8 to M7.1. Faulting evidence exposed 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 earthquakes of M6.0 to M7.0 (Rockwell, 1985).

<u>*San Jacinto Fault*</u>: The San Jacinto Fault is located 61 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 earthquakes (M6.0 to M7.0) having occurred on this fault zone between 1890 and 1986. Earthquakes on the San Jacinto Fault in 1899 and 1918 caused fatalities in the Riverside County area. Offset across this fault is predominantly right-lateral, similar to the San Andreas Fault, although some investigators have suggested that dip-slip motion contributes up to 10% of the net slip (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 April 8, 1968, occurred about 25 miles southeast of the July 7, 2010, M5.4 earthquake.

This M5.4 earthquake follows the 4th of April 2010, Easter Sunday, M7.2 earthquake, located about 125 miles to the south, well south of the US Mexico international border. A M4.9 earthquake occurred in the same area on June 12th at 8:08 pm (Pacific Time). Thus, this section of the San Jacinto fault remains active.

Seismologists are watching two major earthquake faults in southern California. The San Jacinto fault, the most active earthquake fault in southern California, extends for more than 100 miles from the international border into San Bernardino and Riverside, a major metropolitan area often called the Inland Empire. The Elsinore fault is more than 110 miles long, and extends into the Orange County and Los Angeles area as the Whittier fault. The Elsinore fault is capable of a major earthquake that would significantly affect the large metropolitan areas of southern California. The Elsinore fault has not hosted a major earthquake in more than 100 years. The occurrence of these earthquakes along the San Jacinto fault and continued aftershocks demonstrates that the earthquake activity in the region remains at an elevated level. The San Jacinto fault is known as the most active earthquake fault in southern California. Caltech and USGS seismologist continue to monitor the ongoing earthquake activity using the Caltech/USGS Southern California Seismic Network and a GPS network of more than 100 stations.

## B. Other Geologic Hazards

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

*Ground Shaking*: Structural damage caused by seismically induced ground shaking is a detrimental effect directly related to faulting and earthquake activity. Ground shaking is considered to be the greatest seismic hazard in San Diego County. The intensity of ground shaking is dependent 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 significant damage. It is our opinion that the most serious damage to the site would be caused by a large earthquake originating on a nearby strand of the Rose Canyon Fault Zone. Although the chance of such an event is remote, it could occur within the useful life of the structure.

*Landslides*: Based upon our geotechnical investigation, review of the geologic map (Kennedy and Tan, 2008), review of the referenced City of San Diego Seismic Safety Study -- Geologic Hazards Map Sheet 21 and USDA stereo-pair aerial photographs

(3-31-53, AXN-3M-215 and 216), there are no known or suspected ancient landslides located on the site.

*Liquefaction*: The liquefaction of saturated sands during earthquakes can be a major cause of damage to buildings. Liquefaction is the process by which soils are transformed into a viscous fluid that will flow as a liquid when unconfined. It occurs primarily in loose, saturated sands and silts when they are sufficiently shaken by an earthquake.

On this site, the risk of liquefaction of foundation materials due to seismic shaking is considered to be low due to the medium dense to dense nature of the natural-ground material and the lack of a shallow static groundwater surface under the site. In our opinion, the site does not have a potential for soil strength loss to occur due to a seismic event.

*Slope Stability*: Our slope stability analysis is based on geologic cross section A-A' (Figure No. VII), laboratory test results from retrieved soil samples collected during our exploratory excavation, field review of site conditions, and review of aerial photographs, pertinent documents and geologic maps, as well as our experience with similar formational units in this area of San Diego. We performed slope stability calculations on cross section A-A' using Bishop Simplified method and conventional equations for gross and shallow stability. Based on our slope stability analysis, a factor of safety (FS) less than 1.5 against shallow and gross slope failure does not exist at the analyzed geologic cross section across the property. In our professional opinion, the site will have a factor of safety of 1.5 or greater following the proposed construction. Refer to Appendix B for Slope Stability Analysis details.

*Geologic Hazards Summary*: It is our opinion, based upon a review of the available maps, our research and our site investigation, that the site is underlain by relatively stable formational materials and is suited for the proposed residential project and associated improvements provided the recommendations presented herein are implemented.

No significant geologic hazards are known to exist on the site that would prevent the proposed construction. Ground shaking from earthquakes on active southern California faults and active faults in northwestern Mexico is the greatest geologic hazard at the property.

In our explicit professional opinion, no *"active"* or *"potentially active"* faults underlie the project site.

## IX. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based on the field investigation conducted by our firm, our laboratory test results, our analysis of the field and laboratory data, and our experience with similar soils and formational materials.

From a geotechnical engineering standpoint, it is our opinion that the site is suitable for construction of the proposed single-family residential development provided the conclusions and recommendations presented in this report are incorporated into its design and construction.

Detailed earthwork and foundation recommendations are presented in the following paragraphs. The opinions, conclusions, and recommendations presented in this report are contingent upon ***Geotechnical Exploration, Inc***. being retained to review the final plans and specifications as they are developed and to observe the site earthwork and installation of foundations. Accordingly, we recommend that the following paragraph be included on the grading and foundation plans for the project.

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

## *A. Seismic Design Criteria*

1. *Seismic Design Criteria:* Site-specific seismic design criteria for the proposed structure are presented in the following table in accordance with Section 1613 of the 2016 CBC, which incorporates by reference ASCE 7-10 for seismic design. We have determined the mapped spectral acceleration values for the site, based on a latitude of 32.7554 degrees and longitude of -117.1713 degrees, utilizing a tool provided by the USGS, which provides a solution for ASCE 7-10 (Section 1613 of the 2016 CBC) utilizing digitized files for the Spectral Acceleration maps. Based on our past experience with similar conditions, we have assigned a Site Soil Classification of D. Refer to Appendix C.

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

| $S_s$ | $S_1$ | $F_a$ | $F_v$ | $S_{ms}$ | $S_{m1}$ | $S_{ds}$ | $S_{d1}$ |
|---|---|---|---|---|---|---|---|
| 1.212g | 0.467g | 1.015 | 1.533 | 1.230g | 0.716g | 0.820g | 0.478g |

## *B.* ***Site Preparation and Earthwork***

2. *Clearing and Stripping:* The site should be cleared of all obstructions, any utility lines to be abandoned and any miscellaneous debris that may be present at the time of construction. After clearing, the ground surface should be stripped of surface vegetation as well as associated root systems. Holes resulting from the removal of buried obstructions that extend below the proposed finished site grades should be cleared and backfilled with suitable material compacted to the requirements given under Recommendation No. 6, "Compaction." The cleared and stripped materials should be properly disposed of off-site.

3. *Excavation:* Based on the results of our exploratory borings and handpits, as well as our experience with similar materials, it is our opinion that the natural formational materials can be excavated utilizing ordinary heavy earthmoving equipment for the proposed basement excavations. Contractors should not, however, be relieved of making their own independent evaluation of the excavatability of the on-site materials prior to submitting their bids.

4. *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 8 inches, moisture conditioned to ***at least 3 percent*** above the laboratory optimum, and compacted to the requirements for structural fill. In the event that planned cuts expose any medium to highly expansive formational materials in the building area, they should be scarified and moisture conditioned to at least 5 percent over optimum moisture.

5. *Material for Fill:* All existing on-site soils with rock smaller than 3 inches, with an organic content of less than 3 percent by volume are, in general, suitable for use as fill. Imported fill material should be a low expansive (Expansion Index of 20 to 50 per ASTM D4829-11) granular soil with a plasticity index of 12 or less. In addition, imported fill materials should not contain rocks or lumps more than 3 inches in greatest dimension, not more than 15 percent larger than 2½ inches, and no more than 25 percent of the fill should be larger than ¼-inch.

   It should be noted that on-site materials containing gravel or cobbles, if encountered, will be difficult to compact with light equipment in trench and wall backfills. Accordingly, we recommend that on-site soils for trench and wall backfills be screened to eliminate particles greater than ¾-inch. All materials for use as fill should be approved by our representative prior to filling.

6. *Fill Compaction:* All structural fills should be compacted to a minimum degree of compaction of 90 percent at a moisture content ***at least 3 percent*** above the optimum based upon ASTM D1557. 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 the recommended moisture content by either: (1) aerating and drying the fill if it is too wet, or (2) watering the fill if it is too dry. Each lift should be thoroughly mixed before compaction to ensure a uniform distribution of moisture.

7. *Trench and Retaining/Basement Wall Backfill:* ***All*** utility trenches and retaining walls should be backfilled with properly compacted fill. Backfill material should be placed in lift thicknesses appropriate to the type of compaction equipment

utilized and compacted to a minimum degree of compaction of 90 percent by mechanical means. Any portion of the trench backfill within pavement sections should conform to the material and compaction requirements of the adjacent pavement section.

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

8. *Surface Drainage:* Positive surface gradients should be provided adjacent to the proposed new residential structure and roof gutters and downspouts should be installed so as to direct water away from foundations and slabs toward suitable discharge facilities. Ponding of surface water should not be allowed anywhere on the site. In planter areas adjacent to the proposed structure, a 5 percent gradient should slope away from the foundations for a distance of at least 5 feet.

## C. Slopes and Basement Back-Cuts

9. *Temporary Basement Cut Slopes:* Based on our subsurface investigation work, laboratory test results, and engineering analysis, temporary cut slopes in the formational sandstone materials should be safe against gross instability at the recommended maximum slope inclination. Some localized sloughing or raveling of the soils exposed on the slopes may occur in weakly or non-cemented formational materials.

Temporary excavations along the west side of the proposed structure can be made at a slope ratio of 0.5:1.0 (horizontal to vertical) for anticipated excavations up to 22 feet in height. Temporary excavations along the north and south sides can be made at 0.25:1.0 (horizontal to vertical) up to 15 feet in height. If space constraints prevent the implementation of these slope ratios, temporary shoring should be used. For temporary cuts in the northwest retaining wall area, any undocumented, loose fill soils should either be: (1) removed prior to excavating into the underlying formational material; or (2) laid back at a slope ratio of 1.0:1.0 (horizontal to vertical).

Since the stability of temporary construction slopes will depend largely on the contractor's activities and safety precautions (storage and equipment loadings near the tops of cut slopes, surface drainage provisions, etc.), it should be the contractor's responsibility to establish and maintain all temporary construction slopes at a safe inclination appropriate to the methods of operation.

Additionally, no soil stockpiles should be placed at the top of slopes within a horizontal distance equal to one-half the height of the excavation depth.

10. *New Permanent Slopes:* We recommend that any required permanent cut and fill slopes be constructed to an inclination no steeper than 2.0:1.0 (horizontal to vertical). The project plans and specifications should contain all necessary design features and construction requirements to prevent erosion of the on-site soils both during and after construction. Slopes and other exposed ground surfaces should be appropriately planted with a protective groundcover.

Fill slopes should be constructed to assure that the recommended minimum degree of compaction is attained out to the finished slope face. This may be accomplished by "backrolling" with a sheepsfoot roller or other suitable equipment as the fill is raised. Placement of fill near the tops of slopes should be carried out in such a manner as to assure that loose, uncompacted soils are not sloughed over the tops and allowed to accumulate on the slope face.

## *D. Design Parameters for Proposed Foundations*

11. *Footings:* We recommend that the proposed structure be supported on conventional, individual-spread and/or continuous footing foundations bearing on well-compacted fill soil and/or undisturbed formational material. Footings should be founded at least 18 inches below the lowest adjacent finished grade for areas supporting two stories, and 24 inches of embedment for footings supporting three stories.

    If the proposed footings are located closer than 8 feet inside the top or face of slopes, they should be deepened to 1½ feet below a line beginning at a point 8 feet horizontally inside the slopes and projected outward and downward, parallel to the face of the slope and into firm soils (see Figure No. VIII).

    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.

12. *Bearing Values*: At the recommended depth, footings on native, dense formational soil or properly compacted fill soil may be designed for allowable bearing pressures of 2,500 pounds per square foot (psf) for combined dead and live loads and 3,325 psf for all loads, including wind or seismic. The

footings should, however, have a minimum width of 12 inches. Additionally, the footings supporting 2-story structures should have a minimum width of 18 inches and the footings supporting 3-stories or greater should have a minimum width of 24 inches. An increase of 350 psf may be allowed for each additional foot in width and 580 psf for each additional foot in depth, not to exceed a total of 4,500 psf. For static loading, a one-third increment may still be allowed for seismic and/or wind loading analysis.

13. *General Criteria For All Footings:* Footings located adjacent to or on slope face should be extended sufficiently deep so as to provide at least 8 feet of horizontal cover between the slope face and outside edge of the footing at the footing bearing level. Footings located adjacent to utility trenches should have their bearing surfaces situated below an imaginary 1.5 horizontal to 1.0 vertical plane projected upward from the bottom edge of the adjacent utility trench.

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 two No. 5 top and two No. 5 bottom reinforcing bars be provided in the footings. A minimum clearance of 3 inches should be maintained between steel reinforcement and the bottom or sides of the footing. In order for us to document 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.*

14. *Lateral Loads:* Lateral load resistance for the structure supported on footing foundations may be developed in friction between the foundation bottoms and the supporting subgrade. An allowable friction coefficient of 0.35 is considered applicable. An additional allowable passive resistance equal to an equivalent fluid weight of 300 pcf acting against the foundations may be used in design provided the footings are poured neat against the adjacent undisturbed, dense 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 and any shear keys or the recommended slope setback, whichever is greater.

15. *Basement/Retaining Wall Design*: Any 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 the structure's restrained retaining walls (with level backfill) be designed for an equivalent fluid pressure of 64 pcf (for 2:1 sloping backfill, we recommend an equivalent fluid pressure of 88 pcf). The soil pressure should be considered from the upper finish grade corresponding to the influence of the analyzed retaining walls. Pertinent surcharge pressures applicable to each wall may be converted to uniform lateral soil pressure by multiplying the vertical load by a factor of 0.53.

For any exterior unrestrained retaining walls with level backfill, we recommend an equivalent fluid weight of 43 pcf and a conversion coefficient of 0.36 when considering a surcharge behind such walls.

For seismic design of unrestrained walls, we recommend that the seismic pressure increment be taken as a fluid pressure distribution utilizing an equivalent fluid weight of 15 pcf. For restrained walls the soil seismic increment may be waived.

The preceding design pressures assume that there is sufficient drainage behind the walls to prevent the build-up of hydrostatic pressures from surface water infiltration. We recommend that drainage be provided by a composite drainage material such as J-Drain 200/220 and J-Drain SWD or equivalent. No gravel or perforated pipe is used with the J-Drain system. The geodrain board material should terminate 12 inches below the finish surface where the surface is covered by slabs or 18 inches below the finish surface in landscape areas. Waterproofing for the walls should extend at least 6 inches above the ground surface. Refer to Figure No. IX for Retaining Wall Backdrain and Waterproofing Schematic.

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. The structural plans should indicate when the retaining wall backfill may be placed.

16. *Settlement:* Settlement under building loads is expected to be within tolerable limits for the proposed residence. 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 angular rotation should be less than 1/240.

## E. ***Concrete Slab on-grade Criteria***

17. *Minimum Floor Slab Thickness and Reinforcement:* Based on our experience, we have found that, for various reasons, floor slabs occasionally crack, causing brittle surfaces such as ceramic tiles to become damaged. 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.

    17.1 Interior floor slabs should be a minimum of 5 inches actual thickness and be reinforced with No. 4 bars on 24-inch centers, both ways, placed at midheight in the slab. Slab subgrade soil should be verified by a ***Geotechnical Exploration, Inc.*** representative to have the proper moisture content within 48 hours prior to placement of the vapor barrier and pouring of concrete.

    17.2 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.

18. *Concrete Isolation Joints:* We recommend the project Civil/Structural Engineer incorporate isolation joints and 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.

19. *<u>Slab Moisture Protection and Vapor Barrier Membrane:</u>* 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.

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 perms (grains/square foot/hour in 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. We recommend that the slab be poured directly on the vapor barrier, which is placed directly on the prepared subgrade soil.

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

19.4 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. *Exterior Slab Thickness and Reinforcement:* As a minimum for protection of on-site improvements, we recommend that all exterior pedestrian concrete slabs be 4½ inches thick, founded on properly compacted and tested fill, and contain No. 4 bars at 24-inch centers, both ways, at the center of the slab, and contain adequate isolation and control joints. 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.

For exterior slabs with the minimum shrinkage reinforcement, control joints should be placed at spaces no farther than 15 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.

## *F.* ***Concrete Pavement***

21. *Concrete Driveway:* We recommend that the new driveway, subject only to automobile and light truck traffic, be 5 inches thick and be supported directly on properly prepared on-site subgrade soils. The upper 12 inches of the subgrade below the driveway pavement should be compacted to a minimum degree of compaction of 95 percent just prior to paving. The concrete should conform to Section 201 of The Standard Specifications for Public Works Construction, 2015 Edition, for Class 560-C-3250.

    All undocumented fills or loose slopewash soils in proposed driveway areas should be removed down to dense formational soils and properly compacted prior to subgrade soil preparation. A representative from our firm should be present to verify areal extents and depths of removal prior to replacement and compaction of new fill soils.

    The driveway slab may be constructed without reinforcing steel provided saw-cut, weakened-plane joints are provided at about 12-foot centers, both ways, and at re-entrant corners. The driveway slabs should be saw cut as soon as practical but no more than 24 hours after the placement of the concrete. The depth of the joint should be one-quarter of the slab thickness and its width should not exceed 0.02-foot.

    It is our understanding that the proposed garage will be founded above the proposed residence. Therefore, recommendations for any garage concrete slabs should be given by the project's structural engineer.

## G. *General Recommendations*

22. *Project Start Up Notification:* In order to minimize any work delays during site development, this firm should be contacted 24 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.).

## X. GRADING NOTES

***Geotechnical Exploration, Inc.*** recommends that we be retained to verify the actual soil conditions revealed during site grading work and footing excavation to be as anticipated in this "*Report of Preliminary Geotechnical Investigation*" for the project. In addition, the compaction of any fill soils placed during site grading work must be observed and tested by the soil engineer. It is the responsibility of the grading contractor to comply with the requirements on the grading plans and the local grading ordinance. All retaining wall and trench backfill should be properly compacted. ***Geotechnical Exploration, Inc.*** will assume no liability for damage occurring due to improperly or uncompacted backfill placed without our observations and testing.

## XI. LIMITATIONS

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

The work performed and recommendations presented herein are the result of an investigation and analysis that meet the contemporary standard of care in our profession within the City 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 review the project plans once they are available to verify that our recommendations are adequately incorporated in them.

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

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

Once again, should any questions arise concerning this report, please feel free to contact the undersigned. Reference to our **Job No. 17-11683** will expedite a reply to your inquiries.

Respectfully submitted,

**GEOTECHNICAL EXPLORATION, INC.**

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**REFERENCES**
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November 2018


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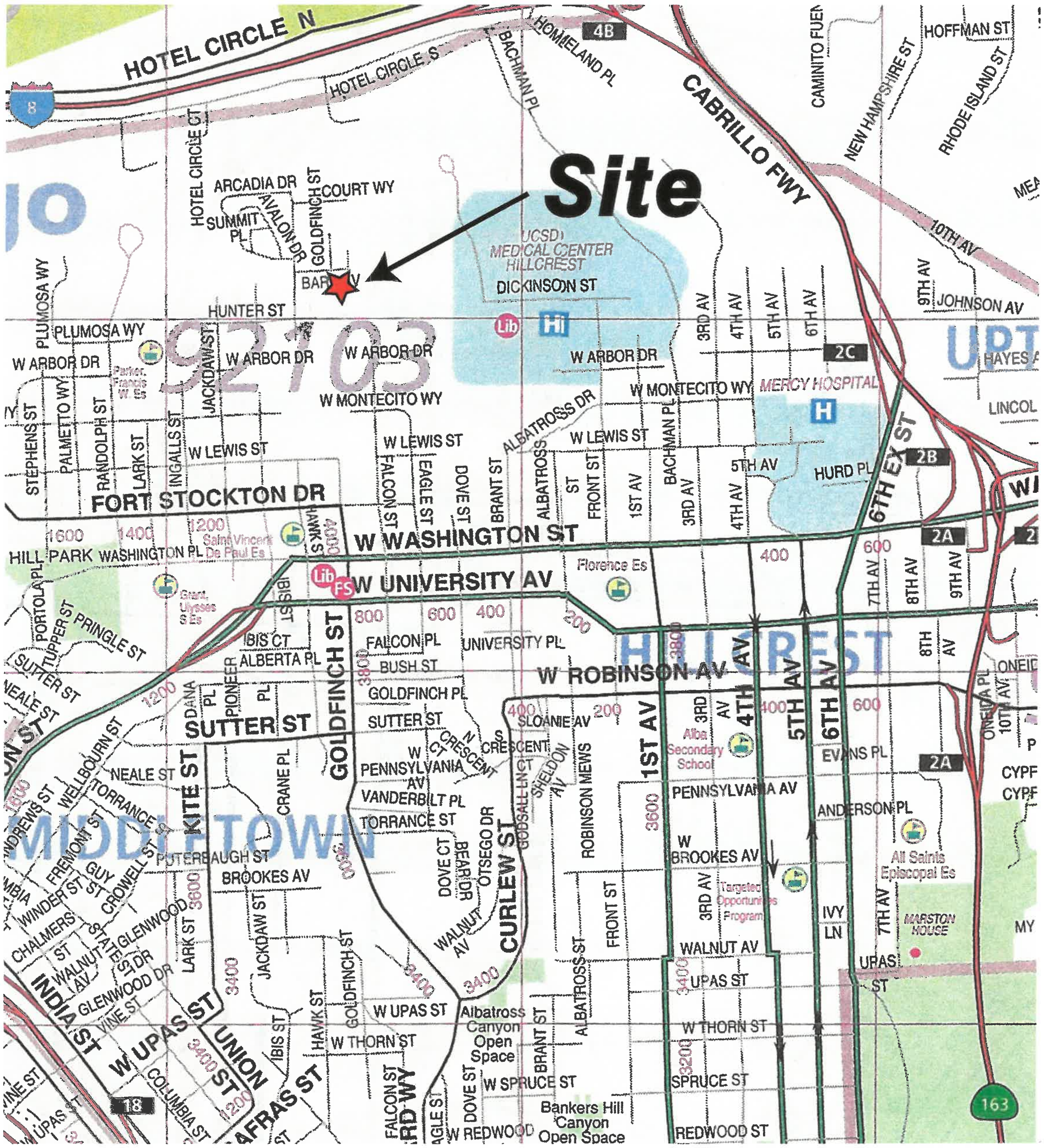
# VICINITY MAP

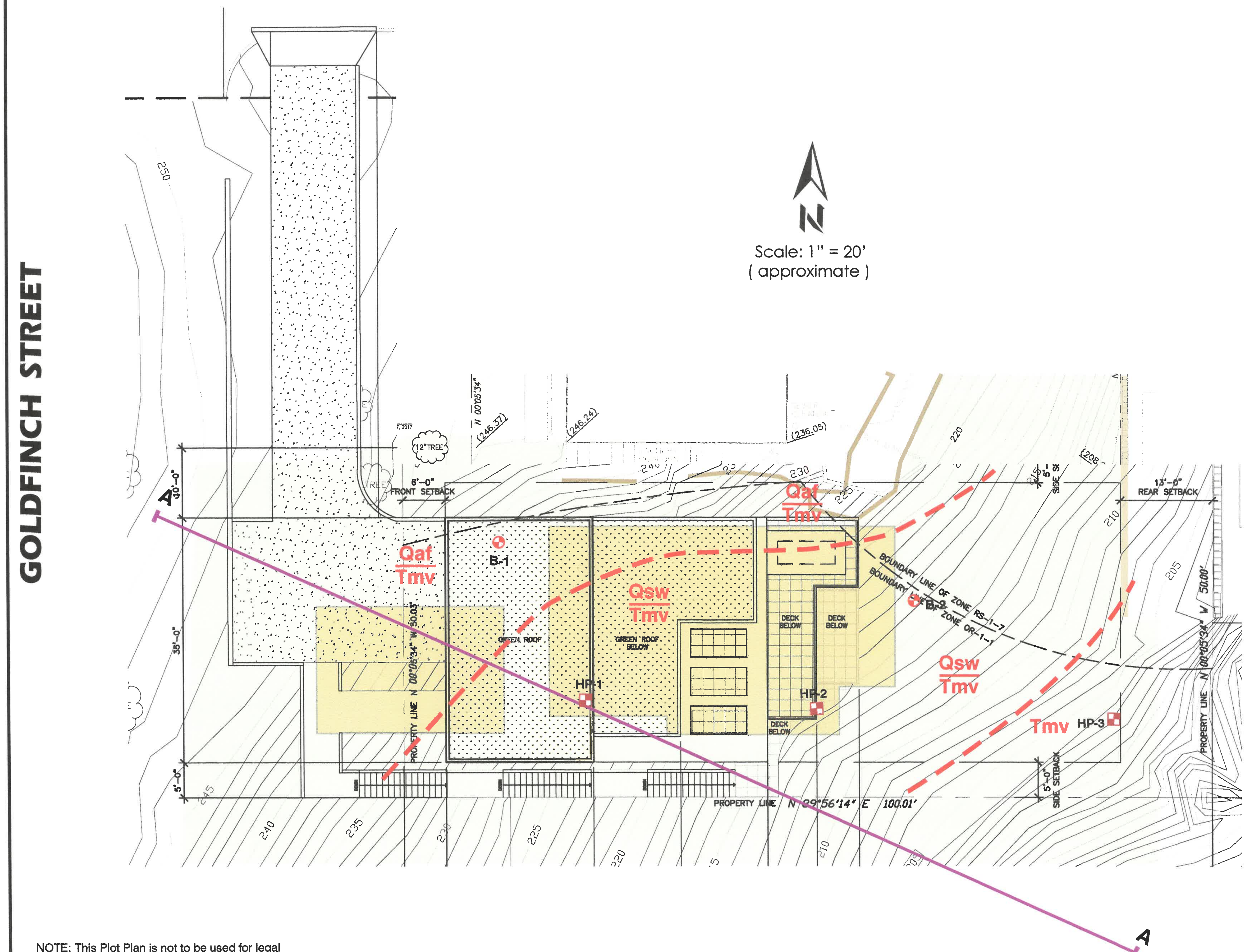



Thomas Bros Guide San Diego County pg 1268-J7

Goldfinch Street Residence
4285 Goldfinch Street
San Diego, CA.

*Figure No. I*
*Job No. 17-11683*

*REFERENCE: This Plot Plan was prepared from an existing "SURVEY WITH BUILDING FOOTPRINT" plan by DI DONATO ASSOCIATES dated November 27, 2017 and from on-site field reconnaissance performed by GEI.*




## LEGEND

| | |
|---|---|
| HP-3 | Approximate Location of Exploratory Handpit |
| B-2 | Approximate Location of Exploratory Boring |
| A — A' | Approximate Line of Cross Section |
| Qaf | Artificial Fill |
| Qsw | Slopewash |
| Tmv | Mission Valley Formation |
| - - - | Approximate Geologic Contact |

# PLOT PLAN AND SITE SPECIFIC GEOLOGIC MAP

*Goldfinch Street Residence*
*4285 Goldfinch Street*
*San Diego, CA.*
*Figure No. II*
*Job No. 17-11683*

**Geotechnical Exploration, Inc.**
*(November 2018)*

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.

17-11683-p3.ai

| EQUIPMENT | DIMENSION & TYPE OF EXCAVATION | DATE LOGGED |
|---|---|---|
| **Limited Access Auger Drill Rig** | **6-inch diameter Boring** | **11-8-17** |
| SURFACE ELEVATION | GROUNDWATER/ SEEPAGE DEPTH | LOGGED BY |
| **± 240' Mean Sea Level** | **Not Encountered** | **JAB** |

| 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 | | | **CLAYEY SAND** , fine- to medium-grained, trace roots and organic debris. Loose. Dry to slightly moist. Gray-brown. **FILL (Qaf)** | SC | | | | | | | | | |
| 2–3.5 | | Modified California Sample | | | | | | | | | | 9 | 3" |
| 3.5–5 | | Standard Penetration Test | | | | | | | | | | 7 | 2" |
| 5–6 | | Bulk Bag Sample | Bulk bag sample from 2'- 5'. -- 40% passing #200 sieve. | | | | 14.7 | 112.5 | | | 51 | | |
| 6.5 | | | **CLAYEY SAND** , fine- to medium-grained, some caliche. Very dense. Slightly moist. Light gray to white. **MISSION VALLEY FORMATION (Tmv)** | SC | | | | | | | | | |
| 7–8.5 | | Modified California Sample | | | | | | | | | | 92 | 3" |
| 8.5–10 | | Standard Penetration Test | | | | | | | | | | 56 | 2" |
| 10–12 | | Bulk Bag Sample | Bulk bag sample from 7'- 12'. | | | | | | | | | | |
| 12–13.5 | | Standard Penetration Test | | | | | | | | | | 81 | 2" |
| 13.5 | | | Bottom @ 13.5' | | | | | | | | | | |

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

| | | |
|---|---|---|
| JOB NAME: **Goldfinch Street Residence** | | |
| SITE LOCATION: **4285 Goldfinch Street, San Diego, CA** | | |
| JOB NUMBER: **17-11683** | REVIEWED BY: **JAB/WDH** | LOG No. **B-1** |
| FIGURE NUMBER: **IIIa** | Geotechnical Exploration, Inc. | |

EXPLORATION LOG 11683 GOLDFINCH.GPJ GEO EXPL.GDT 12/7/17

| EQUIPMENT | DIMENSION & TYPE OF EXCAVATION | DATE LOGGED |
|---|---|---|
| **Limited Access Auger Drill Rig** | **6-inch diameter Boring** | **11-8-17** |
| SURFACE ELEVATION | GROUNDWATER/ SEEPAGE DEPTH | LOGGED BY |
| **± 215' Mean Sea Level** | **Not Encountered** | **JAB** |

| 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 | | 1 | **SANDY CLAY/ CLAYEY SAND** , fine- to medium-grained sand, some formational fragments, trace roots and organc debris. Soft/ loose. Dry. Dark gray-brown. | CL/ SC | 7.8 | | | | | | | |
| 1–2 | | | **SLOPEWASH (Qsw)** -- 48% passing #200 sieve. | SC | | | | | | | | |
| 2–3 | | | **CLAYEY SAND** , fine-grained sand. Very dense. Slightly moist. Light gray. **MISSION VALLEY FORMATION (Tmv)** -- 45% passing #200 sieve. | | 13.1 | | | | | | 61 | 2" |
| 4 | | | Bulk bag sample from 2'- 7'. | | | | | | | | | |
| 7–8 | | | | | 13.7 | 101.0 | | | | | 90/ 9" | 3" |
| 8 | | | -- no sample recovery. | | | | | | | | 69/ 8" | 2" |
| 9 | | | Bottom @ 8.5' | | | | | | | | | |

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

| | | |
|---|---|---|
| JOB NAME **Goldfinch Street Residence** | | |
| SITE LOCATION **4285 Goldfinch Street, San Diego, CA** | | |
| JOB NUMBER **17-11683** | REVIEWED BY **JAB/WDH** | LOG No. **B-2** |
| FIGURE NUMBER **IIIb** | Geotechnical Exploration, Inc. | |

| EQUIPMENT | DIMENSION & TYPE OF EXCAVATION | DATE LOGGED |
|---|---|---|
| **Hand Tools** | **2' X 2' X 3' Handpit** | **11-17-17** |
| SURFACE ELEVATION | GROUNDWATER/ SEEPAGE DEPTH | LOGGED BY |
| **± 229' Mean Sea Level** | **Not Encountered** | **JAB** |

| 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 | | | **SANDY CLAY/ CLAYEY SAND** , fine- to medium-grained sand, some organics. Loose/soft. Dry. Dark brown. **SLOPEWASH (Qsw)** | CL/ SC | | | | | | | | |
| 1–3 | | | **SANDSTONE/ CLAYEY SAND** , fine-grained sand, abundant subhorizontal to subvertical fractures infilled with caliche; severely weathered to 2 feet, then moderately weathered. Very dense. Slightly moist. Light gray. **MISSION VALLEY FORMATION (Tmv)** | SC | | | | | | | | |
| 3 | | | Bottom @ 3' | | | | | | | | | |
| 4 | | | | | | | | | | | | |
| 5 | | | | | | | | | | | | |

EXPLORATION LOG 11683 GOLDFINCH.GPJ GEO_EXPL.GDT 12/7/17

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

| JOB NAME | | |
|---|---|---|
| **Goldfinch Street Residence** | | |
| SITE LOCATION | | |
| **4285 Goldfinch Street, San Diego, CA** | | |
| JOB NUMBER **17-11683** | REVIEWED BY **JAB/WDH** | LOG No. **HP-1** |
| FIGURE NUMBER **IIIc** | Geotechnical Exploration, Inc. | |

| EQUIPMENT | DIMENSION & TYPE OF EXCAVATION | DATE LOGGED |
|---|---|---|
| Hand Tools | 2' X 2' X 3' Handpit | 11-17-17 |
| SURFACE ELEVATION | GROUNDWATER/ SEEPAGE DEPTH | LOGGED BY |
| ± 213' Mean Sea Level | Not Encountered | JAB |

| 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 | | | **SANDY CLAY/ CLAYEY SAND** , fine- to medium-grained sand, some rootlets. Loose/soft. Dry. Dark gray-brown.<br><br>**SLOPEWASH (Qsw)** | CL/ SC | | | | | | | | |
| 1–3 | | | **SANDSTONE/ CLAYEY SAND** , fine-grained sand, abundant subhorizontal to subvertical fractures infilled with caliche; severely weathered to 2 feet, then moderately weathered. Very dense. Slightly moist. Light gray.<br><br>**MISSION VALLEY FORMATION (Tmv)** | SC | | | | | | | | |
| 3–6 | | | Bottom @ 3' | | | | | | | | | |

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

| JOB NAME | | |
|---|---|---|
| Goldfinch Street Residence | | |
| SITE LOCATION | | |
| 4285 Goldfinch Street, San Diego, CA | | |
| JOB NUMBER 17-11683 | REVIEWED BY JAB/WDH | LOG No. |
| FIGURE NUMBER IIId | Geotechnical Exploration, Inc. | HP-2 |

| EQUIPMENT | DIMENSION & TYPE OF EXCAVATION | DATE LOGGED |
|---|---|---|
| **Hand Tools** | **2' X 2' X 4' Handpit** | **11-17-17** |
| SURFACE ELEVATION | GROUNDWATER/ SEEPAGE DEPTH | LOGGED BY |
| **± 196' Mean Sea Level** | **Not Encountered** | **JAB** |

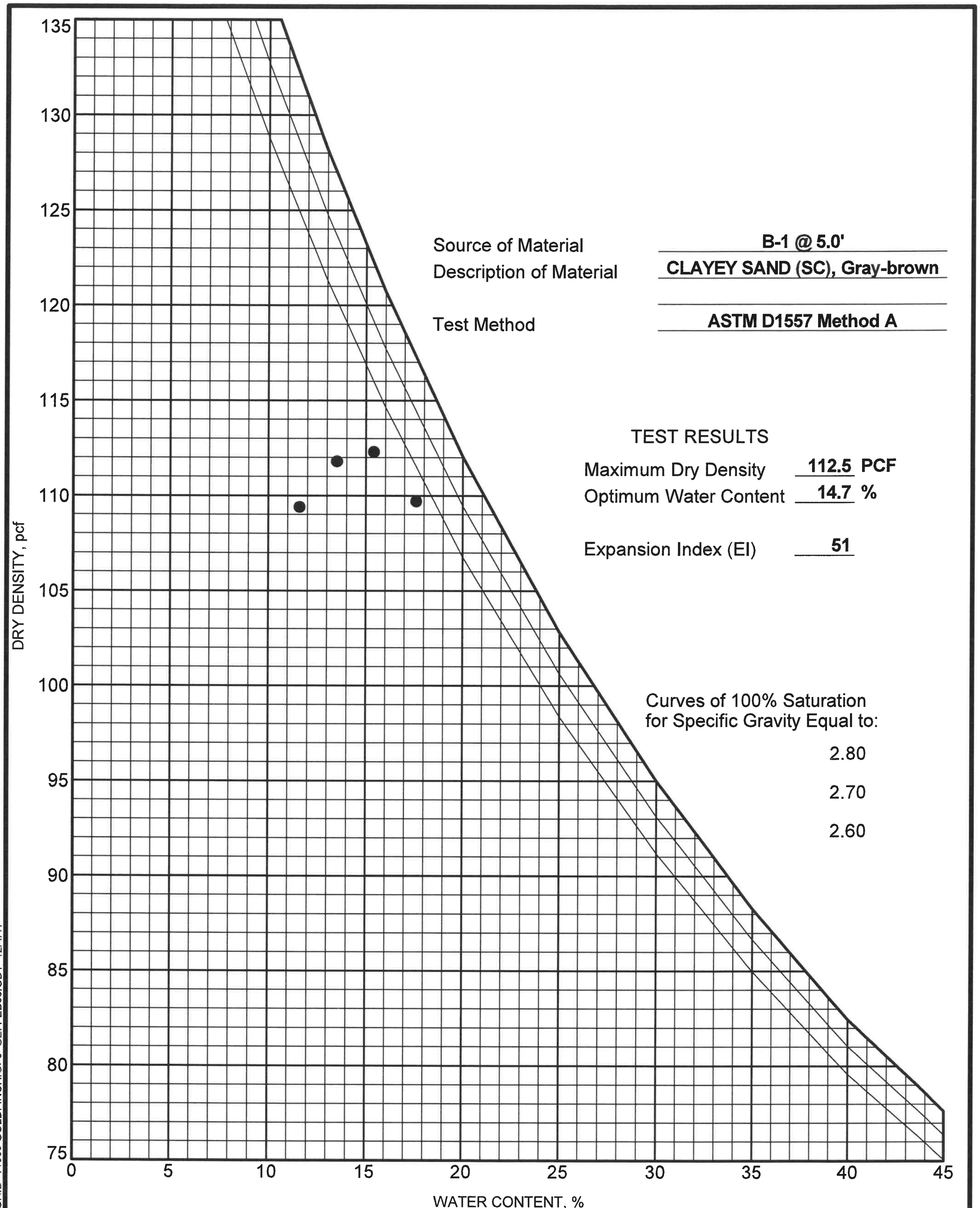
| DEPTH (feet) | SYMBOL | SAMPLE | FIELD DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color) | U.S.C.S. | IN-PLACE MOISTURE (%) | IN-PLACE DRY DENSITY (pcf) | OPTIMUM MOISTURE (%) | MAXIMUM DRY DENSITY (pcf) | DENSITY (% of M.D.D.) | EXPAN. + CONSOL. - (%) | BLOW COUNTS/FT. | SAMPLE O.D. (INCHES) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–4 | | | **SANDSTONE/ CLAYEY SAND** , fine-grained sand, friable to 3 feet, abundant subhorizontal to subvertical fractures infilled with caliche; severely weathered to 2 feet, then moderately weathered. Dense to very dense. Slightly moist. Light gray.<br>**MISSION VALLEY FORMATION (Tmv)** | SC | | | | | | | | |
| 4 | | | Bottom @ 4'<br>* HP-3 located at SE corner of property on exposed cut slope at property line. | | | | | | | | | |

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 | **Goldfinch Street Residence** |
| SITE LOCATION | **4285 Goldfinch Street, San Diego, CA** |
| JOB NUMBER | **17-11683** |
| REVIEWED BY | **JAB/WDH** |
| FIGURE NUMBER | **IIIe** |
| LOG No. | **HP-3** |

Geotechnical Exploration, Inc.

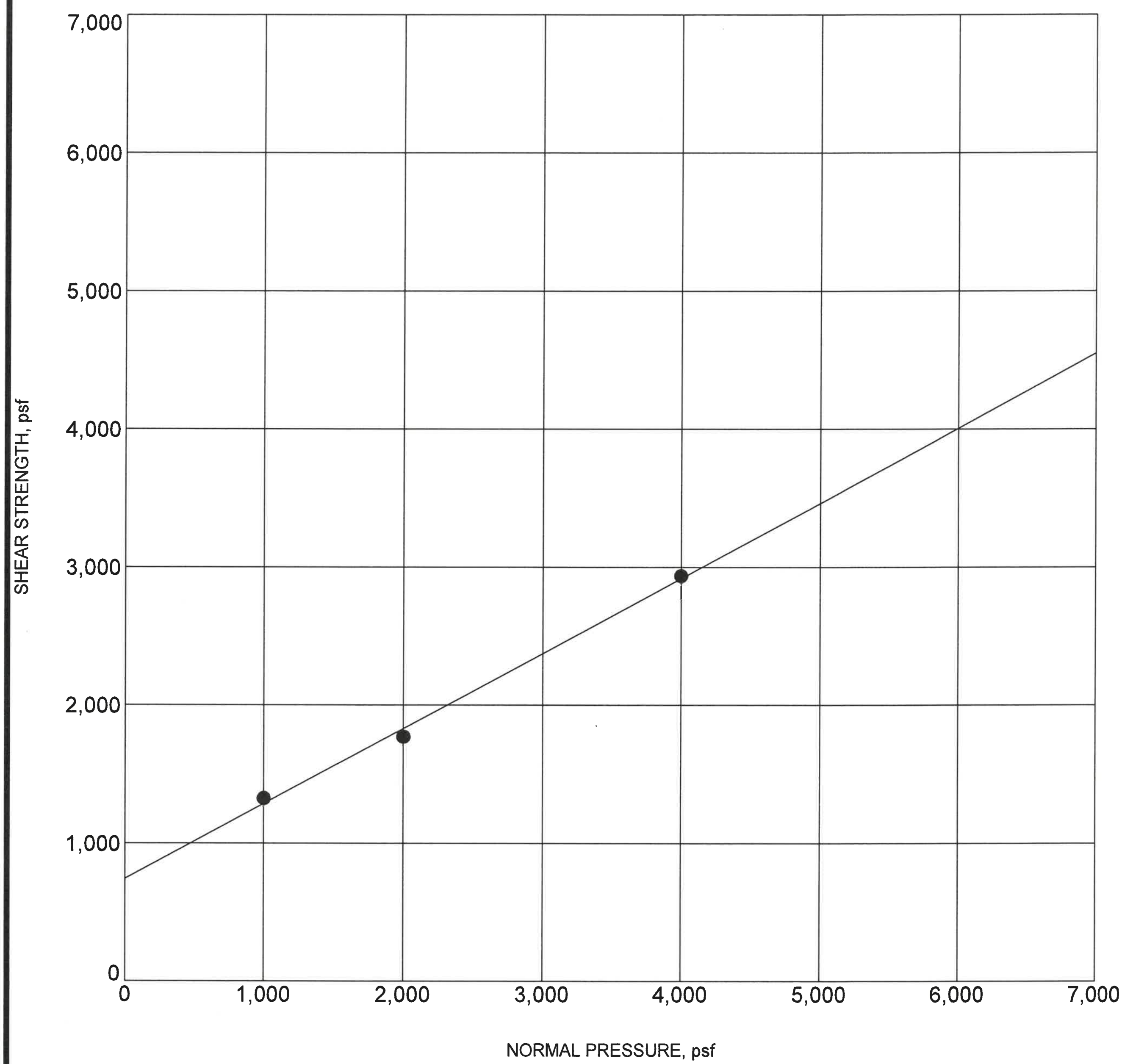



COMPACTION + EI DARK GRID 11683 GOLDFINCH.GPJ GEI FEB06.GDT 12/4/17

**Geotechnical Exploration, Inc.**

## MOISTURE-DENSITY RELATIONSHIP

Figure Number: IVa
Job Name: Goldfinch Street Residence
Site Location: 4285 Goldfinch Street, San Diego, CA
Job Number: 17-11683



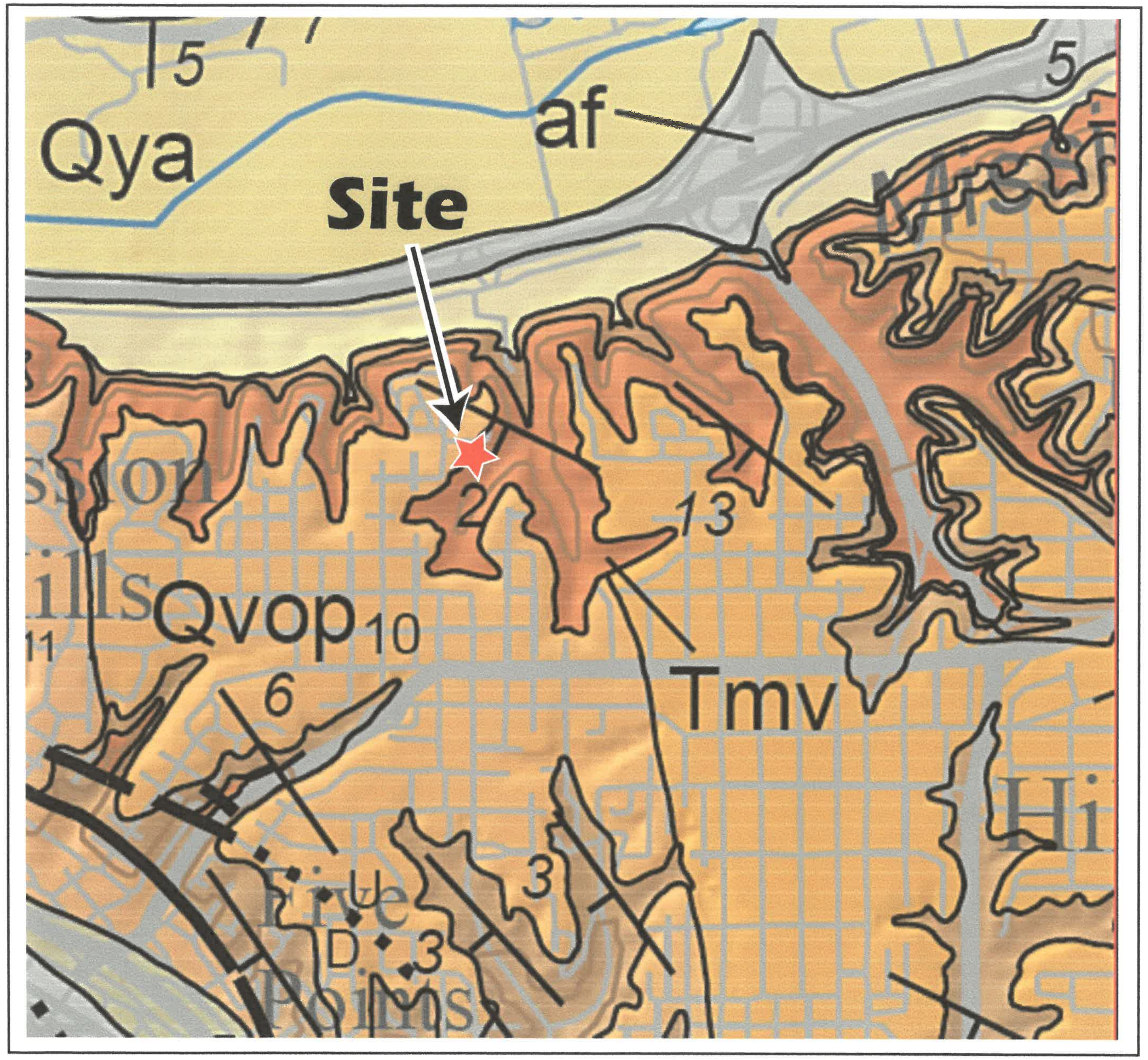

| | Specimen Identification | Classification | $\gamma_d$ | MC% | c | $\phi$ |
|---|---|---|---|---|---|---|
| ● | **B-2 @ 7.0'** | **CLAYEY SAND (SC), Light gray** | | | **742** | **29** |
| | | (ring samples) | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |
| | | | | | | |

**Geotechnical Exploration, Inc.**

## DIRECT SHEAR TEST

Figure Number: IVb
Job Name: Goldfinch Street Residence
Site Location: 4285 Goldfinch Street, San Diego, CA
Job Number: 17-11683

US DIRECT SHEAR 11683 GOLDFINCH.GPJ GEO EXPL.GDT 12/4/17




Goldfinch Street Residence
4285 Goldfinch Street
San Diego, CA.

Base Map

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

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

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



Goldfinch-2008-geo.ai

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

By
Michael P. Kennedy[1] and Siang S. Tan[1]
2008
Digital preparation by
Kelly R. Bovard[2], Anne G. Garcia[2], Diane Burns[2], and Carlos I. Gutierrez[1]

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

## ONSHORE MAP SYMBOLS

70 U D

Contact - Contact between geologic units; dotted where concealed.

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

Anticline - Solid where accurately located; dashed where approximately located; dotted where concealed. Arrow indicates direction of axial plunge.

Syncline - Solid where accurately located; dotted where concealed. Arrow indicates direction of axial plunge.

Qls?

Landslide - Arrows indicate principal direction of movement. Queried where existence is questionable.

Strike and dip of beds

70 Inclined

Strike and dip of igneous joints

60 Inclined

Vertical

Strike and dip of metamorphic foliation

55 Inclined

## ABBREVIATED EXPLANATION

| | |
|---|---|
| $Qvop_{10}$ | Very old paralic deposits |
| Tmv | Mission Valley Formation (middle Eocene) |

*Figure No. V*
*Job No. 17-11683*

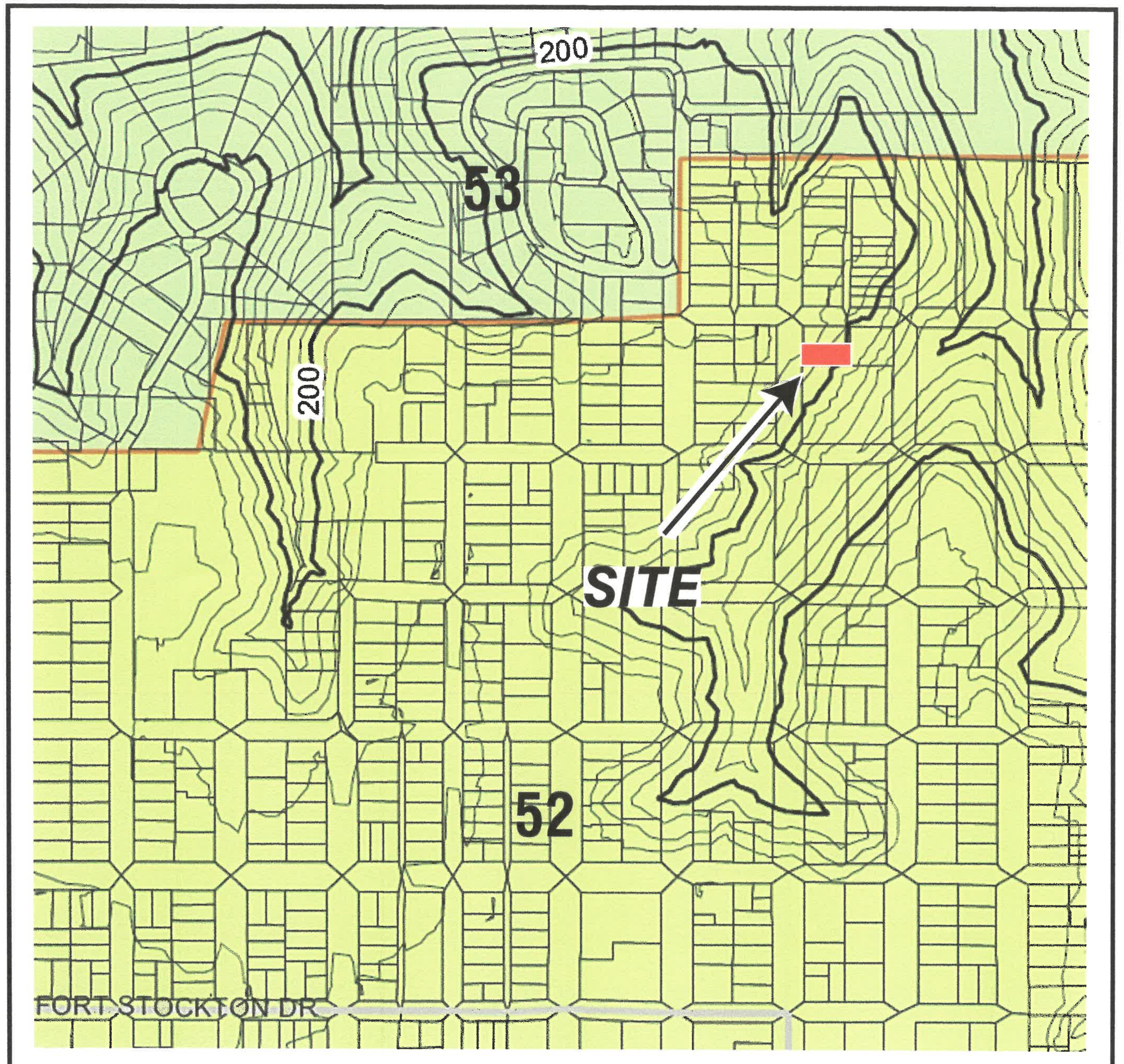
Geotechnical Exploration, Inc.

November 2018

# Geologic Hazards Map Excerpt from City of San Diego Geologic Hazards and Fault Map Sheet 21

Development Services Department

DATE: 4/3/2008




Goldfinch Street Residence
4285 Goldfinch Street
San Diego, CA.

## *LEGEND*

### *Geologic Hazard Categories*

**FAULT ZONES**

11 Active, Alquist-Priolo Earthquake Fault Zone

12 Potentially Active, Inactive, Presumed Inactive, or Activity Unknown

13 Downtown special fault zone

**LANDSLIDES**

21 Confirmed, known, or highly suspected

22 Possible or conjectured

**SLIDE-PRONE FORMATIONS**

23 Friars: neutral or favorable geologic structure

24 Friars: unfavorable geologic structure

25 Ardath: neutral or favorable geologic structure

26 Ardath: unfavorable geologic structure

27 Otay, Sweetwater, and others

**LIQUEFACTION**

31 High Potential -- shallow groundwater major drainages, hydraulic fills

32 Low Potential -- fluctuating groundwater minor drainages

**COASTAL BLUFFS**

41 Generally unstable Numerous landslides, high steep bluffs, severe erosion, unfavorable geologic structure

42 Generally unstable Unfavorable bedding plains, high erosion

43 Generally unstable Unfavorable jointing, local high erosion

44 Moderately stable Mostly stable formations, local high erosion

45 Moderately stable Some minor landslides, minor erosion

46 Moderately stable Some unfavorable geologic structure, minor or no erosion

47 Generally stable Favorable geologic structure, minor or no erosion, no landslides

48 Generally stable Broad beach areas, developed harbor

**OTHER TERRAIN**

51 Level mesas -- underlain by terrace deposits and bedrock nominal risk

52 Other level areas, gently sloping to steep terrain, favorable geologic structure, Low risk

53 Level or sloping terrain, unfavorable geologic structure, Low to moderate risk

54 Steeply sloping terrain, unfavorable or fault controlled geologic structure, Moderate risk

55 Modified terrain (graded sites) Nominal risk

**Water (Bays and Lakes)**

**FAULTS**

Fault

Inferred Fault

Concealed Fault

Shear Zone

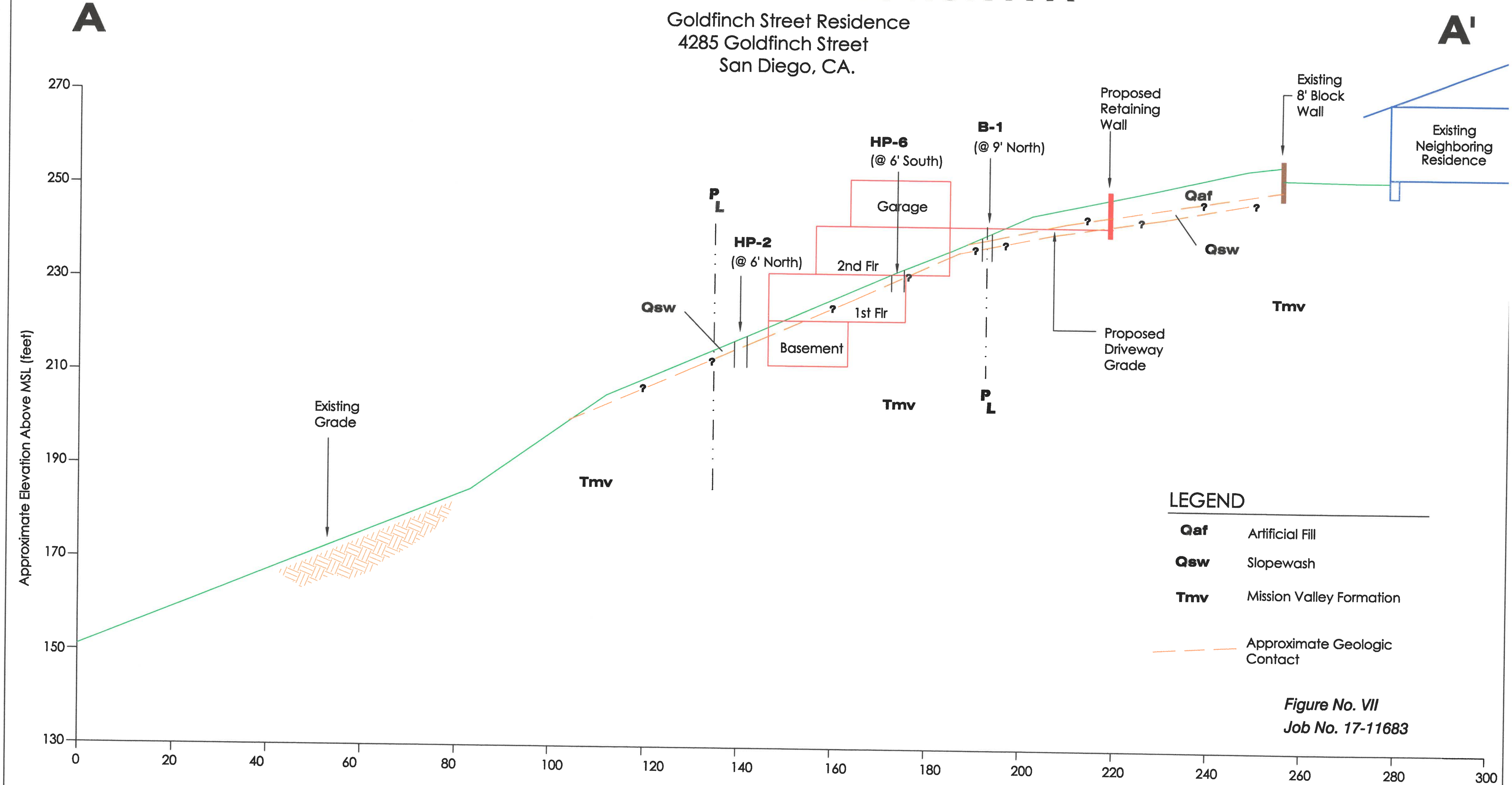
*Figure No. VI*
*Job No. 17-11683*

Geotechnical Exploration, Inc.

November 2018

# GEOLOGIC CROSS SECTION A-A'

Goldfinch Street Residence
4285 Goldfinch Street
San Diego, CA.




Relative Horizontal Distance (feet)
Scale:1" = 20'
(Horizontal and Vertical)

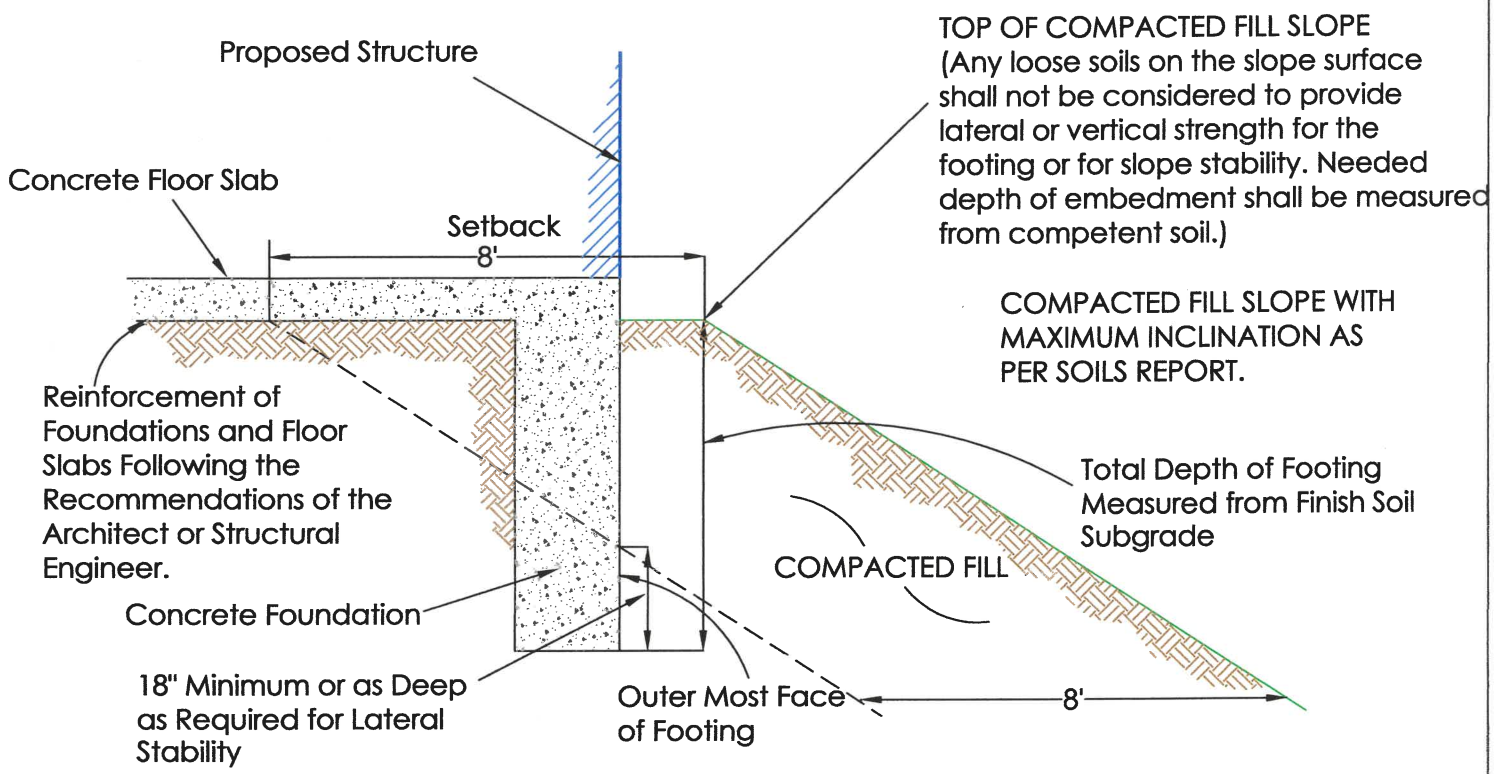
NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

*Figure No. VII*
*Job No. 17-11683*

Geotechnical Exploration, Inc.
*November 2018*

17-11683-AA

# 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

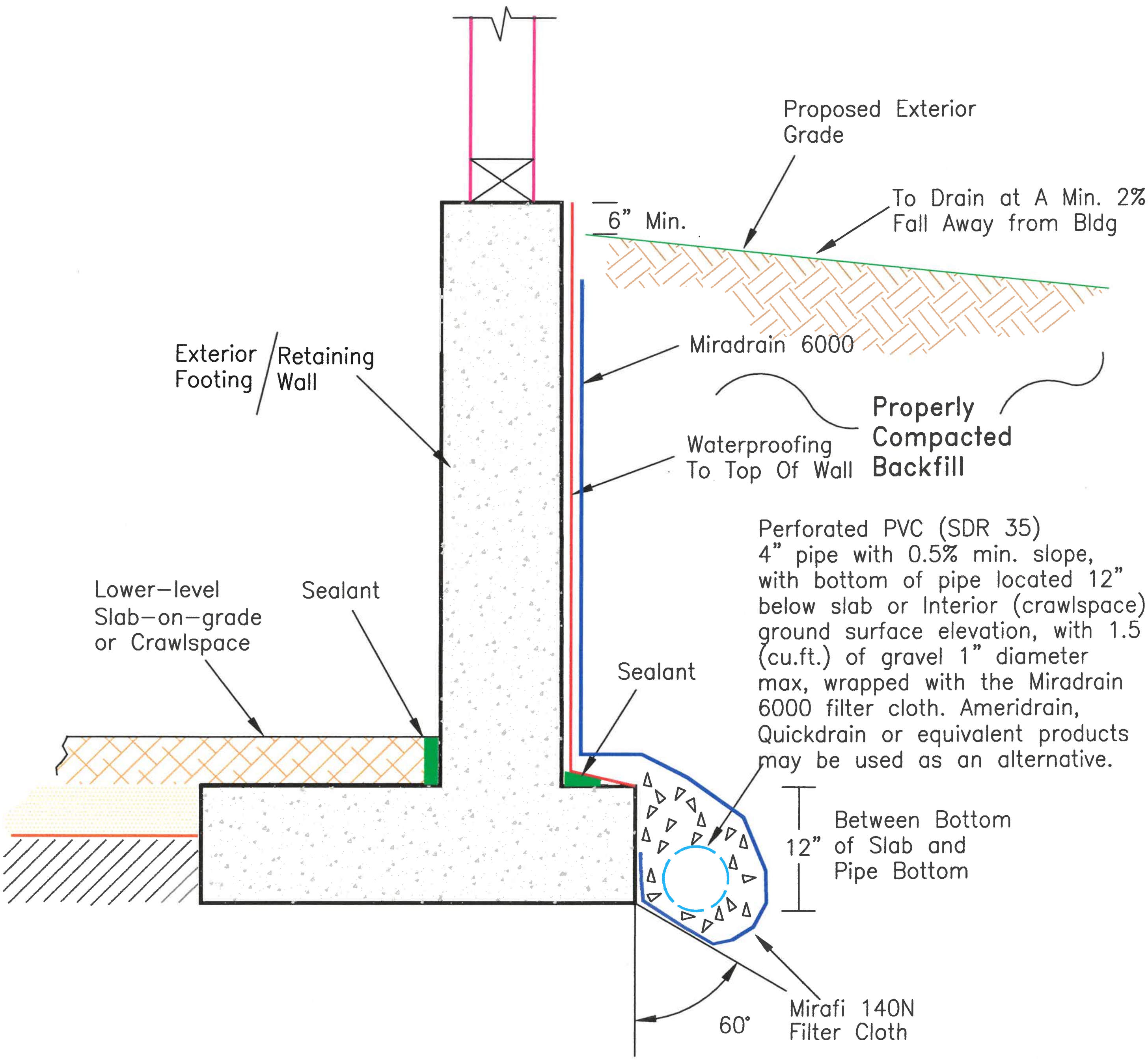
*Figure No. VIII*
*Job No. 17-11683*

Geotechnical Exploration, Inc.



*November 2018*

# RECOMMENDED SUBGRADE RETAINING WALL DRAINAGE SCHEMATIC




**NOT TO SCALE**

*Figure No. IX*

*Job No. 17-11683*

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.



November 2018

# 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

# SLOPE STABILITY CALCULATIONS

## *Appendix B*

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

*Goldfinch Street Residence*

***GEI Job No. 17-11683***

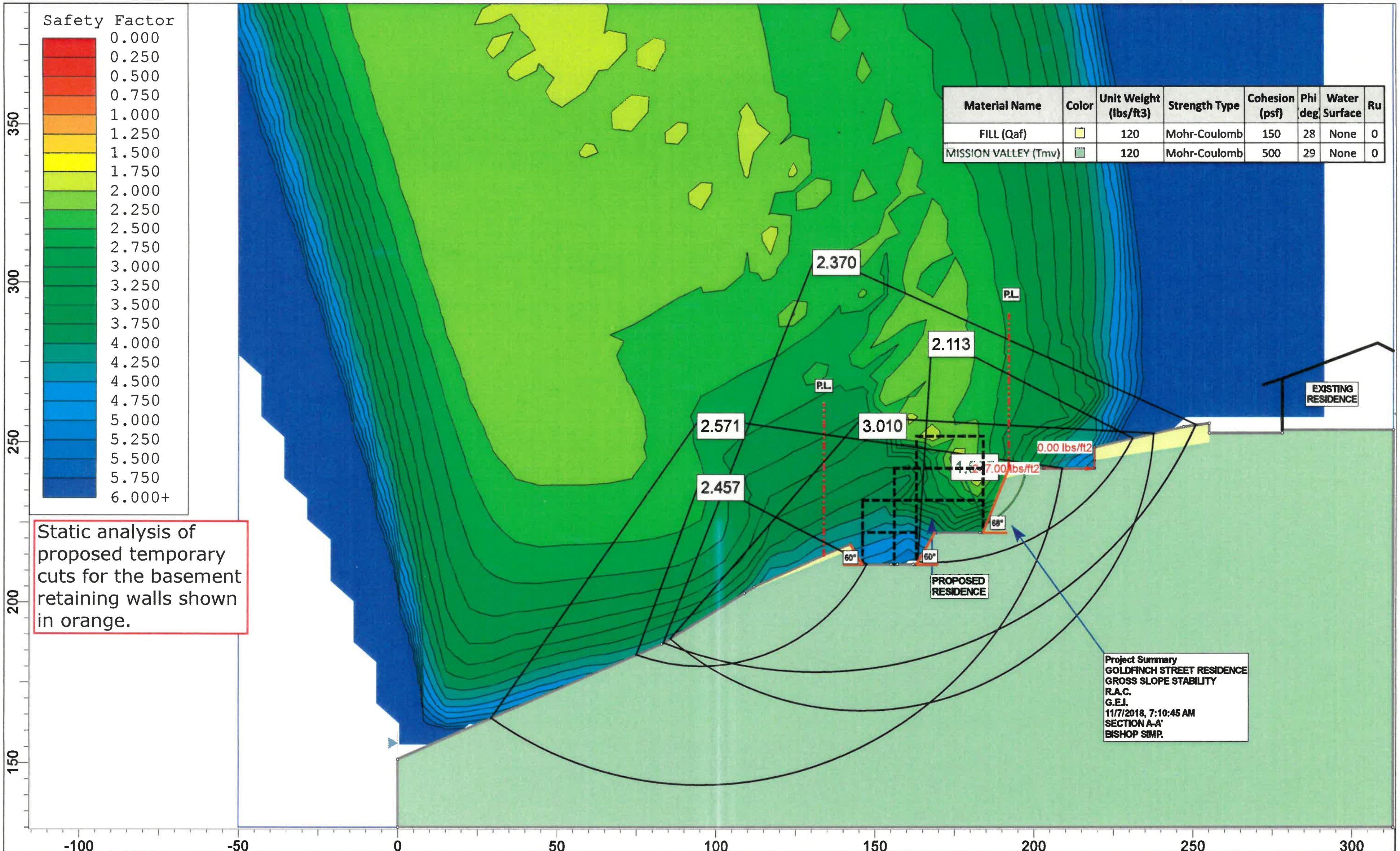
We performed gross slope stability calculations using the *SLIDE 6* program by Roc Science. The program is a limit equilibrium slope stability program that allows the use of several slope stability methods to calculate the factors of safety against shear failure. On this project, we used the Bishop Simplified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

The program calculates the factor of safety against shear 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 A-A' 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 and Direct shear tests that were performed on the on-site soils. The values used in the analysis were conservatively adjusted.

The analysis was performed under static conditions using the Bishop Simplified circular method, globally and locally, under different conditions. In the first condition, the proposed slope was analyzed without the lateral support of the basement and the exterior retaining walls. In the second condition, the proposed slope was analyzed with the effect of lateral retaining wall support. The equivalent fluid pressure used in the gross stability analysis on the basement retaining wall was 45 pcf (pounds per cubic feet) and 38 pcf for the exterior retaining walls. The third condition shows the proposed slope with temporary cuts that will be necessary to build the basement retaining wall.

Once the static gross stability of different slide planes was calculated, we analyzed the same section(s) (excluding the temporary cut analysis) by 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 soil strength values used for the shallow stability calculations were conservatively adjusted.

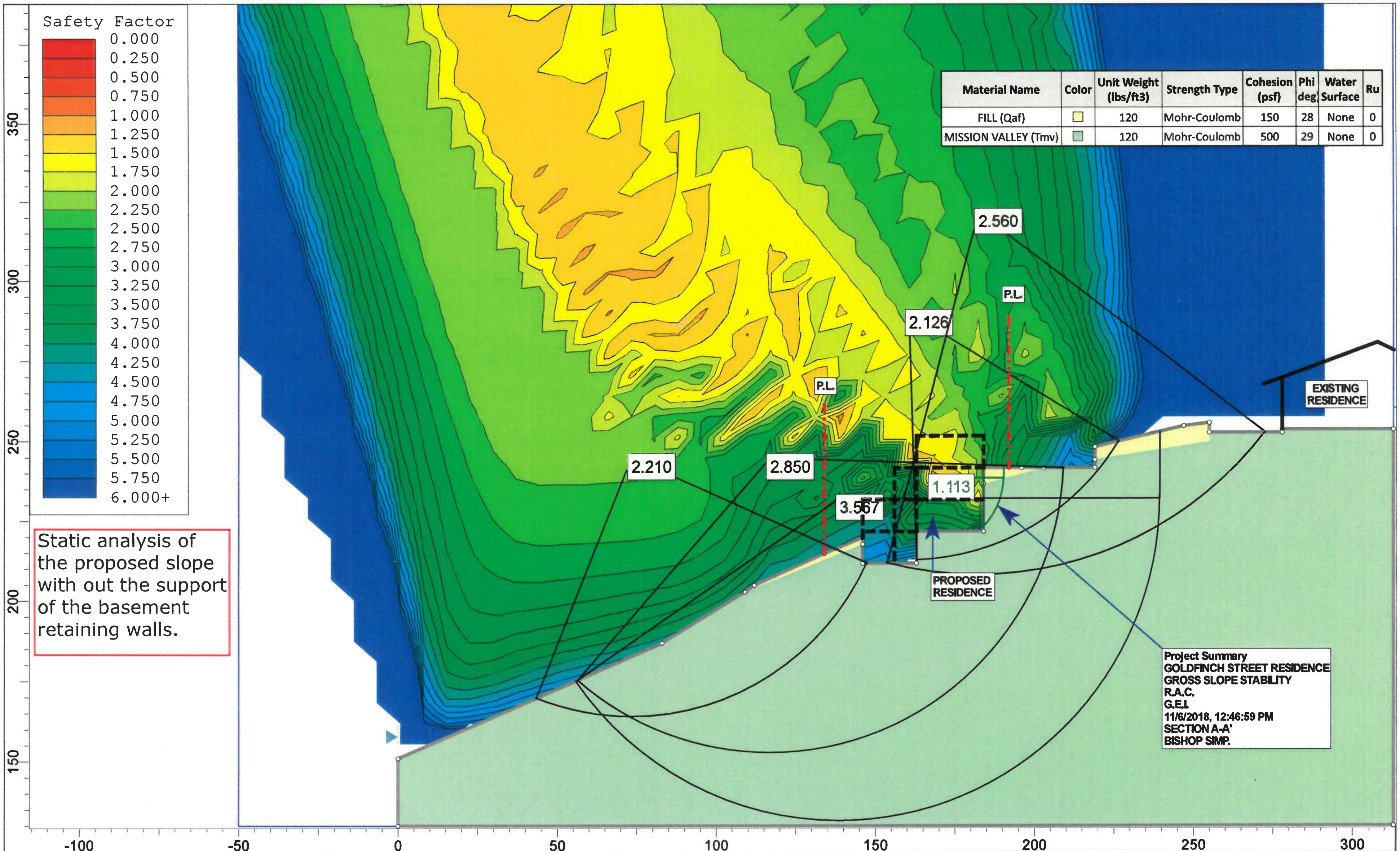



| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi deg | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| FILL (Qaf) | | 120 | Mohr-Coulomb | 150 | 28 | None | 0 |
| MISSION VALLEY (Tmv) | | 120 | Mohr-Coulomb | 500 | 29 | None | 0 |

**Geotechnical Exploration, Inc.**

| | | | |
|---|---|---|---|
| Project | GOLDFINCH STREET RESIDENCE | | |
| Analysis Description | GROSS SLOPE STABILITY | | |
| Drawn By | R.A.C. | Scale | 1:500 |
| Company | G.E.I. | | |
| Date | 11/7/2018, 7:10:45 AM | File Name | JOB NO. 17-11683_SA_03.slim |

SLIDEINTERPRET 6.039

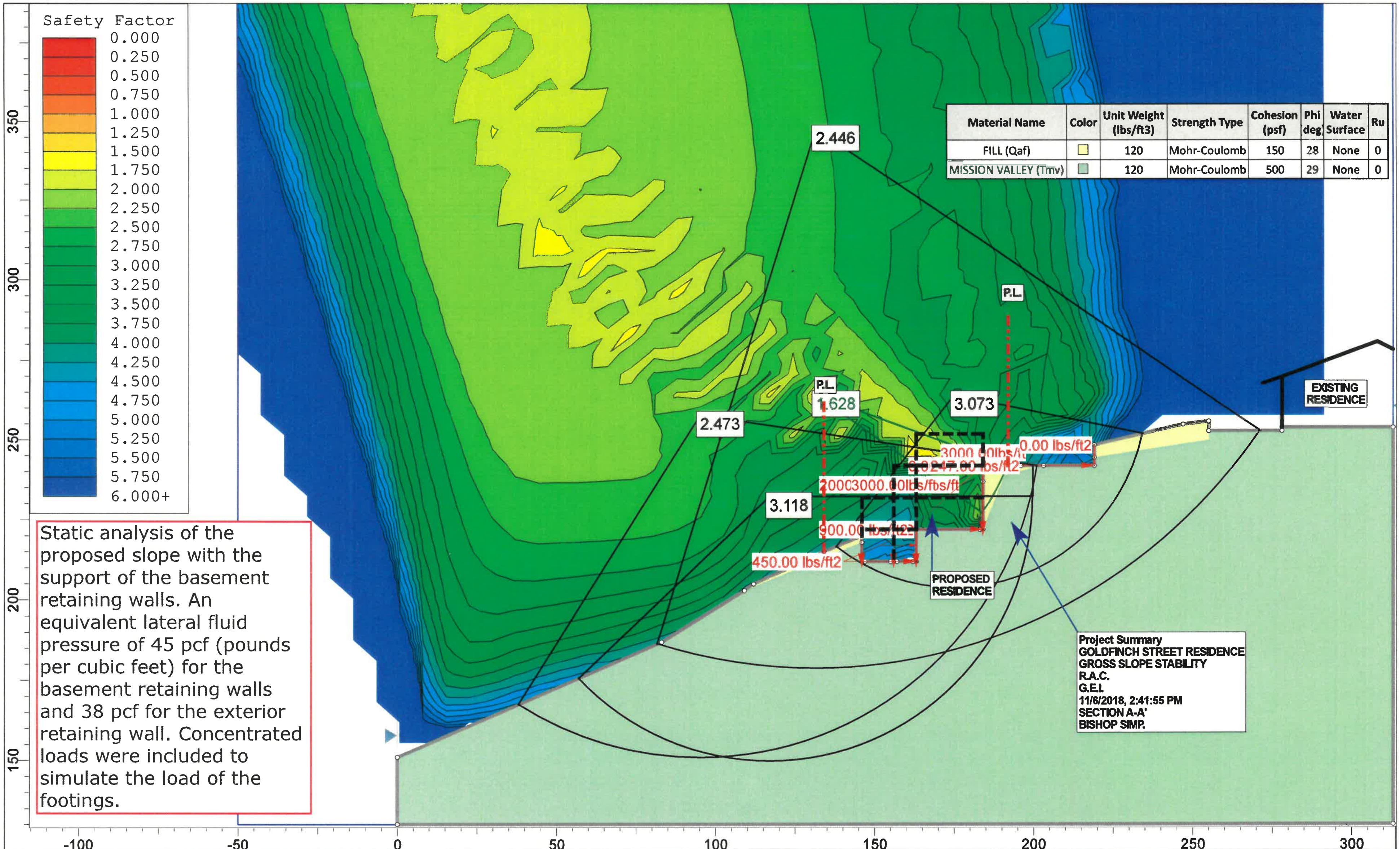



| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi deg | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| FILL (Qaf) | | 120 | Mohr-Coulomb | 150 | 28 | None | 0 |
| MISSION VALLEY (Tmv) | | 120 | Mohr-Coulomb | 500 | 29 | None | 0 |

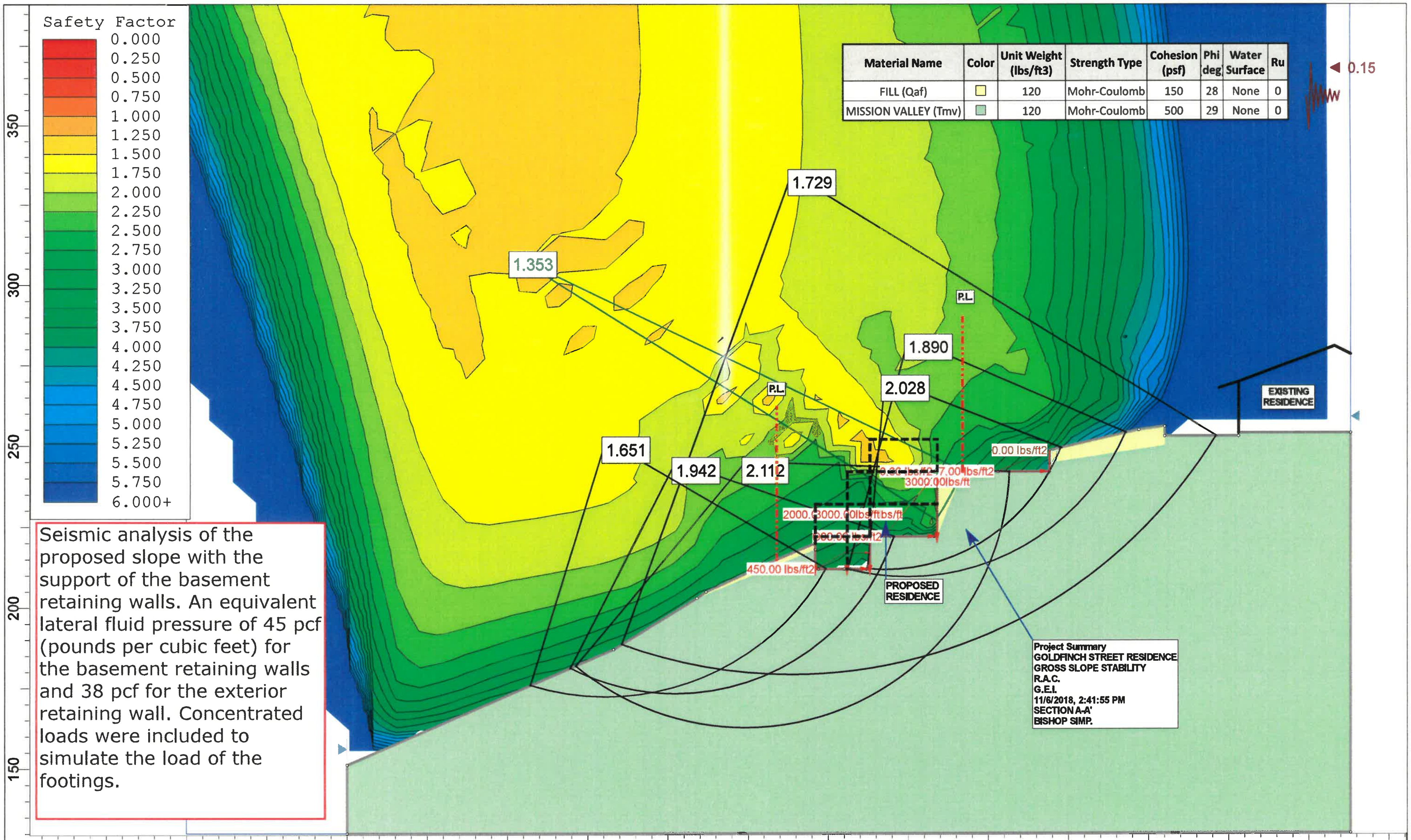
**Geotechnical Exploration, Inc.**

SLIDEINTERPRET 6.039

| | | | |
|---|---|---|---|
| Project | GOLDFINCH STREET RESIDENCE | | |
| Analysis Description | GROSS SLOPE STABILITY | | |
| Drawn By | R.A.C. | Scale | 1:500 |
| Company | G.E.I. | | |
| Date | 11/6/2018, 12:46:59 PM | File Name | JOB NO. 17-11683_SA_01.slim |




| | | | |
|---|---|---|---|
| **Geotechnical Exploration, Inc.** | Project | GOLDFINCH STREET RESIDENCE | |
| | Analysis Description | GROSS SLOPE STABILITY | |
| | Drawn By: R.A.C. | Scale: 1:500 | Company: G.E.I. |
| SLIDEINTERPRET 6.039 | Date: 11/6/2018, 2:41:55 PM | | File Name: JOB NO. 17-11683_SA_02.slim |

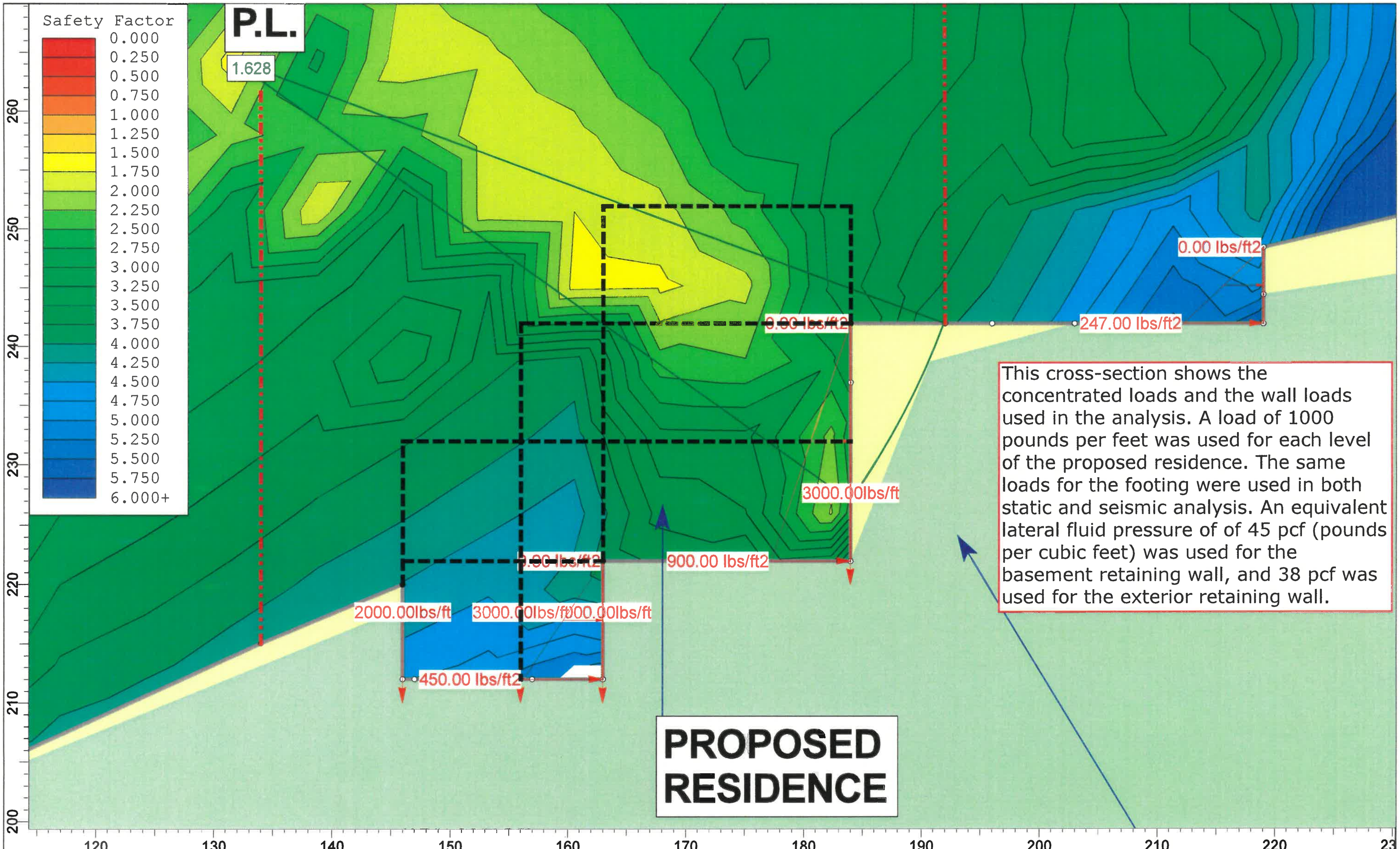



| Material Name | Color | Unit Weight (lbs/ft3) | Strength Type | Cohesion (psf) | Phi deg | Water Surface | Ru |
|---|---|---|---|---|---|---|---|
| FILL (Qaf) | | 120 | Mohr-Coulomb | 150 | 28 | None | 0 |
| MISSION VALLEY (Tmv) | | 120 | Mohr-Coulomb | 500 | 29 | None | 0 |

**Geotechnical Exploration, Inc.**

SLIDEINTERPRET 6.039

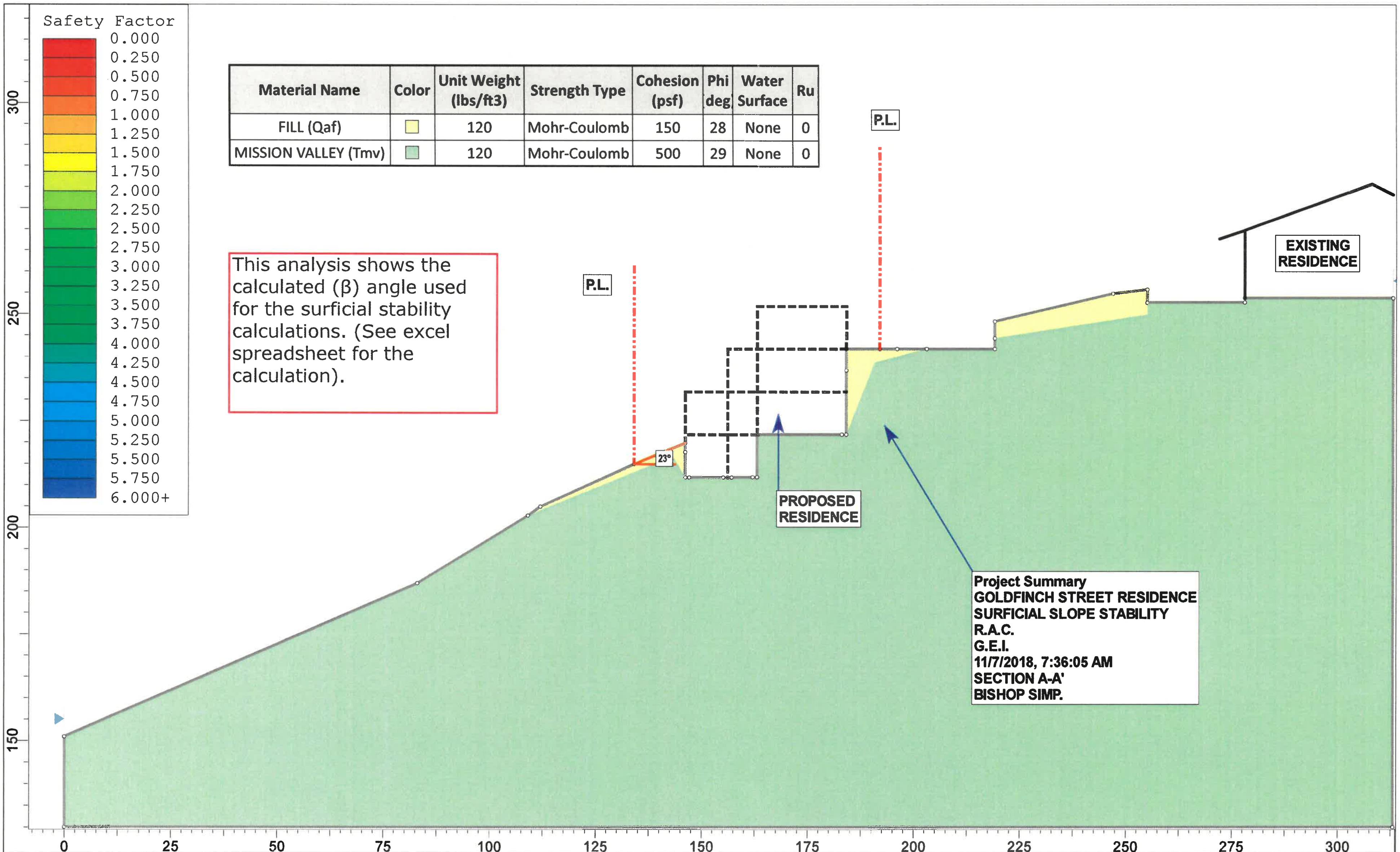
| | |
|---|---|
| Project | GOLDFINCH STREET RESIDENCE |
| Analysis Description | GROSS SLOPE STABILITY |
| Drawn By | R.A.C. |
| Scale | 1:500 |
| Company | G.E.I. |
| Date | 11/6/2018, 2:41:55 PM |
| File Name | JOB NO. 17-11683_SA_02w_0.15gSHAKE.slim |




Geotechnical Exploration, Inc.

| | | | |
|---|---|---|---|
| Project | GOLDFINCH STREET RESIDENCE | | |
| Analysis Description | GROSS SLOPE STABILITY | | |
| Drawn By | R.A.C. | Scale | 1:135 |
| | | Company | G.E.I. |
| Date | 11/6/2018, 2:41:55 PM | File Name | JOB NO. 17-11683_SA_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 | 28 | None | 0 |
| MISSION VALLEY (Tmv) | | 120 | Mohr-Coulomb | 500 | 29 | None | 0 |

**Geotechnical Exploration, Inc.**

SLIDEINTERPRET 6.039

| | |
|---|---|
| Project | GOLDFINCH STREET RESIDENCE |
| Analysis Description | SURFICIAL SLOPE STABILITY |
| Drawn By | R.A.C. |
| Scale | 1:375 |
| Company | G.E.I. |
| Date | 11/7/2018, 7:36:05 AM |
| File Name | JOB NO. 17-11683_SA_04.slim |

## SURFICIAL FAILURE

**EQUATION 1**

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

| $\gamma_{sat}$ | $\gamma_{water}$ | $\gamma'$ | H |
|---|---|---|---|
| pcf | pcf | pcf | ft |
| 130 | 62.4 | 67.6 | 3 |

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

| SECTION A-A' | | | | |
|---|---|---|---|---|
| SOIL TYPE | C (psf) | ϕ(°) | β(°) | F.S. |
| FILL ($Q_{af}$) | 150 | 28 | 23 | 1.721 |

| | |
|---|---|
| β | Slope inclination with respect to the horizontal plane |
| ϕ | Friction angle of the soil |
| C | Cohesion of the soil |
| $\gamma_{sat}$ | Saturated unit weight of the soil |
| $\gamma'$ | 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.

# APPENDIX C

# USGS DESIGN MAPS SUMMARY REPORT

# USGS Design Maps Summary Report

## User–Specified Input

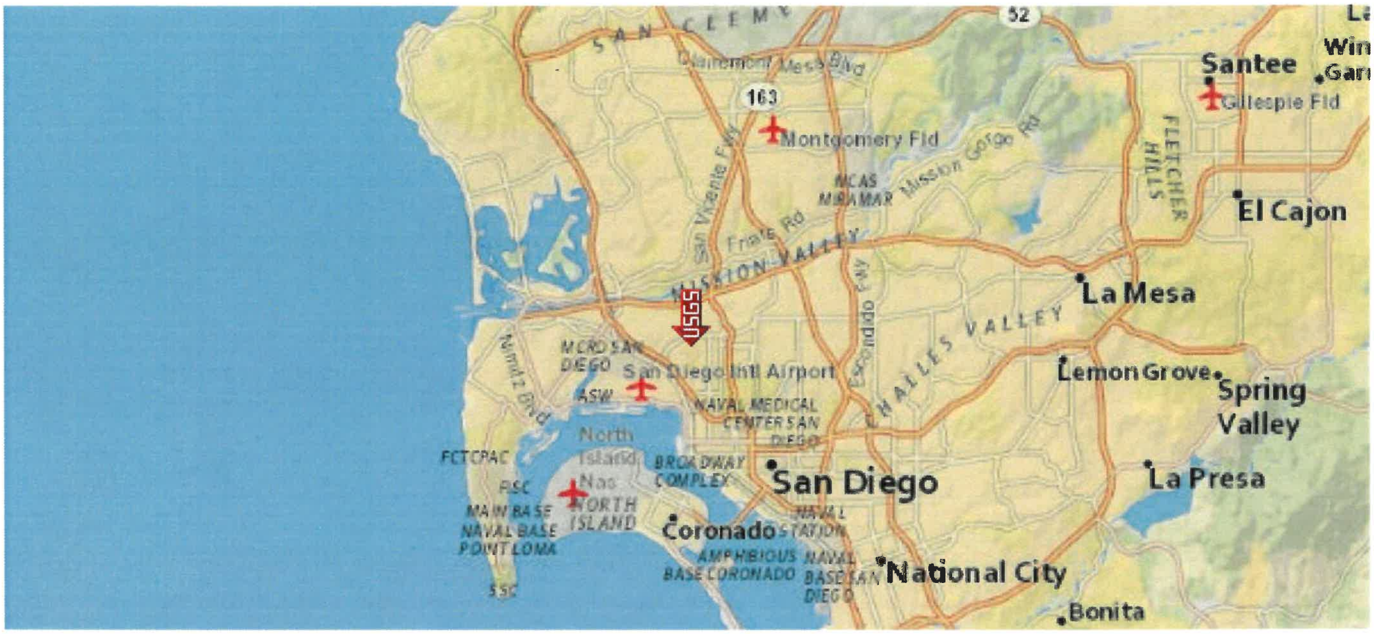
**Report Title** Goldfinch Street Residence
Tue November 6, 2018 18:53:20 UTC

**Building Code Reference Document** ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

**Site Coordinates** 32.7554°N, 117.1713°W

**Site Soil Classification** Site Class D – "Stiff Soil"

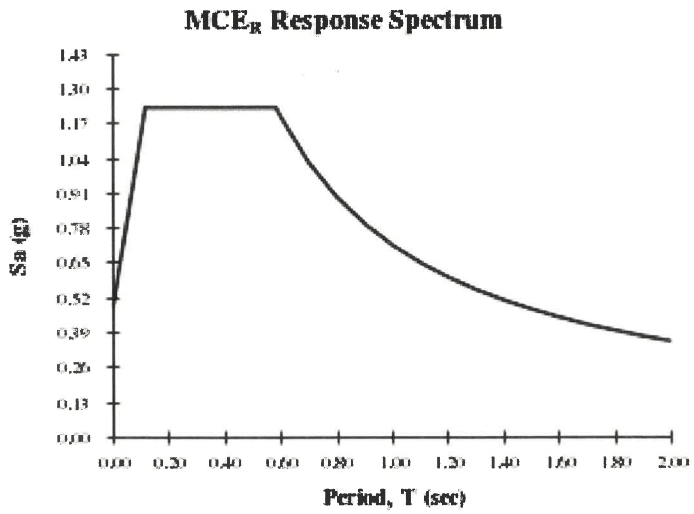
**Risk Category** I/II/III

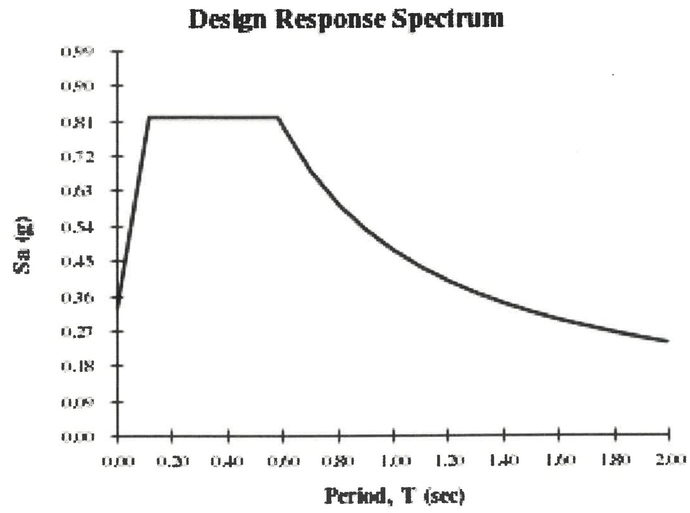



## USGS–Provided Output

| | | | | | |
|---|---|---|---|---|---|
| $S_S =$ | 1.212 g | $S_{MS} =$ | 1.230 g | $S_{DS} =$ | 0.820 g |
| $S_1 =$ | 0.467 g | $S_{M1} =$ | 0.716 g | $S_{D1} =$ | 0.478 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.

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# Design Maps Detailed Report

## ASCE 7-10 Standard (32.7554°N, 117.1713°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

## Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain $S_S$) and 1.3 (to obtain $S_1$). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

| | |
|---|---|
| **From Figure 22-1** [1] | $S_S = 1.212$ g |
| **From Figure 22-2** [2] | $S_1 = 0.467$ g |

## Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

| **Site Class** | $\bar{v}_s$ | $\bar{N}$ or $\bar{N}_{ch}$ | $\bar{s}_u$ |
|---|---|---|---|
| A. Hard Rock | >5,000 ft/s | N/A | N/A |
| B. Rock | 2,500 to 5,000 ft/s | N/A | N/A |
| C. Very dense soil and soft rock | 1,200 to 2,500 ft/s | >50 | >2,000 psf |
| D. Stiff Soil | 600 to 1,200 ft/s | 15 to 50 | 1,000 to 2,000 psf |
| E. Soft clay soil | <600 ft/s | <15 | <1,000 psf |
| | Any profile with more than 10 ft of soil having the characteristics:<br>• Plasticity index *PI* > 20,<br>• Moisture content $w \geq 40\%$, and<br>• Undrained shear strength $\bar{s}_u$ < 500 psf | | |
| F. Soils requiring site response analysis in accordance with Section 21.1 | See Section 20.3.1 | | |

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

## Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake ($MCE_R$) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient $F_a$

| Site Class | Mapped $MCE_R$ Spectral Response Acceleration Parameter at Short Period | | | | |
|---|---|---|---|---|---|
| | $S_S \le 0.25$ | $S_S = 0.50$ | $S_S = 0.75$ | $S_S = 1.00$ | $S_S \ge 1.25$ |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.2 | 1.2 | 1.1 | 1.0 | 1.0 |
| D | 1.6 | 1.4 | 1.2 | 1.1 | 1.0 |
| E | 2.5 | 1.7 | 1.2 | 0.9 | 0.9 |
| F | See Section 11.4.7 of ASCE 7 | | | | |

Note: Use straight–line interpolation for intermediate values of $S_S$

**For Site Class = D and $S_S$ = 1.212 g, $F_a$ = 1.015**

Table 11.4–2: Site Coefficient $F_v$

| Site Class | Mapped $MCE_R$ Spectral Response Acceleration Parameter at 1–s Period | | | | |
|---|---|---|---|---|---|
| | $S_1 \le 0.10$ | $S_1 = 0.20$ | $S_1 = 0.30$ | $S_1 = 0.40$ | $S_1 \ge 0.50$ |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.7 | 1.6 | 1.5 | 1.4 | 1.3 |
| D | 2.4 | 2.0 | 1.8 | 1.6 | 1.5 |
| E | 3.5 | 3.2 | 2.8 | 2.4 | 2.4 |
| F | See Section 11.4.7 of ASCE 7 | | | | |

Note: Use straight–line interpolation for intermediate values of $S_1$

**For Site Class = D and $S_1$ = 0.467 g, $F_v$ = 1.533**

**Equation (11.4–1):** $S_{MS} = F_a S_S = 1.015 \times 1.212 = 1.230$ g

**Equation (11.4–2):** $S_{M1} = F_v S_1 = 1.533 \times 0.467 = 0.716$ g

## Section 11.4.4 — Design Spectral Acceleration Parameters

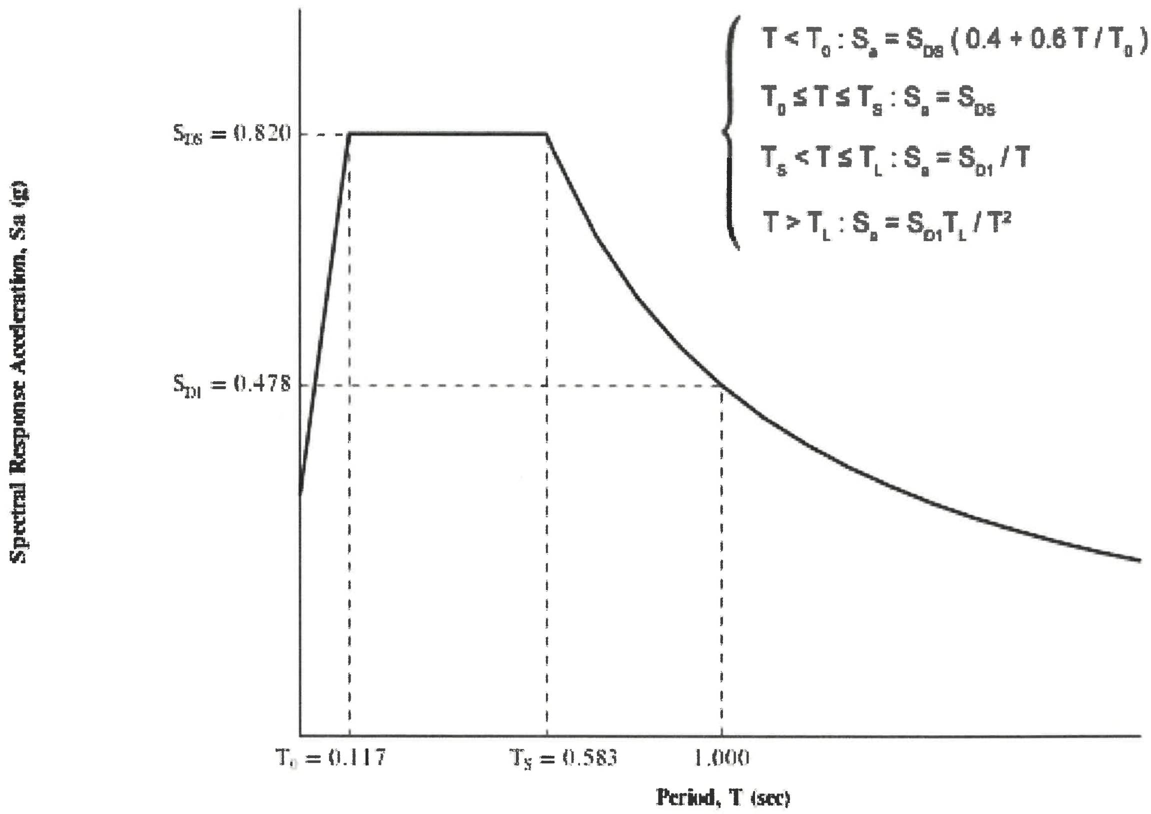
**Equation (11.4–3):** $S_{DS} = ⅔\, S_{MS} = ⅔ \times 1.230 = 0.820$ g

**Equation (11.4–4):** $S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.716 = 0.478$ g

## Section 11.4.5 — Design Response Spectrum

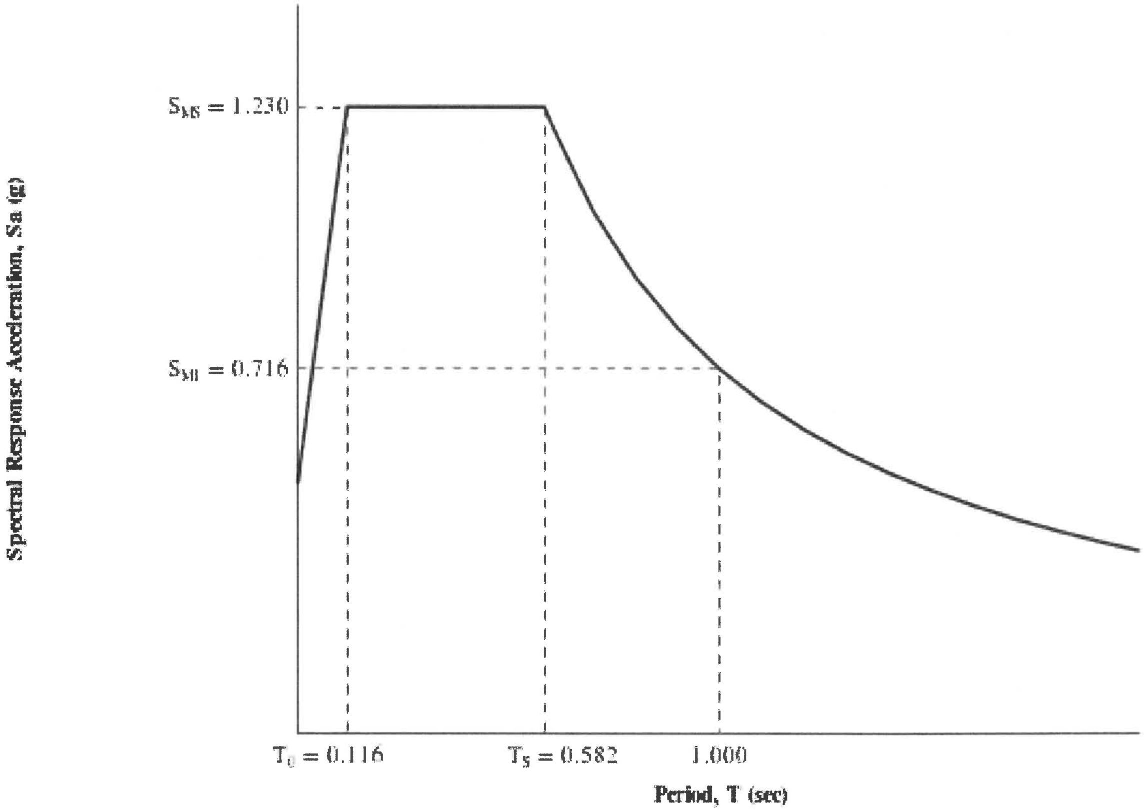
**From Figure 22-12** [3] $T_L = 8$ seconds

Figure 11.4–1: Design Response Spectrum



## Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake ($MCE_R$) Response Spectrum

The $MCE_R$ Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



## Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

**From Figure 22-7** [4] PGA = 0.541

**Equation (11.8–1):** $PGA_M = F_{PGA}PGA = 1.000 \times 0.541 = 0.541$ g

Table 11.8–1: Site Coefficient $F_{PGA}$

| Site Class | Mapped MCE Geometric Mean Peak Ground Acceleration, PGA | | | | |
|---|---|---|---|---|---|
| | PGA ≤ 0.10 | PGA = 0.20 | PGA = 0.30 | PGA = 0.40 | PGA ≥ 0.50 |
| A | 0.8 | 0.8 | 0.8 | 0.8 | 0.8 |
| B | 1.0 | 1.0 | 1.0 | 1.0 | 1.0 |
| C | 1.2 | 1.2 | 1.1 | 1.0 | 1.0 |
| D | 1.6 | 1.4 | 1.2 | 1.1 | $\boxed{1.0}$ |
| E | 2.5 | 1.7 | 1.2 | 0.9 | 0.9 |
| F | See Section 11.4.7 of ASCE 7 | | | | |

Note: Use straight–line interpolation for intermediate values of PGA

**For Site Class = D and PGA = 0.541 g, $F_{PGA}$ = 1.000**

## Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

**From Figure 22-17** [5] $C_{RS}$ = 0.848

**From Figure 22-18** [6] $C_{R1}$ = 0.885

## Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

| VALUE OF $S_{DS}$ | RISK CATEGORY | | |
|---|---|---|---|
| | I or II | III | IV |
| $S_{DS} < 0.167g$ | A | A | A |
| $0.167g \leq S_{DS} < 0.33g$ | B | B | C |
| $0.33g \leq S_{DS} < 0.50g$ | C | C | D |
| $0.50g \leq S_{DS}$ | D | D | D |

**For Risk Category = I and $S_{DS}$ = 0.820 g, Seismic Design Category = D**

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

| VALUE OF $S_{D1}$ | RISK CATEGORY | | |
|---|---|---|---|
| | I or II | III | IV |
| $S_{D1} < 0.067g$ | A | A | A |
| $0.067g \leq S_{D1} < 0.133g$ | B | B | C |
| $0.133g \leq S_{D1} < 0.20g$ | C | C | D |
| $0.20g \leq S_{D1}$ | D | D | D |

**For Risk Category = I and $S_{D1}$ = 0.478 g, Seismic Design Category = D**

Note: When $S_1$ is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

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Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

## References


1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf

5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf