REPORT OF PRELIMINARY
GEOTECHNICAL INVESTIGATION
Hicks Residential Remodel
8405 Paseo Del Ocaso
La Jolla, California

JOB NO. 17-11479
22 June 2017

Prepared for:

Tom & Cinda Hicks

Geotechnical Exploration, Inc.

SOIL AND FOUNDATION ENGINEERING • GROUNDWATER • ENGINEERING GEOLOGY

22 June 2017

Tom & Cinda Hicks
1918 N. Olive Street, #3001
Dallas, TX 75201

Job No. 17-11479

Subject: **<u>Report of Preliminary Geotechnical Investigation</u>**
Hicks Residential Remodel
8405 Paseo Del Ocaso
La Jolla, California

Dear Mr. & Mrs. Hicks:

In accordance with your request, and our proposal of December 16, 2016, ***Geotechnical Exploration, Inc.*** has performed a preliminary geotechnical investigation for the subject property. The field work was performed on April 27, 2017.

In our opinion, if the conclusions and recommendations presented in this report are implemented during site preparation and construction, the site will be suited for the proposed residential remodel and associated improvements.

This opportunity to be of service is sincerely appreciated. Should you have any questions concerning the following report, please do not hesitate to contact us. Reference to our **Job No. 17-11479** will expedite a response to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

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8405 Paseo Del Ocaso
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The following report presents the findings and recommendations of ***Geotechnical Exploration, Inc.*** for the subject project.

I. PROJECT SUMMARY

It is our understanding, based on communications with you, that the existing residence will undergo an extensive remodel, including a new second-story addition and associated improvements. We understand that the existing two-car garage will remain and become attached to the remodeled structure. The remodeled residential structure is to be constructed of standard-type building materials utilizing a conventional foundation system with raised wood floors and slab on-grade.

Final construction plans have not been provided to us during the preparation of this report, however, when completed they should be made available for our review. Additional or modified recommendations will be provided at that time if warranted.

II. SCOPE OF WORK

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, and slab-on-grade floors.

III. SITE DESCRIPTION

The property is known as Assessor's Parcel No. 346-082-03-00, Lot 18, Block 29 per Recorded Map 2061, in the La Jolla area of the City of San Diego, County of San Diego, State of California. Refer to Figure No. I, the Vicinity Map, for the site location.

The site, more particularly referred to as 8405 Paseo del Ocaso, consists of approximately 5,800 square feet. The lot is located on the east side of Paseo Del Ocaso, in the La Jolla area of the City of San Diego. The property is bordered on the north and east at approximately the same elevation by a similar residential properties; and to the south at approximately the same elevation by Camino Del Oro. The lot slopes gently to the west. Refer to Figure No. II for the Site Plan.

Existing structures on the property consist of a single-story, single-family residence with a detached garage and associated improvements. The garage is located near the southeast corner of the property and is accessed by Camino del Oro. Vegetation consists of ornamental landscaping including trees, decorative shrubbery and lawn grass.

The building pad gently slopes to the west and is at an approximate elevation of 25 feet above mean sea level (MSL). Elevations across the property range from approximately 28 feet above MSL along the east property line to approximately 20 feet above MSL along the west property line.

IV. FIELD & LABORATORY TESTS & SOIL INFORMATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using a limited access drill to investigate and sample the subsurface soils. Two exploratory borings were advanced around the existing residential structure. Both exploratory borings were drilled to maximum depths of 15 and 9 feet, respectively, in order to obtain representative soil samples and to define a soil profile across the project area.

The soils encountered in the exploratory borings 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 borings are shown on the Site Plan, Figure No. II.

Representative samples were obtained from the exploratory borings at selected depths appropriate to the investigation. All samples were returned to our laboratory for evaluation and testing. Exploratory boring logs were prepared on the basis of our observations and laboratory test results. Logs of the exploratory borings are attached as Figure Nos. IIIa-b.

A. Field Tests

Relatively undisturbed samples were obtained from the borings by driving a 3-inch outside-diameter (O.D.) by 2⅜-inch inside-diameter (I.D.) split-tube sampler a distance of 12 inches. Standard Penetration Tests were also performed by using a 140-pound weight falling 30 inches to drive a 2-inch O.D. by 1⅜-inch I.D. sampler tube a distance of 18 inches. The number of blows required to drive the sampler the last 12 inches was recorded for use in evaluation of the soil consistency. The following chart provides an in-house correlation between the number of blows and the consistency 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 Non-plastic 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
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	15-30	19-45
	Hard	31-60	46-90
	Very Hard	Over 60	Over 90

In general the tests performed in the field included: the Standard Practice for Soil Investigation and Sampling by Auger Borings (ASTM D1452), Test Method for Penetration Test and Split-barrel Sampling of Soils (ASTM D1586) and Standard Practice for Ring-lined Barrel Sampling of Soils (ASTM D3550). Bulk (disturbed) samples of the encountered soils were also retrieved for subsequent laboratory testing.

B. <u>Laboratory Tests</u>

Laboratory tests were performed on the retrieved soil samples in order to evaluate their index, strength, expansion, and compressibility properties. The test results are presented on Figures Nos. IIIa-c. The following tests were conducted on representative soil samples:

1. *Moisture Content (ASTM D2216-10)*
2. *Determination of Percentage of Particles Smaller than #200 Sieve (ASTM D1140-14)*
3. *Density Measurements (ASTM D2937-10)*

Moisture content measurements were performed to establish the in situ moisture of samples retrieved from the exploratory excavations. Moisture content and density measurements were performed by ASTM methods D2216 and D2937. These density tests help to establish the in situ moisture and density of samples retrieved from the exploratory excavations.

Laboratory compaction values (ASTM D1557) establish the optimum moisture content and the laboratory maximum dry density of the tested soils. The relationship between the moisture and density of remolded soil samples helps to establish the relative compaction of the existing fill soils and soil compaction conditions to be anticipated during any future grading operation.

The particle size smaller than a No. 200 sieve analysis (ASTM D1140-06) tests (ASTM D4318-05) aid in classifying the tested soils in accordance with the Unified Soil Classification System and provide qualitative information related to engineering characteristics such as expansion potential, permeability, and shear strength.

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

EXPANSION INDEX	POTENTIAL EXPANSION
0 to 20	Very low
21 to 50	Low
51 to 90	Medium
91 to 130	High
Above 130	Very high

Based on the particle size test results and our experience with the encountered soils, it is our opinion that the on-site formational soils in general possess a low expansion potential.

V. REGIONAL GEOLOGIC DESCRIPTION

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

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

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

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

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

In California, major earthquakes can generally be correlated with movement on active faults. As defined by the California Division of Mines and Geology (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 Geologist, 1973). The California Division of Mines and Geology (now the California Geological Survey) defines a *"potentially active"* fault as one that has had ground surface displacement during Quaternary time, that is, between 11,000 and 1.6 million years (Hart, E.W., 1980).

During recent history, prior to April 2010, the San Diego County area has been relatively quiet seismically. No fault ruptures or major earthquakes had been experienced in historic time within the greater San Diego area. Since earthquakes have been recorded by instruments (since the 1930s), the San Diego area has experienced scattered seismic events with Richter magnitudes generally less than M4.0. During June 1985, a series of small earthquakes occurred beneath San Diego Bay, three of which were recorded at M4.0 to M4.2. In addition, the

Oceanside earthquake of July 13, 1986, located approximately 26 miles offshore of the City of Oceanside, had a magnitude of M5.3 (Hauksson and Jones, 1988).

On June 15, 2004, a M5.3 earthquake occurred approximately 45 miles southwest of downtown San Diego (26 miles west of Rosarito, Mexico). Although this earthquake was widely felt, no significant damage was reported. Another widely felt earthquake on a distant southern California fault was a M5.4 event that took place on July 29, 2008, west-southwest of the Chino Hills area of Riverside County.

Several earthquakes ranging from M5.0 to M6.0 occurred in northern Baja California, centered in the Gulf of California on August 3, 2009. These were felt in San Diego but no injuries or damage was reported. A M5.8 earthquake followed by a M4.9 aftershock occurred on December 30, 2009, centered about 20 miles south of the Mexican border city of Mexicali. These were also felt in San Diego, swaying high-rise buildings, but again no significant damage or injuries were reported.

On Easter Sunday April 4, 2010, a large earthquake occurred in Baja California, Mexico. It was widely felt throughout the southwest including Phoenix, Arizona and San Diego in California. This M7.2 event, the Sierra El Mayor earthquake, occurred in northern Baja California, approximately 40 miles south of the Mexico-USA border at shallow depth along the principal plate boundary between the North American and Pacific plates. According to the U. S. Geological Survey this is an area with a high level of historical seismicity, and it has recently also been seismically active, though this is the largest event to strike in this area since 1892. The April 4, 2010, earthquake appears to have been larger than the M6.9 earthquake in 1940 or any of the early 20th century events (e.g., 1915 and 1934) in this region of northern Baja California. The event caused widespread damage to structures, closure of businesses, government offices and schools, power outages, displacement of people from their homes and injuries in the nearby major metropolitan areas of Mexicali in Mexico and Calexico in Southern California.

This event's aftershock zone extends significantly to the northwest, overlapping with the portion of the fault system that is thought to have ruptured in 1892. Some structures in the San Diego area experienced minor damage and there were some injuries. Ground motions for the April 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 and south of the original event, including within San Diego County, closer to the San Diego metropolitan area. There have been hundreds of these earthquakes including events up to M5.7.

On July 7, 2010, a M5.4 earthquake occurred in Southern California at 4:53 pm (Pacific Time) about 30 miles south of Palm Springs, 25 miles southwest of Indio, and 13 miles north-northwest of Borrego Springs. The earthquake occurred near the Coyote Creek segment of the San Jacinto Fault. The earthquake exhibited right lateral slip to the northwest, consistent with the direction of movement on the San Jacinto Fault. The earthquake was felt throughout Southern California, with strong shaking near the epicenter. It was followed by more than 60 aftershocks of M1.3 and greater during the first hour. Seismologists expect continued aftershock activity.

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

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 Quaternary-age Old Paralic Deposits (Qop_6) that are bordered to the west by Quaternary-age Young Alluvium (Qya) formational materials. The formational soils are overlain by approximately 3 feet of fill soils across the lot (refer to the boring logs, Figure Nos. IIIa-b). Figure No. V presents a plan view geologic map (Kennedy and Tan, 2008) of the general area of the site and Figure No. VI displays the geologic hazards of the area.

Fill Soils (Qaf): The lot is overlain by approximately 3 feet of fill soils. The fill soils encountered in both boring locations, consist of dark brown silty sand with some roots. The encountered fill soils were generally loose to medium dense, damp and are considered to have a low expansion potential. Refer to Figure No. III.

Old Paralic Deposits (Qop_6): The encountered formational materials consist of medium dense to dense, damp, reddish brown and grayish brown to orange, silty sand. The formational soils were encountered at a depth of approximately 3 feet in both borings. The formational soils are considered to have a low expansion potential and bearing strength increasing from low to high within the upper 3 feet. Refer to Figure No. III.

VII. GEOLOGIC HAZARDS

A review of the City of San Diego Seismic Safety Study, Geologic Hazards Map Sheet No. 30, indicates that the site is located in a low risk geologic hazard area designated as Category 52. Category 52 is identified as being underlain by "*Other level areas, gently sloping to steep terrain, favorable geologic structure, low risk.*" An excerpted portion of the Geologic Hazards Map Sheet 30 and the legend are presented as Figure No. VI.

The following is a discussion of the geologic conditions and hazards common to this area of the City 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, Figure No. V (Kennedy and Tan, 2008), and the City of San Diego Seismic Safety Study, Geologic Hazards Map No. 30, Figure No. VI, 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 0.5-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 13 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 20 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 37 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).

San Jacinto Fault: The San Jacinto Fault is located 59 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 were to take place on a local fault, an estimated surface-rupture length 1 mile long could be expected (Greensfelder, 1974). Our investigation indicates that the subject site is not directly on a known active fault trace and, therefore, the risk of ground rupture is remote.

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 significant soil strength loss to occur due to a seismic event.

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

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

Historical wave heights and run-up elevations from tsunamis that have impacted the San Diego Coast have historically fallen within the normal range of the tides (Joy, 1968). The risk of a tsunami affecting the site is considered moderate as the site is situated at an elevation of approximately 25 feet above mean sea level and approximately 600 feet to an exposed beach. The site is not mapped within a possible inundation zone on the California Geological Survey's 2009 "*Tsunami*

Inundation Map for Emergency Planning, La Jolla Quadrangle, San Diego County," however, the inundation zone is mapped approximately 200 feet to the west of the subject site.

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 for the proposed residential remodel and associated improvements provided the recommendations herein are implemented.

No significant geologic hazards are known to exist on the site that would prevent the proposed construction. 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.

VIII. GROUNDWATER

No groundwater was encountered during the course of our field investigation and we do not anticipate significant groundwater problems to develop in the future, ***if the property is developed as proposed and proper drainage is implemented and maintained***. The true groundwater surface is assumed to be at a depth of over 25 feet below the existing and planned building pads. Based on exploratory drilling throughout San Diego County, we would expect minor seeps between the ground surface and true water table due to transient *"perching"* of vadose water on exceptionally dense, low permeability beds within the formational materials.

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

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

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

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

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

IX. CONCLUSIONS AND RECOMMENDATIONS

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

> *If the geotechnical consultant of record is changed for the project, the work shall be stopped until the replacement has agreed in writing to accept the responsibility within their area of technical competence for approval upon completion of the work. It shall be the responsibility of the permittee to notify the 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 residence 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 32.8593 degrees and longitude of -117.2542 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 experience with similar soil conditions, we have assigned a Site Soil Classification of D. Refer to the *"USGS Design Maps Summary Report"* presented as Appendix B.

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}
1.294g	0.502g	1.000	1.500	1.294g	0.753g	0.863g	0.502g

B. *Preparation of Soils for Site Development*

2. *Clearing and Stripping:* The existing parts of the structure to be demolished, and vegetation on the lot should be removed prior to the preparation of the building pad and areas to receive associated improvements. This includes any roots from existing trees and shrubbery. Holes resulting from the removal of root systems or other buried obstructions that extend below the planned grades should be cleared and backfilled with properly compacted fill.

3. *Building Pad Surface and Subgrade Preparation:* After the building pad has been cleared, stripped, and the required excavations made to remove the existing loose or disturbed surface fill, at least the upper 3 feet of pad fill soils should be removed and recompacted. The bottom of the excavation should be extended to expose consist of medium dense to dense old paralic deposit soils. The bottom of the excavation should be scarified to a depth of 6 inches, moisture conditioned, and compacted to the requirements for structural fill.

4. *Material for Fill:* Existing on-site soils with an organic content of less than 3 percent by volume are, in general, suitable for use as fill. Imported fill material, where required, should have a low-expansion potential (Expansion Index of 50 or less per ASTM D4829-11). In addition, both imported and existing on-site materials for use as fill should not contain rocks or lumps more than 6 inches in greatest dimension if the fill soils are compacted with heavy compaction equipment (or 3 inches in greatest dimension if compacted with lightweight equipment). All materials for use as fill should be approved by our representative prior to importing to the site.

5. *Expansive Soil Conditions:* We do not anticipate that expansive soils will be encountered during grading. Should such on-site soils be used as fill, they should be moisture conditioned to at least 5 percent above optimum moisture content, compacted to 88 to 92 percent. Soils of medium or greater expansion potential should not be used as retaining wall backfill soils. If basement slabs are placed directly on medium expansive formational materials, the moisture content of the soil should be verified to be at least 3 percent above optimum, or scarification and moisture conditioning will be required.

6. *Fill Compaction:* All structural fill should be compacted to a minimum degree of compaction of 90 percent based upon ASTM D1557-12. Fill material should be spread and compacted in uniform horizontal lifts not exceeding 8 inches in uncompacted thickness. Before compaction begins, the fill should be brought to a water content that will permit proper compaction by either: (1) aerating and drying the fill if it is too wet, or (2) moistening the fill with water if it is too dry. Each lift should be thoroughly mixed before compaction to ensure a uniform distribution of moisture. For low expansive soils, the moisture content should be within 2 percent of optimum. For medium to highly expansive soils, the moisture content should be at least 5 percent over optimum. Once placed, soil moisture content of the fill soils should be maintained by sprinkling daily. Medium to highly expansive soils should be compacted to between 88 and 92 percent of Maximum Dry Density.

   The areal extent required to remove the surficial soils should be confirmed by our representatives during the excavation work based on their examination of the soils being exposed. The lateral extent of the excavation and recompaction should be at least 5 feet beyond the edge of the perimeter ground level foundations of the new residential additions and any areas to receive exterior improvements where feasible.

   If heavy compaction equipment is utilized, oversize material more than 6 inches in diameter should be removed from the fill. If lightweight compaction equipment is used, oversize material more than 3 inches in diameter should be removed.

Any rigid improvements founded on the existing surface soils can be expected to undergo movement and possible damage. Existing footings to support new second story loads should be reviewed by the structural engineer for evaluation of the new loads applied with the allowable bearing capacity of 1500 psf. If this bearing capacity is not sufficient, the existing footings should be widened as needed of deepened to penetrate into dense formational soils. ***Geotechnical Exploration, Inc.*** takes no responsibility for the performance of any improvements built on loose natural soils or inadequately compacted fills. Subgrade soils in any exterior area receiving concrete improvements should be verified for compaction and moisture within 48 hours prior to concrete placement.

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

7. *Trench Backfill:* New utility trenches should be backfilled with imported low-expansive compacted fill; gravel is also a suitable backfill material but should be used only if space constraints will not allow the use of compaction equipment. Gravel can also be used as backfill around perforated subdrains. All backfill material should be placed in lift thicknesses appropriate to the type of compaction equipment utilized and compacted to a minimum degree of compaction of 90 percent by mechanical means.

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

C. Design Parameters for Proposed Foundations

8. *Deepened Footings*: If the existing surface soils are not removed and recompacted, deepened new footings for proposed structures should be founded at least 3 feet below the lowest adjacent finished grade and penetrate at least 12 inches in dense formational soils and have a minimum width of 15 inches. The deepened footings should contain top and bottom reinforcement to provide structural continuity and to permit spanning of local irregularities. The final dimensions and reinforcing should be specified by the structural engineer. A minimum clearance of 3 inches should be maintained between steel reinforcement and the bottom or sides of the footing. If deepened footings are used, the new floor slabs should be designed to structurally span the distance between foundations. If existing footings are designed to carry new loads, they should be reviewed by the structural engineer as discussed in section 6.

   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.

9. *Shallow Footings*: New shallow footings for new structures or improvements should bear on undisturbed formational materials or properly compacted fill soils. The footings should be founded at least 18 inches below the lowest adjacent finished grade when founded into properly compacted fill (or 12 inches into formational material). Footings located adjacent to utility trenches should have their bearing surfaces situated below an imaginary 1.5:1.0 plane projected upward from the bottom edge of the adjacent utility trench.

10. *Bearing Values*: At the recommended depths, footings on native, medium 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 increased one-third for all loads, including wind or seismic. The footings should have a minimum width of 12 inches.

11. *Footing Reinforcement*: All continuous footings should contain top and bottom reinforcement to provide structural continuity and to permit spanning of local irregularities. We recommend that a minimum of two No. 5 top and two No. 5 bottom reinforcing bars be provided in the footings. Footings over 18 inches in depth should be reinforced as specified by the structural engineer. A minimum clearance of 3 inches should be maintained between steel reinforcement and the bottom or sides of the footing. Isolated square footings should contain, as a minimum, a grid of three No. 4 steel bars on 12-inch centers, both ways. In order for us to offer an opinion as to whether the footings are founded on soils of sufficient load bearing capacity, it is essential that our representative inspect the footing excavations prior to the placement of reinforcing steel or concrete.

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

12. *Lateral Loads*: Lateral load resistance for structure foundations may be developed in friction between the foundation bottoms and the supporting subgrade. An allowable friction coefficient of 0.40 is considered applicable. An additional allowable passive resistance equal to an equivalent fluid weight of 300 pounds per cubic foot acting against the new foundations may be used in design provided the footings are poured neat against the adjacent undisturbed formational materials and/or properly compacted fill materials. In areas where existing loose fill soils are present in front of existing or new foundations (a horizontal distance equal to 3 times the depth of embedment), the allowable passive resistance should be reduced to 220 pcf and friction coefficient to 0.35. 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.

13. *Settlement*: Settlements under foundations with building loads that comply with our recommendations are expected to be within tolerable limits for the proposed additions. For footings designed in accordance with the recommendations presented in the preceding paragraphs, we anticipate that total settlements should not exceed 1 inch and that post-construction differential angular rotation should be less than 1/240.

D. Concrete Slab On-Grade Criteria

Slabs on-grade may only be used on new, properly compacted fill or when bearing on dense natural soils. Existing undamaged slabs may remain, if no new loads are to be supported.

14. *Minimum Floor Slab Reinforcement*: Based on our experience, we have found that, for various reasons, floor slabs occasionally crack. Therefore, we recommend that all slabs on-grade contain at least a minimum amount of reinforcing steel to reduce the separation of cracks, should they occur.

    New interior floor slabs should be a minimum of 5 inches actual thickness and be reinforced with No. 4 bars on 18-inch centers, both ways, placed at midheight in the slab. ***The slabs should be underlain by a moisture retardant membrane such as StegoWrap 15-mil, on a properly compacted subgrade or a 4-inch-thick base layer (or as indicated by the membrane manufacturer).*** Soil moisture content should be kept above the optimum prior to moisture barrier or waterproofing placement under the new concrete 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.

15. *Slab Moisture Emission:* Although it is not the responsibility of geotechnical engineering firms to provide moisture protection recommendations, as a service to our clients we provide the following discussion and suggested

minimum protection criteria. Actual recommendations should be provided by the Project Architect and waterproofing consultants or product manufacturer.

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

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

The following American Society for Testing and Materials (ASTM) and American Concrete Institute (ACI) sections address the issue of moisture transmission into and through concrete slabs: ASTM E1745-97 (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.

15.1 Based on the above, we recommend that the vapor barrier consist of a minimum 15-mil extruded polyolefin plastic (no recycled content or woven materials permitted). Permeance as tested before and after mandatory conditioning (ASTM E1745 Section 7.1 and subparagraphs 7.1.1-7.1.5) should be less than 0.01 perms (grains/square foot/hour/per inch of Mercury) and comply with the ASTM E1745 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.

15.2 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.

15.3 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. Water-proofing 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.

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

16. *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. Structural slabs should not be provided with control joints.

17. *Exterior Slab Reinforcement*: Exterior concrete slabs should be at least 4 inches thick. As a minimum for protection of on-site improvements, we recommend that all nonstructural concrete slabs (such as patios, sidewalks, etc.), be founded on properly compacted and tested fill or dense native formation and be underlain by 2 inches and no more than 3 inches of clean leveling sand, with No. 3 bars at 18-inch centers, both ways, at the center of the slab. Exterior slabs should 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. The subgrade of exterior improvements should be verified as properly prepared within 48 hours prior to concrete placement. A minimum thickness of 3 feet of properly recompacted soils should underlie the exterior slabs on-grade or they should be constructed on dense formational soils.

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 and isolation joints in exterior slabs should be sealed with elastomeric joint sealant. The sealant should be inspected every 6 months and be properly maintained.

E. ***Site Drainage Considerations***

18. *Erosion Control*: Appropriate erosion control measures should be taken at all times during and after construction to prevent surface runoff waters from entering footing excavations or ponding on finished building pad areas.

19. *Surface Drainage:* Adequate measures should be taken to properly finish-grade the lot after the structures and other improvements are in place. Drainage waters from this site and adjacent properties should be directed away from the footings, floor slabs, and slopes, onto the natural drainage direction for this area or into properly designed and approved drainage facilities provided by the project civil engineer. Roof gutters and downspouts

should be installed on the residence, with the runoff directed away from the foundations via closed drainage lines. Proper subsurface and surface drainage will help minimize the potential for waters to seek the level of the bearing soils under the footings and floor slabs.

Failure to observe this recommendation could result in undermining and possible differential settlement of the structure or other improvements on the site or cause other moisture-related problems. Currently, the CBC requires a minimum 1-percent surface gradient for proper drainage of building pads unless waived by the building official. Concrete pavement may have a minimum gradient of 0.5-percent.

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

F. General Recommendations

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

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

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

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 placement and compaction of any fill or backfill soils during site grading work must be observed and tested by the soil engineer.

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

XI. LIMITATIONS

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

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

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

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

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

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 grading and structural plans. We should be retained to review the project plans once they are available, to verify that our recommendations are adequately incorporated in the plans. Additional or modified recommendations may be issued if warranted after plan review.

This firm does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and we cannot be responsible for the safety of personnel other than our own 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. 17-11479** will expedite a reply to your inquiries.

Respectfully submitted,

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JOB NO. 17-11479
June 2017

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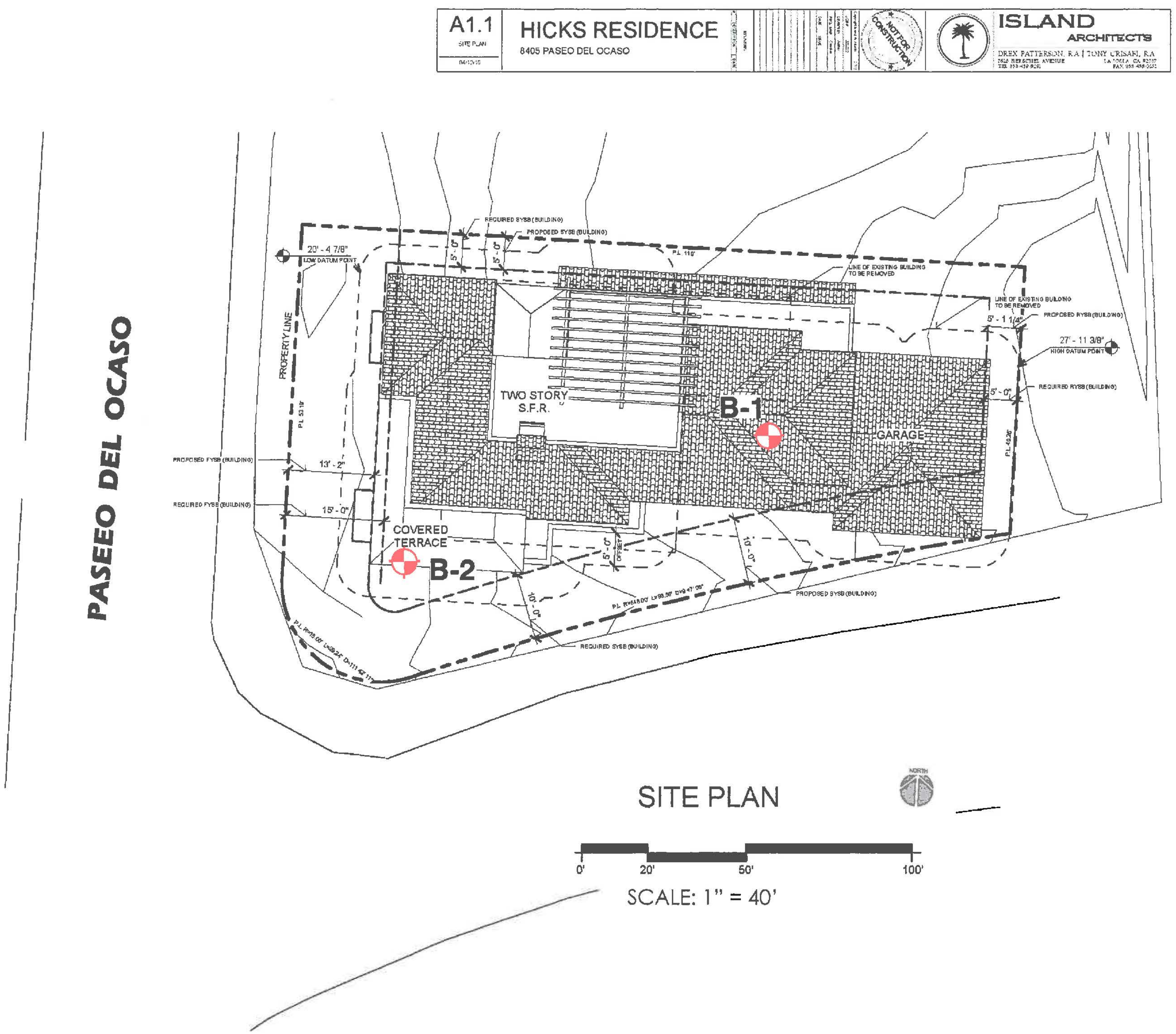
VICINITY MAP

Hicks Residence Remodel
8405 Paseo Del Ocaso
La Jolla, CA.

Figure No. I
Job No. 17-11479



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Sheet Title:
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Revision 14:
Revision 13:
Revision 12:
Revision 11:
Revision 10:
Revision 09:
Revision 08:
Revision 07:
Revision 06:
Revision 05:
Revision 04:
Revision 03:
Revision 02:
Revision 01:

Original Date: 08/01/2012
Sheet 1 of: 10
DEP#:

Area Legend

	HARDSCAPE	2896 SQ. FT.	XX%
	BUILDING FOOTPRINT	YY SQ. FT.	XX%
	LANDSCAPE	XX SQ. FT.	XX%
	LOT TOTAL	SQ. FT.	100%

LEGEND

B-2 Approximate Location of Exploratory Boring

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.

REFERENCE: This Plot Plan was prepared from an existing SITE PLAN provided by ISLAND ARCHITECTS dated 04/12/16 and from on-site field reconnaissance performed by GEI.

PLOT PLAN

Hicks Residential Remodel
8405 Paseo Del Ocaso
La Jolla, CA.
Figure No. II
Job No. 17-11479

Geotechnical Exploration, Inc.

June 2017

17-11479-p.ai

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Limited Access Auger Drill Rig	**6-inch diameter Boring**	**4-27-17**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 26' Mean Sea Level	**Not Encountered**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION — DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
			SILTY SAND , fine- to medium-grained, with some roots. Loose to medium dense. Damp. Dark brown.	SM								
2			**FILL (Qaf)**									
			SILTY SAND , fine- to medium-grained; poorly cemented. Medium dense. Damp. Red-brown.	SM	6.4	117.6					16	3"
4			**OLD PARALIC DEPOSITS (Qop $_6$)**								8	2"
6			-- 13% passing #200 sieve.				8.5	128.5				
8					4.5	113.5					26	3"
											25	2"
10												
			-- becomes dense and fine- to coarse-grained.									
12												
14					5.3	105.8					63	3"
											50+	2"
16			Bottom @ 15'									

EXPLORATION LOG 11479 HICKS.GPJ GEO_EXPL.GDT 5/19/17

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

JOB NAME		
Hicks Residence Remodel		
SITE LOCATION		
8405 Paseo del Ocaso, La Jolla, CA		
JOB NUMBER: **17-11479**	REVIEWED BY: **LDR/JAC**	LOG No.
FIGURE NUMBER: **IIIa**	Geotechnical Exploration, Inc.	**B-1**

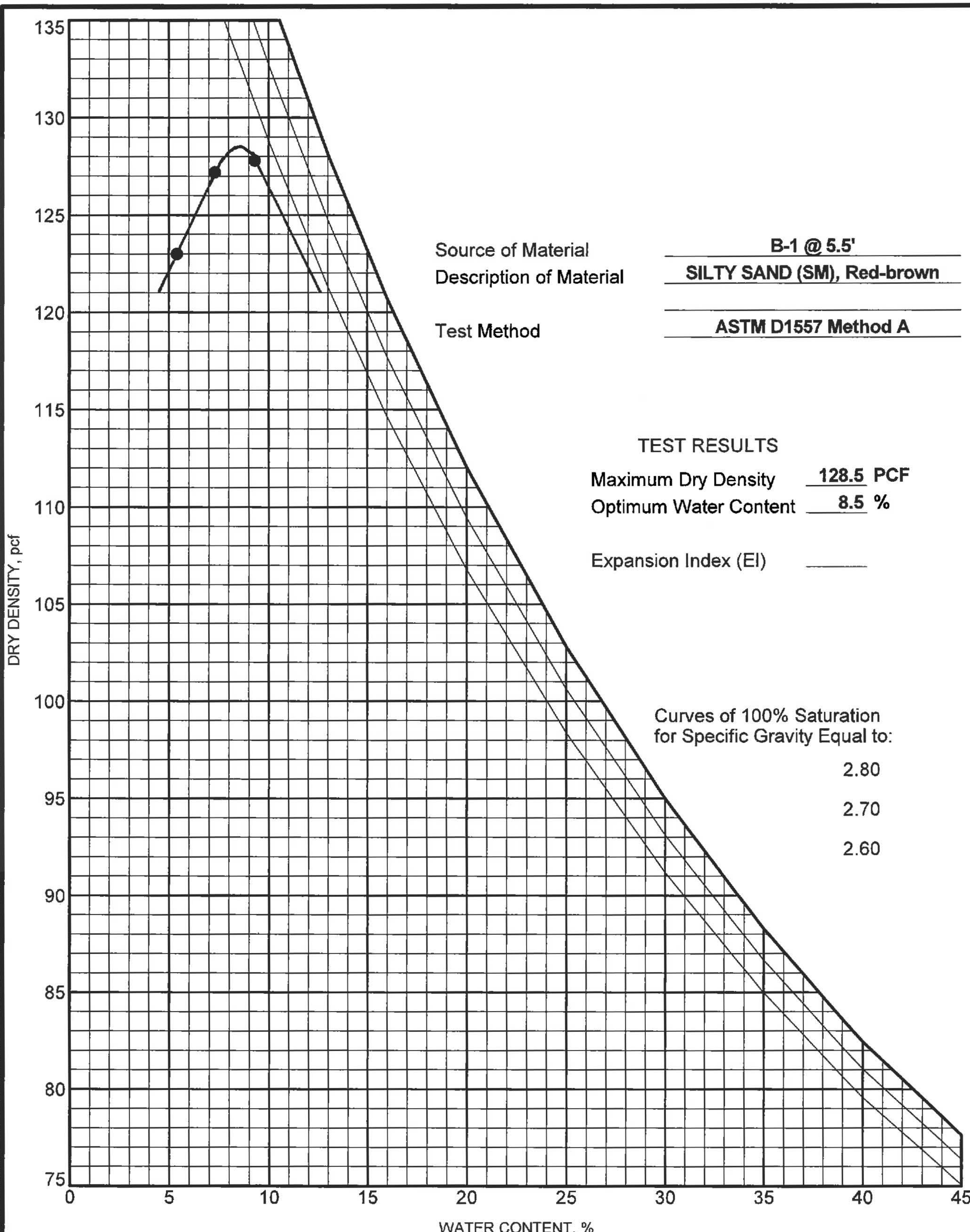
EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Limited Access Auger Drill Rig	**6-inch diameter Boring**	**4-27-17**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 22' Mean Sea Level	**Not Encountered**	**JKH**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–3			**SILTY SAND**, fine- to medium-grained, with some roots. Loose to medium dense. Damp. Dark brown. **FILL (Qaf)**	SM								
					7.9	118.2					9	3"
3–9			**SILTY SAND**, fine- to medium-grained; poorly to moderately cemented. Medium dense to dense. Damp. Red-brown. **OLD PARALIC DEPOSITS (Qop $_6$)**	SM							9	2"
					15.0	116.9					67+/ 10"	3"
			-- becomes light gray and orange.								50+/ 6"	2"
9–10			Bottom @ 9'									

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

JOB NAME	**Hicks Residence Remodel**
SITE LOCATION	**8405 Paseo del Ocaso, La Jolla, CA**
JOB NUMBER	**17-11479**
REVIEWED BY	**LDR/JAC**
FIGURE NUMBER	**IIIb**
LOG No.	**B-2**

Geotechnical Exploration, Inc.

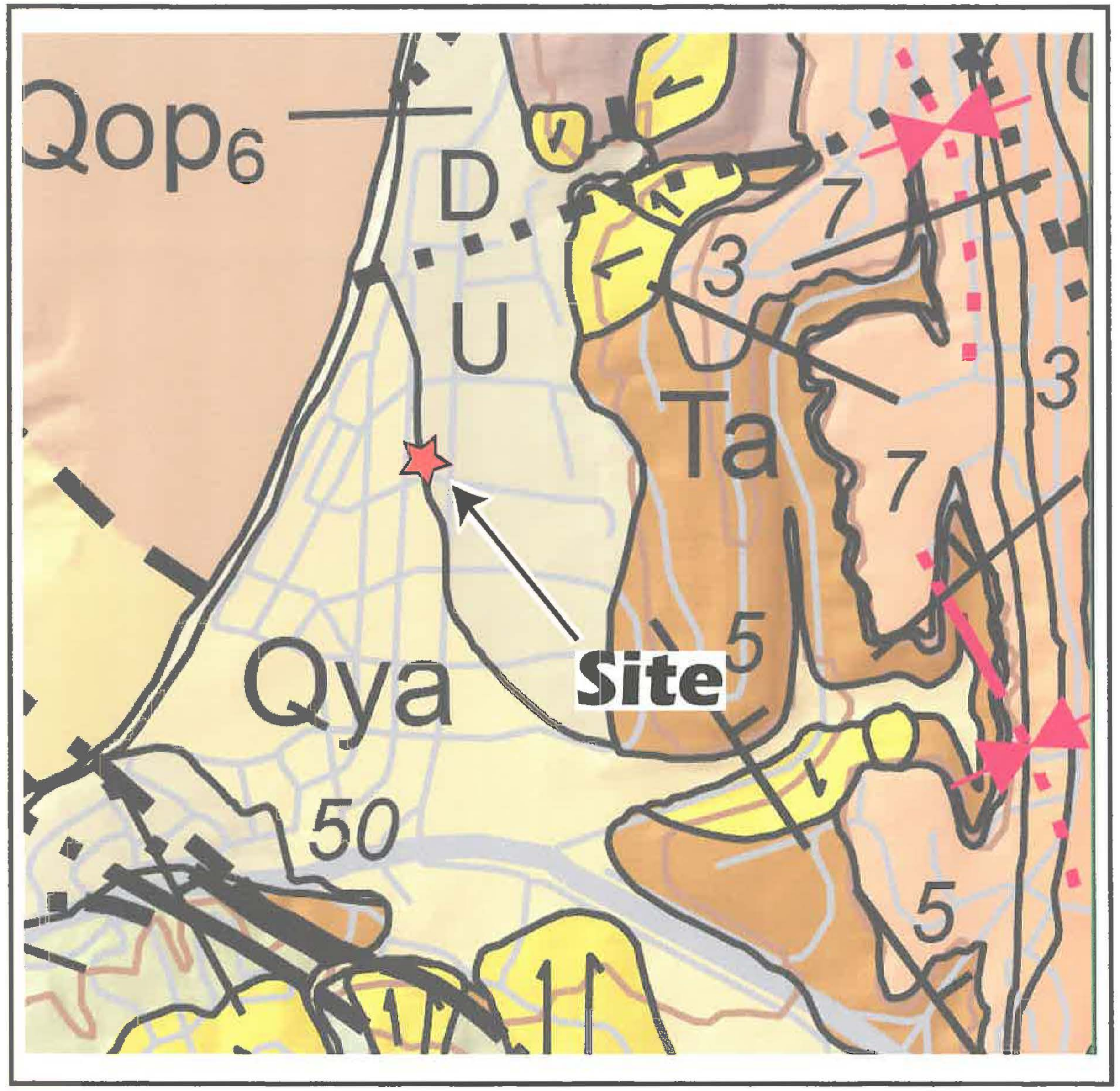


Geotechnical Exploration, Inc.

MOISTURE-DENSITY RELATIONSHIP

Figure Number: IV
Job Name: Hicks Residence Remodel
Site Location: 8405 Paseo del Ocaso, La Jolla, CA
Job Number: 17-11479

COMPACTION + EI DARK GRID 11479 HICKS.GPJ GEI FEB06.GDT 5/19/17



Hicks Residence Remodel
8405 Paseo Del Ocaso
La Jolla, CA.

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

By
Michael P. Kennedy[1] and Siang S. Tan[1]
2008
Digital preparation by
Kelly R. Bovard[2], Anne G. Garcia[2], Diane Burns[2], and Carlos I. Gutierrez[1]
1. Department of Conservation California Geological Survey
2. U.S. Geological Survey, Department of Earth Sciences, University of California, Riverside

ONSHORE MAP SYMBOLS

Contact - Contact between geologic units; dotted where concealed.

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

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

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

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

Strike and dip of beds

70 Inclined

Strike and dip of igneous joints

60 Inclined

Vertical

Strike and dip of metamorphic foliation

55 Inclined

DESCRIPTION OF MAP UNITS

Qop6 — Unit 6 — Old paralic deposits, undivided (late to middle Pleistocene)

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.

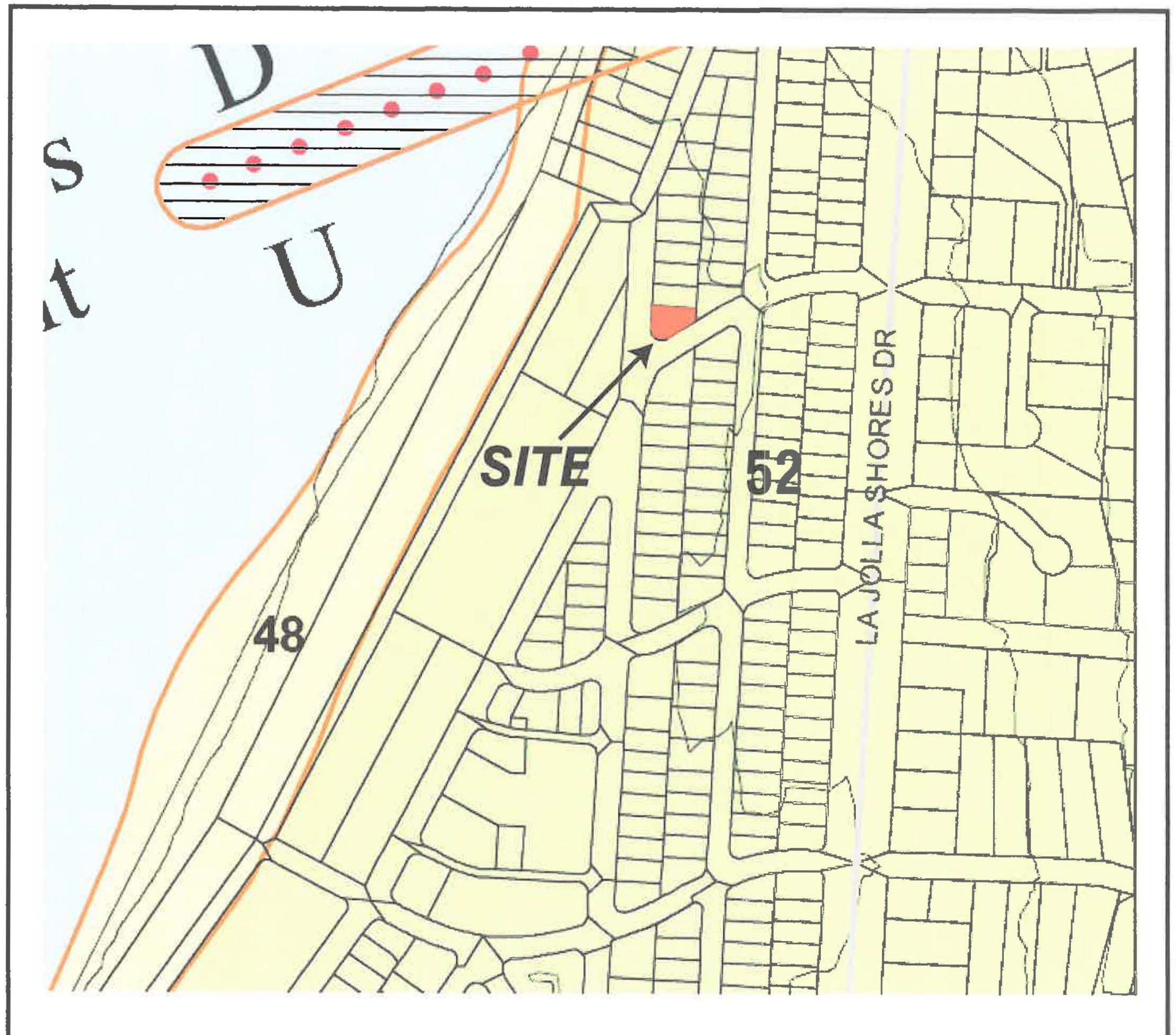
Hicks-Res-combo-2008-geo.ai

Figure No. V
Job No. 17-11479
Geotechnical Exploration, Inc.
June 2017

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

Development Services Department

DATE: 4/3/2008



Hicks Residence Remodel
8405 Paseo Del Ocaso
La Jolla, CA.

LEGEND

Geologic Hazard Categories

FAULT ZONES

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

LANDSLIDES

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

SLIDE-PRONE FORMATIONS

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

LIQUEFACTION

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

COASTAL BLUFFS

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

OTHER TERRAIN

- 51 Level mesas -- underlain by terrace deposits and bedrock 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. 17-1147

Geotechnical Exploration, Inc.

June 2017

Hicks-seis-combo.ai

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)

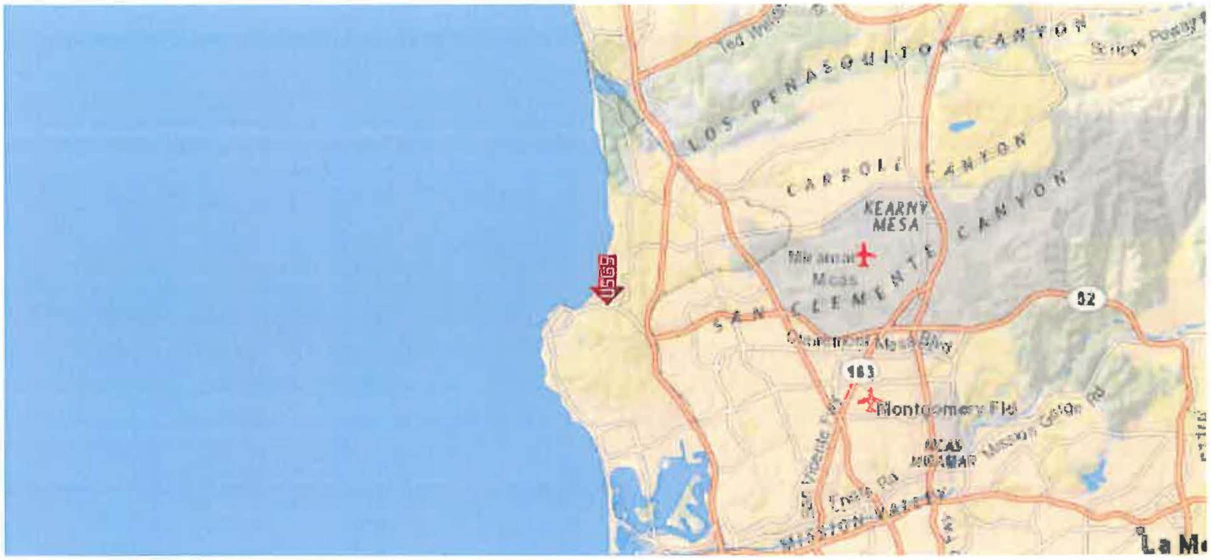
APPENDIX B

USGS DESIGN MAPS SUMMARY REPORT

USGS Design Maps Summary Report

User–Specified Input

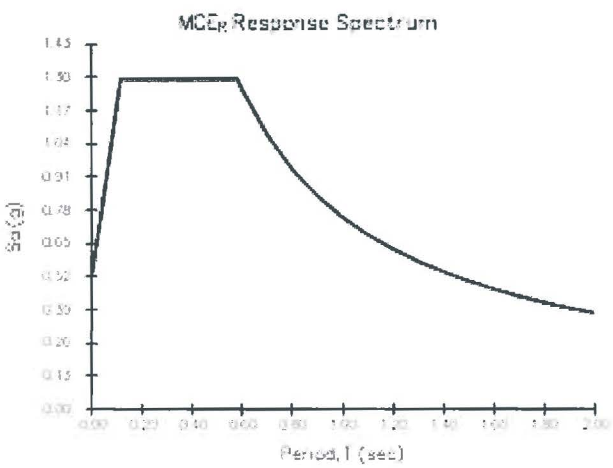
Report Title	Hicks Residence Thu June 15, 2017 22:02:51 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	32.8593°N, 117.2542°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

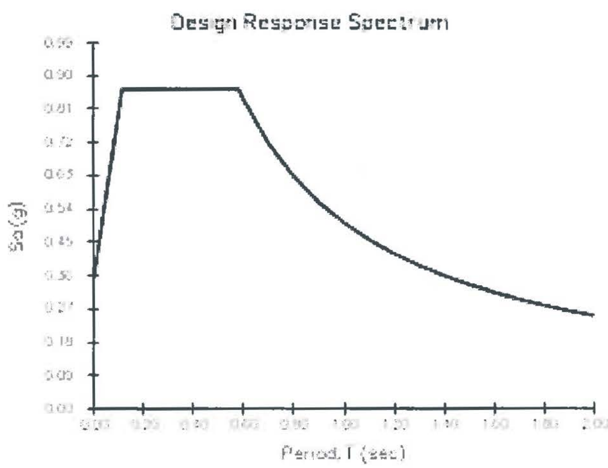


USGS–Provided Output

$S_S =$	1.294 g	$S_{MS} =$	1.294 g	$S_{DS} =$	0.863 g
$S_1 =$	0.502 g	$S_{M1} =$	0.753 g	$S_{D1} =$	0.502 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.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (32.8593°N, 117.2542°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 = 1.294 g
From Figure 22-2 [2]	S_1 = 0.502 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: • Plasticity index PI > 20, • Moisture content $w \geq 40\%$, and • 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				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
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 = 1.294 g, F_a = 1.000

Table 11.4-2: Site Coefficient F_V

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1-s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
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 = 0.502 g, F_v = 1.500

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 1.294 = 1.294$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.500 \times 0.502 = 0.753$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

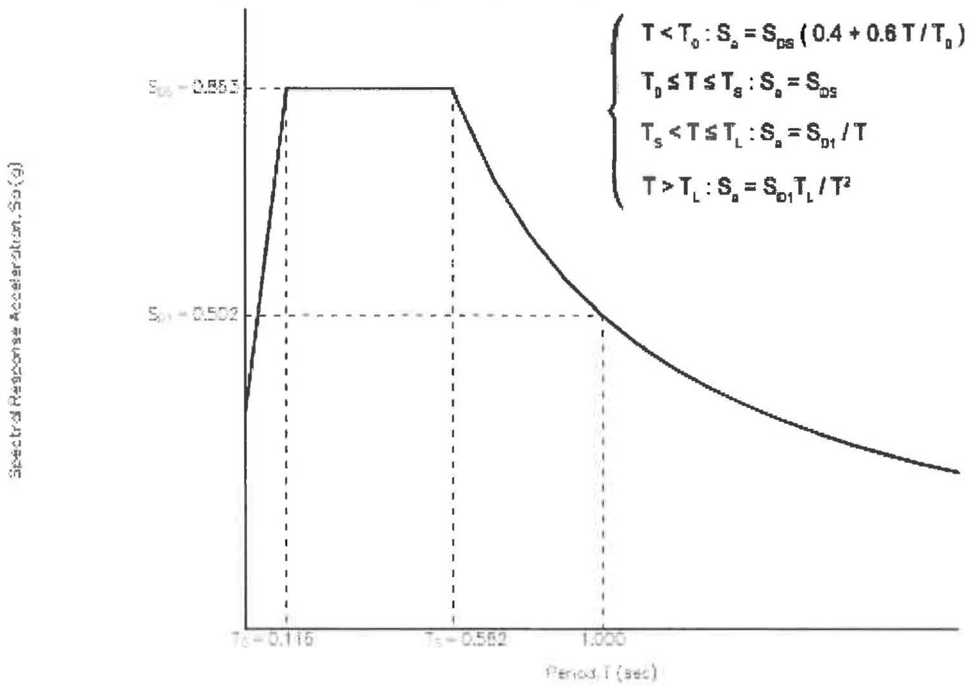
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 1.294 = 0.863$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 0.753 = 0.502$ g

Section 11.4.5 — Design Response Spectrum

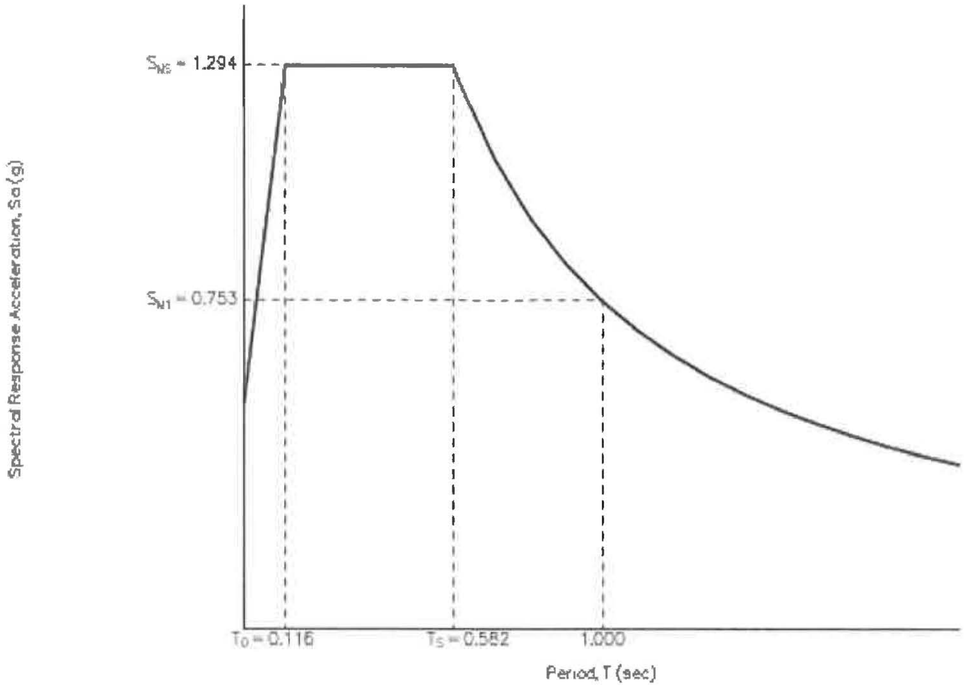
From Figure 22-12 [3] $T_L = 8$ seconds

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 = 0.586

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.586 = 0.586$ 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 = 0.586 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} = 0.836

From Figure 22-18 [6] C_{R1} = 0.870

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
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.863 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
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.502 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

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