REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION

Davidson Residential Development
811-827 Coast Boulevard South
La Jolla, California

JOB NO. 20-12787
24 September 2020

Prepared for:

Ms. Dawn Davidson

Geotechnical Exploration, Inc.
SOIL AND FOUNDATION ENGINEERING ● GROUNDWATER ● ENGINEERING GEOLOGY

24 September 2020

Job No. 20-12787

Ms. Dawn Davidson
c/o Will and Fotsch Architects
1298 Prospect Street, Suite 2S
La Jolla, CA 92037

Subject: **Report of Preliminary Geotechnical Investigation**
Davidson Residential Development
811-827 Coast Boulevard South
La Jolla, California

Dear Ms. Davidson:

In accordance with our proposal dated June 08, 2020, ***Geotechnical Exploration, Inc.*** has performed a preliminary geotechnical investigation for the subject project in La Jolla, California. The field work was performed on June 25, 2020.

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

This opportunity to be of service is sincerely appreciated. Should you have any questions concerning the following report, please do not hesitate to contact us. Reference to our **Job No. 20-12787** will expedite a response to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

Jaime A. Cerros, P.E.
R.C.E. 34422/G.E. 2007
Senior Geotechnical Engineer

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

7420 TRADE STREET ● SAN DIEGO, CA. 92121 ● (858) 549-7222 ● FAX: (858) 549-1604 ● EMAIL: geotech@gei-sd.com

TABLE OF CONTENTS

REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION

Davidson Residential Development
811-827 Coast Boulevard South
La Jolla, California

JOB NO. 20-12787

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 our communications with your architect, Andy Fotsch of Will and Fotsch Architects, and review of architectural plan set, dated August 21, 2020, that the single-family residential structures currently addressed as 813, 815, 817 and 819 Coast Boulevard will be demolished.

The upper level of the structure addressed as 821 Coast Boulevard South and the foundation for the structure at 827 Coast Boulevard South will be demolished. The structure currently addressed as 827 Coast Boulevard South will be moved to the upper level of the structure at 821 Coast Boulevard South. The existing foundation for the structure at 827 Coast Boulevard South will be demolished.

A multi-story, multi-family residential apartment structure over parking garages will be developed in the southeast portion of the site. The three detached, single-family residential structures currently addressed as 811, 821 and 825 will be remodeled. The entire site will receive associated improvements. The new structures are to be constructed of standard-type building materials utilizing conventional shallow foundations with either concrete slabs on-grade or raised wood floors. Foundation loads are expected to be typical for this type of relatively light to moderate weight construction.

Based on our understanding of the proposed construction, excavations for the parking level garage basement in the multi-family residential structure will require shoring to prevent collapse of the walls of the excavation and destabilization of adjacent properties. Furthermore, it is our explicit opinion that the proposed site development would not destabilize neighboring properties or induce the settlement of adjacent structures if designed and constructed in accordance with our recommendations.

When final architectural and engineering plans have been prepared, 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 under the direction of our geologist with the placement, logging and sampling of five exploratory borings and one hand auger boring, review of available published information pertaining to the site geology, laboratory testing of sampled soils, 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, shoring walls and slab on-grade floors.

III. SITE DESCRIPTION

The subject site consists of multiple lots addressed as 811-827 Coast Boulevard South, and is known as Assessor's Parcel Nos. 350-070-10-00 and 350-070-11-00, Lots 9, 10 and 11, Block 55, per Recorded Map No. 352, in the La Jolla region of the

City and County of San Diego, State of California. Refer to Figure No. I, the Vicinity Map, for the site location.

The roughly rectangular-shaped site is approximately 0.45-acre. The site is bordered on the southwest by three single-family residences at similar elevations; on the northeast by a multi-family residential apartment structure at similar elevations; on the southeast by an alley at higher elevation; and on the northwest by Coast Boulevard South at a lower elevation. The site is located on a gently sloping 4:1 (horizontal:vertical) hillside descending to the northwest, with the natural drainage trending in the same direction.

The site is currently developed with eight detached single-family residential structures, with landscaping and associated improvements surrounding each individual structure. The three northwestern structures with numbered addresses 811, 821 and 825 have a split-level design, with the main level situated over lower level parking garages and concrete driveways, which provide access to the site. Access is also available by an alley in the rear of the site.

Elevations across the site range from approximately 63 feet above mean sea level (MSL) along the northwestern property line, to approximately 93 feet above MSL along the southeastern property line. Information concerning approximate elevations across the site was obtained from Google Earth Imagery and the Existing Exterior Elevation sheets in the architectural plan set by Will and Fotsch architects, dated August 21, 2020. Refer to the Plot Plan with Site-Specific Geology (Figure No. IIa) and Geologic Cross Sections A-A' and B-B' (Figure Nos. IIb-c) for topographic and locational information.

Vegetation on the site consists primarily of ornamental shrubbery and some mature trees. Recycled rubber and mulch are spread across much of the site in landscaping areas.

IV. FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program utilizing a limited-access tripod drill rig with a continuous flight solid stem auger and a hand auger to investigate and sample the subsurface soils on June 25, 2020. Five exploratory borings (B-1 to B-5) and one hand auger boring (HA-1) were advanced around the existing residential structures and in the vicinity of the proposed development and associated improvements. The exploratory borings were advanced to depths ranging from 8 to 21.5 feet in order to obtain representative soil samples and to define the soil profile across the project area. The soils encountered in the exploratory borings were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (refer to Appendix A). The approximate locations of the exploratory borings are shown on the Plot Plan, Figure No. IIa.

Representative samples were obtained from the exploratory borings at selected depths appropriate to the investigation. Sampling consisted of a 140-pound hammer falling 30 inches onto a 2-inch outer diameter Standard Penetration Test (SPT) split-spoon sampler (ASTM D1586-18) and a 3-inch outer diameter ring-lined Modified California split-tube sampler (ASTM D3550-17). 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 2-inch O.D. Standard Penetration Test and the 3-inch O.D. Modified California 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 Dense	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

Bulk samples were also collected from the exploratory borings to aid in classification and for appropriate laboratory testing. 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, and are attached as Figure Nos. IIIa-f.

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

V. <u>LABORATORY TESTS AND SOIL INFORMATION</u>

Laboratory tests were performed on disturbed and relatively undisturbed soil samples in order to evaluate their physical and mechanical properties and their ability to support the proposed residential structure and improvements. The test results are presented on Figure Nos. IIIa-f and IV. The following tests were conducted on representative soil samples:

1. *Moisture Content (ASTM D2216-19)*
2. *Density Measurements (ASTM D2937-17e2)*
3. *Laboratory Compaction Characteristics (ASTM D1557-12e1)*
4. *Determination of Percentage of Particles Smaller than #200 Sieve (ASTM D1140-17)*
5. *Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions (ASTM D3080-11)*

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

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

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

Two direct shear tests (ASTM D3080-11) were performed on undisturbed soil samples in order to evaluate strength characteristics of the formational material. The shear tests were performed with a constant strain rate direct shear machine. The specimens tested were saturated and then sheared under various normal loads. The direct shear test results are presented on Figure No. IV.

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

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

Based on our visual classification and our past experience with similar soils, it is our opinion that the existing fill and formational materials of the Old Paralic Deposits, Unit 7 and Point Loma Formation encountered in the borings possess a very low to low potential for expansion. Therefore, we have assigned a maximum expansion index of less than 50 to these soils.

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 for foundations, retaining walls, and slope stability calculations. Slope stability calculations are included in Appendix C.

VI. REGIONAL GEOLOGIC DESCRIPTION

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

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

The Peninsular Range 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, now the California Geological Survey, an "active" fault is one that has had ground surface displacement within Holocene time, about the last 11,000 years (Hart and Bryant, 1997). 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 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 and Bryant, 1997).

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 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, although 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.

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.

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

VII. SITE-SPECIFIC SOIL & GEOLOGIC DESCRIPTION

Our field investigation, 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 at depth by late to middle Pleistocene-Aged Old Paralic Deposits, Unit 7 (Qop_7) and upper Cretaceous-Aged Point Loma Formation (Kp) formational materials. In the northwestern portion of the site, Old Paralic Deposits overlies the Point Loma Formation. An unconformity exists at the geologic contact, where a significant time gap has occurred between the depositional events of younger Old Paralic Deposits sediments over significantly older Point Loma sediments. The Point Loma Formation underlies the southeastern portion of the site at shallower depth, where the Old Paralic Deposits was not encountered in our investigation. During the course of our field investigation, Old Paralic Deposit formational materials were encountered in five of our exploratory borings, specifically the borings in the northeastern portion of the site. Point Loma formational materials were encountered in four of our exploratory borings. The encountered formational materials are, in general, overlain by approximately 2.5 to 5 feet of artificial fill soils (Qaf). An excerpt of the geological map (Kennedy and Tan, 2008) is included as Figure No. V, Geologic Map and Legend.

A. ***Stratigraphy***

Artificial Fill Soils (Qaf): The entire site is overlain by artificial fill soils that were encountered in all of the exploratory borings. The encountered fill soils were observed to consist of loose to medium dense, fine- to medium-grained silty sand and soft to stiff sandy clay. The fill soils are, in general, slightly moist to moist, light to dark brown to reddish brown. In our opinion, the artificial fill soils are not suitable in their current condition for support of loads from structures or additional fill. The artificial fill soils are considered to have a low expansion potential and the materials are suitable for use as new fill or wall backfill on the site. Refer to Figure Nos. IIIa-f for details.

Old Paralic Deposits, Unit 7 (Qop$_7$): Underlying the artificial fill soils is the late to middle Pleistocene-Aged Old Paralic Deposits, Unit 7 formational materials. These formational materials were encountered in five exploratory borings (B-1 and B-2, B-4 and B-5, and HA-1). These materials were observed to consist of loose to dense, fine-to coarse-grained silty sand, clayey sand and coarse-grained poorly graded sand with silt. They are, in general, slightly moist to wet, brown to reddish brown, olive and yellowish brown and olive yellow.

In our opinion, the Old Paralic Deposits, Unit 7 formational soils at a depth of 8 feet and below from existing grade are suitable in their current condition for support of loads from structures or additional fill. Refer to Figure Nos. IIIa-b and IIId-f for details. According to the aforementioned geologic map, Kennedy and Tan (2008) describe the Old Paralic Deposits, Unit 7 as *"Poorly sorted, moderately permeable, reddish brown, interfingered strandline, beach estuarine and colluvial deposits composed of siltstone, sandstone and conglomerate."*

Point Loma Formation (Kp): Underlying the artificial fill soils, the Old Paralic Deposit, Unit 7, and the entire site is underlain at depth by the upper Cretaceous-aged Point Loma Formation. The Point Loma Formation was encountered in four exploratory borings (B-2 to B-5). The encountered formational materials were observed to consist of loose, very dense, fine-grained silty sand. Fine-grained clayey sand from 4 to 8½ feet was observed in boring B-3. These formational materials are, in general, slightly moist to moist, bluish gray to olive to olive yellow with reddish brown lenses. The formational materials are considered to have a low expansion potential and have very good bearing strength characteristics and are suitable in their current condition for support of loads from structures or additional fill. Refer to Figure Nos. IIIb-e for details. According to the aforementioned geologic map, Kennedy and Tan (2008) describe the Point Loma Formation as *"Interbedded, fine-grained, dusky-yellow sandstone and olive-gray siltstone."*

B. Structure

The Old Paralic Deposits, Unit 7 (Qop_7), do not contain any visible geologic structure. The paralic deposits, also called marine terrace deposits, form on near horizontal wave-cut benches during sea-level regression and regional uplift. The geologic map by Kennedy and Tan (2008) depicts a relatively level contact between the Old Paralic

Deposits and the underlying Point Loma Formation. Lensed structure was observed in the Point Loma Formation during our subsurface investigation. The Point Loma Formation has been observed by our geologists to contain stratified and laminated structure in outcropping bluff areas within close proximity to the subject site.

Regional geologic structure was corelated by using information obtained in our exploratory borings and the mapped geology of the La Jolla area. The geologic contact between the Old Paralic Deposits and the Point Loma Formation is an unconformity. The basal contact of the Old Paralic Deposits is near horizontal, at an elevation of approximately 52 feet above MSL in the northwestern portion of the site. The inland edge of the marine terrace bench occurs in the eastern area of the site, where the Point Loma formation was observed to be directly underlying the fill soils. For greater understanding of the geologic structure, refer to Figure Nos. IIb-c, the Geologic Cross Sections.

Review of the Point Loma Formation in the geologic map by Kennedy and Tan, 2008, "*Geologic Map of San Diego, 30'x60' Quadrangle, CA*", depicts a strike measurement of N50°E and a dip of 7 degrees southeast, on the outcropping bluff located approximately 400 feet to the northwest of the subject site. The measured strike and dip will produce an apparent dip of the Point Loma Formation bedding at the site of 6 to 7 degrees in to the plane of the slope.

It is our opinion that the strength characteristics of the Old Paralic Deposits, Unit 7 are favorable and suitable for bearing proposed structures and improvements. The near horizontal basal contact of the Old Paralic Deposits is considered favorable in the northwestern portion of the site. In the central portion of the site, from the northeast to southwest property line, the inland edge contact of the Old Paralic Deposits is steeply sloped to near vertical. This could present a soil stability issue and may result in differential settlement of soils with different bearing characteristics under the proposed multi-family apartment structure.

As such, for support of the entire multi-family apartment structure, we recommend either: (1) a combination of caisson and grade beam design with shallow individual-spread and/or continuous footings, deepened sufficiently where necessary to bear structural foundation loads entirely in Point Loma Formation, with a removal and recompaction of the building pad undertaken so garage floor slabs are bearing on uniformly compacted soils, or (2) conventional, individual-spread and/or continuous shallow footing foundations with a full removal and recompaction of the entire multi-family apartment structure pad, including an undercut of Point Loma Formation in the southeastern portion of the building pad. Further details for site preparation, foundation design and slabs on-grade are discussed in the Conclusions and Recommendations section.

Based on the shallow dip angle, the direction of the dipping beds and the Point Loma Formation strength characteristics, it is our opinion that the geologic structure of Point Formation is considered favorable from a slope stability perspective. However, shoring designed and constructed in accordance with our recommendations will be required to make the excavation for the multi-family apartment structure.

VIII. GEOLOGIC HAZARDS

Our review of the City of San Diego Seismic Safety Study -- Geologic Hazards Map Sheet 29, dated 2008, indicates that the site is located in a geologic hazard area designated as Category 53. Category 53 is identified as being underlain by *"Level or sloping terrain, unfavorable geologic structure, low to moderate risk."* The site is located on a gently sloping 4:1 (h:v) hillside descending to the northwest. From our reconnaissance and the data obtained in our field investigation, it is our opinion, in general, that the unfavorable geologic structure does not apply from a hillside stability perspective and the site is low risk. An excerpt from the seismic study sheet 29 is presented as Figure No. VI.

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

A. Local and Regional Faults

Reference to the Geologic Map and Legend, Figure No. IV (Kennedy and Tan, 2008), 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 mapped approximately 2.2 miles east of the 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 since 1769 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 and Bryant, 1997).

Rockwell (2010) has suggested that the RCFZ underwent a cluster of activity including 5 major earthquakes in the early Holocene, with a long period of inactivity following, suggesting major earthquakes on the RCFZ behaves in a cluster-mode, where earthquake recurrence is clustered in time rather than in a consistent recurrence interval. With the most recent earthquake (MRE) nearly 500 years ago, it is suggested that a period of earthquake activity on the RCFZ may have begun. Rockwell (2010) and a compilation of the latest research implies a long-term slip rate of approximately 1 to 2 mm/year.

Coronado Bank Fault: The Coronado Bank Fault is located approximately 11.4 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 et al., 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 et al., 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 offshore portion of the Newport-Inglewood Fault Zone is located approximately 23 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 (Grant Ludwig and Shearer, 2004).

Elsinore Fault: The Elsinore Fault is located approximately 38 to 55 miles east and northeast of the site. The fault extends approximately 200 km (125 miles) from the Mexican border to the northern end of the Santa Ana Mountains. The Elsinore Fault zone is a 1- to 4-mile-wide, northwest-southeast-trending zone of discontinuous and en echelon faults extending through portions of Orange, Riverside, San Diego, and Imperial Counties. Individual faults within the Elsinore Fault Zone range from less than 1 mile to 16 miles in length. The trend, length and geomorphic expression of the Elsinore Fault Zone identify it as being a part of the highly active San Andreas Fault system.

Like the other faults in the San Andreas system, the Elsinore Fault is a transverse fault showing predominantly right-lateral movement. According to Hart et al. (1979), this movement averages less than 1 centimeter per year. Along most of its length, the Elsinore Fault Zone is marked by a bold topographic expression consisting of linearly aligned ridges, swales and hallows. Faulted Holocene alluvial deposits (believed to be less than 11,000 years old) found along several segments of the fault zone suggest that at least part of the zone is currently active.

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

San Jacinto Fault: The San Jacinto Fault is located approximately 61 to 81 miles northeast of the site. The San Jacinto Fault Zone consists of a series of closely spaced faults, including the Coyote Creek Fault, that form the western margin of the San Jacinto Mountains. The fault zone extends from its junction with the San Andreas Fault in San Bernardino, southeasterly toward the Brawley area, where it continues south of the international border as the Imperial Transform Fault (Rockwell et al., 2014).

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

The segments of the San Jacinto Fault that are of most concern to major metropolitan areas are the San Bernardino, San Jacinto Valley and Anza segments. Fault slip rates on the various segments of the San Jacinto are less well constrained than for the San

Andreas Fault, but the available data suggest slip rates of 12 ±6 mm/yr for the northern segments of the fault, and slip rates of 4 ±2 mm/yr for the southern segments. For large ground-rupturing earthquakes on the San Jacinto fault, various investigators have suggested a recurrence interval of 150 to 300 years. The Working Group on California Earthquake Probabilities (2008) has estimated that there is a 31 percent probability that an earthquake of M6.7 or greater will occur within 30 years on this fault. Maximum credible earthquakes of M6.7, M6.9 and M7.2 are expected on the San Bernardino, San Jacinto Valley and Anza segments, respectively, capable of generating peak horizontal ground accelerations of 0.48g to 0.53g in the County of Riverside. A M5.4 earthquake occurred on the San Jacinto Fault on July 7, 2010.

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

> The San Jacinto fault, along with the Elsinore, San Andreas, and other faults, is part of the plate boundary that accommodates about 2 inches/year of motion as the Pacific plate moves northwest relative to the North American plate. The largest recent earthquake on the San Jacinto fault, near this location, the M6.5 1968 Borrego Mountain earthquake April 8, 1968, occurred about 25 miles southeast of the July 7, 2010, M5.4 earthquake.
>
> This M5.4 earthquake follows the 4th of April 2010, Easter Sunday, M7.2 earthquake, located about 125 miles to the south, well south of the US Mexico international border. A M4.9 earthquake occurred in the same area on June 12th at 8:08 pm (Pacific Time). Thus, this section of the San Jacinto fault remains active.
>
> Seismologists are watching two major earthquake faults in southern California. The San Jacinto fault, the most active earthquake fault in southern California, extends for more than 100 miles from the international border into San Bernardino and Riverside, a major metropolitan area often called the Inland Empire. The Elsinore fault is more than 110 miles long, and extends into the Orange County and Los Angeles area as the Whittier fault. The Elsinore fault is capable of a

major earthquake that would significantly affect the large metropolitan areas of southern California. The Elsinore fault has not hosted a major earthquake in more than 100 years. The occurrence of these earthquakes along the San Jacinto fault and continued aftershocks demonstrates that the earthquake activity in the region remains at an elevated level. The San Jacinto fault is known as the most active earthquake fault in southern California. Caltech and USGS seismologists continue to monitor the ongoing earthquake activity using the Caltech/USGS Southern California Seismic Network and a GPS network of more than 100 stations.

B. Other Geologic Hazards

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

Ground Shaking: Structural damage caused by seismically induced ground shaking is a detrimental effect directly related to faulting and earthquake activity. Ground shaking is considered to be the greatest seismic hazard in San Diego County. The intensity of ground shaking is dependent 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 29 and stereo-pair aerial photographs (4-11-53, AXN-8M-89 and 90), there are no known or suspected ancient landslides located on the site.

Slope Stability: A gentle 4:1 (h:v) northwesterly descending slope exists across the entire site. Slope stability analysis has been performed for the proposed project, and is included in Appendix C. The overall site stability of the site is considered stable. However, the southeastern area of the site is proposed to have a multi-family apartment structure over parking level garages, that will require a large excavation into the hillside slope. Based on our review of the architectural drawings dated August 21, 2020, by Will and Fotsch Architects, excavations for the multi-family structure garage basement and the detached single-family residences will require shoring to prevent collapse of the walls of the excavation and destabilization of adjacent properties and the alley. Shoring should be designed and constructed in accordance with the criteria in the Conclusions and Recommendations section of this report. Other areas of the site may require shoring where excavations greater than 5 feet in depth are proposed and space constraints prohibit the implementation of a 1:1 (h:v) temporary slope.

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 very low due to the dense nature of the natural-ground material and the lack of a true shallow static groundwater surface under the site. In our opinion, the site has a very low potential for soil strength loss to occur due to a seismic event.

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

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 site is located 400 feet from the exposed coastline and at an elevation of approximately 41 to 72 feet above MSL. There is risk of tsunami inundation at the site is very low due to the elevation.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is not located within immediately downstream from a lake or embayment. There are no significant bodies of water located at higher elevation or in the general vicinity of the capable of producing a seiche and inundating the subject site.

C. 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 multi-family residential apartment structure, detached residential structures and associated improvements provided the recommendations presented herein are implemented. The proposed work will not, in our opinion, destabilize or result in settlement of adjacent property if the recommendations presented in this report are implemented. In addition, no significant geologic hazards are known to exist on the subject site that would prohibit 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 site. Design of building structures in accordance with the current building codes would reduce the potential for injury or loss of human life. Buildings constructed in accordance with current building codes may suffer significant damage but should not undergo total collapse.

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

VIII. GROUNDWATER

Water seepage was observed in borings B-2 and B-5 at the geologic contact of Old Paralic Deposits, Unit 7, and the Point Loma Formation, at depths of 20 feet and 15.5

feet, respectively. At these depths, the Old Paralic Deposits were observed to be coarse-grained poorly graded sand with silt with free water visible. The poorly graded sands are very porous and easily allow for surface water infiltration and migration. The underlying fine-grained silty sand of the Point Loma Formation was observed to be very dense. Very low porosity and permeability would be expected in the Point Loma Formation due to the fine-grained and very dense nature of the soils, leading to the appearance of perched water conditions at the geologic contact. Perched water fluctuations 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 formational soils are fine-grained and of low permeability, water problems may not become apparent for extended periods of time.

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

IX. CONCLUSIONS AND RECOMMENDATIONS

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

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

We recommend that the planned residential structures be supported and founded on dense to very dense formational soils and/or minimum 90 percent compacted structural fill soils. Existing fill soils across the site are not suitable in their current condition to support the loads from structures or additional fill soils. A full removal and recompaction of existing loose and disturbed fill soils across the site will be required to support structures intended to bear on structural fill soils and for associated site improvements. Fill soils across the site will be required to be compacted to at least 90 percent relative compaction. In the building pad areas where a transition from dense formational soils to new compacted fill may exist , the fills should be compacted to at least 95 percent relative compaction. Existing fill soils are suitable for use as recompacted fill soils. Any buried trash encountered during site demolition and fill soil recompaction should be removed.

Foundations supporting the three detached single-family residential structures in the northwestern portion of the site may by supported on conventional, individual-spread and/or continuous footing foundations. Undisturbed formational soils or compacted structural fill soils are suitable to support the loads from these structures.

A geological contact is anticipated to be exposed during excavation for the basement garage level of the multi-family apartment structure in the southeastern portion of the site. Therefore, the multi-family apartment structure should be entirely supported on either: (1) a combination of caisson and grade beam design with shallow individual-spread and/or continuous footings, deepened sufficiently where necessary to bear structural foundation loads entirely in Point Loma Formation, with a removal and recompaction of the building pad undertaken so garage floor slabs are bearing on uniformly compacted soils, or (2) conventional, individual-spread and/or continuous shallow footing foundations with a full removal and recompaction of the entire multi-family apartment structure pad, including an undercut of Point Loma Formation in the southeastern portion of the building pad.

Should a combination of deepened caisson and grade beam design with shallow individual-spread and/or continuous footing foundation design be utilized for the multi-family apartment structure, the structural loads should bear entirely on the Pont Loma Formation. Caissons will be required on the northwestern portion of the structure and should penetrate entirely through the Old Paralic Deposits. Where the Point Loma Formation is exposed at footing bearing elevation, shallow footings may be utilized. The geological contact is anticipated to be exposed trending in a northeast to southwest direction located approximately midway through the bottom of the excavation. Approximate depths from pad elevation along the northwestern perimeter of the structure to the top of the Point Loma Formation is anticipated to be 18 to 20 feet.

Based on our review of the architectural plan set by Will and Fotsch Architects dated August 21, 2020, shoring will be required to stabilize the side walls of the multi-family apartment structure cut during excavation. Shoring is required to support adjacent properties during site construction if the depth of the excavation exceeds the distance to the adjacent property lines. It is our understanding the garage excavation depth will exceed the distance to the adjacent property lines. Shoring may also be required for the detached single-family residences if space constraints prohibit a temporary cut slope of 1:1 (h:v) during grading operations or footing excavation.

It is our opinion that the site is suitable for the planned residential project provided the recommendations herein are incorporated during design and construction. Further, it is our explicit opinion that the proposed site development would not destabilize neighboring properties or induce the settlement of adjacent structures if designed and constructed in accordance with our recommendations.

A. Site Soil Preparation and Earthwork

1. *Clearing and Stripping:* Complete demolition of the abandoned residential structures should be undertaken. This is to include the complete removal of all subsurface footings, utility lines and miscellaneous debris. After clearing, the ground surface should be stripped of existing vegetation within the areas

of proposed new construction. This includes any roots from existing trees and shrubbery. Holes resulting from the removal of root systems or other buried obstructions that extend below the planned grades should be cleared and backfilled with suitable compacted material compacted to the requirements provided under Recommendation Nos. 3, 4 and 5 below after the excavation bottom has exposed dense formational soils as confirmed by our representative. Prior to any filling operations, the cleared and stripped vegetation and debris should be disposed of off-site.

2. *Shoring Installation and Excavation:* After the site has been cleared and stripped, soldier beam installation for the shoring should be performed around the area of the multi-family structure in the southeast portion of the site. During excavation of the multi-family apartment structure, drainage and lagging should be installed. All building pad areas should have all existing fill and any loose natural soils entirely removed until dense to very dense formational materials are exposed. Structures should bear entirely on formational soils or properly recompacted fill soils. To reduce the potential for differential settlement, if a fill thickness differential greater than 5 feet is observed for any building pad area, or a cut-fill transition occurs at pad elevation, then an undercut of at least 5 feet should be performed.

   Based on the results of our exploratory borings and test holes, as well as our experience with similar materials in the project area, it is our opinion that the existing artificial fill soils, Old Paralic Deposits and Point Loma Formation materials can be excavated utilizing ordinary light to heavy weight earthmoving equipment. Contractors should not, however, be relieved of making their own independent evaluation of excavating the on-site materials prior to submitting their bids. Variability in excavating the subsurface materials should be expected across the project area.

The areal extent and depth required to remove the loose existing fill and loose natural 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 structures and any areas to receive exterior improvements, where feasible, or to the depth of excavation or required fill at that location, whichever is greater.

3. *Subgrade Preparation:* After the site has been cleared, stripped, and the required excavations made, the exposed subgrade soils in areas to receive new fill and/or slab on grade building improvements should be scarified to a depth of 6 inches, moisture conditioned, 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 areas, they should be scarified and moisture conditioned to at least 5 percent over optimum moisture.

4. *Material for Fill:* Existing on-site low-expansion potential (Expansion Index of 50 or less per ASTM D4829-19) soils with an organic content of less than 3 percent by volume are, in general, suitable for use as fill. Imported fill material, where required, should have a low-expansion potential. In addition, both imported and existing on-site materials for use as fill should not contain rocks or lumps more than 6 inches in greatest dimension if the fill soils are compacted with heavy compaction equipment (or 3 inches in greatest dimension if compacted with lightweight equipment). All materials for use as fill should be approved by our representative prior to importing to the site.

Medium to highly expansive soils should not be used as structural fill at a depth of less than 1 foot from footing bearing surface elevation or behind retaining walls. Backfill material to be placed behind retaining walls should be low expansive (E.I. less than 50), with rocks no larger than 3 inches in diameter.

5. *Fill Compaction:* All structural fill, and areas to receive any associated improvements, should be compacted to a minimum degree of compaction of 90 percent based upon ASTM D1557-12e1. For building pads where no transition from cut to fill soils exist, fill soils will still need to be recompacted to 95 percent relative compaction, primarily in areas where the dense natural soils are more compacted than the recompacted fill areas. Fill material should be spread and compacted in uniform horizontal lifts not exceeding 8 inches in uncompacted thickness. Before compaction begins, the fill should be brought to a water content that will permit proper compaction by either: (1) aerating and drying the fill if it is too wet, or (2) watering the fill if it is too dry. Each lift should be thoroughly mixed before compaction to ensure a uniform distribution of moisture.

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

   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.

6. *Trench and Retaining Wall Backfill:* All utility trenches and retaining walls should be backfilled with properly compacted fill. Backfill material should be placed in lift thicknesses appropriate to the type of compaction equipment utilized and compacted to a minimum degree of compaction of 90 percent by mechanical means. Any portion of the trench backfill in public street areas within pavement sections should conform to the material and compaction requirements of the adjacent pavement section. Our experience has shown that even shallow, narrow trenches, such as for irrigation and electrical lines, that are not properly compacted can result in problems, particularly with respect to shallow groundwater accumulation and migration.

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

7. *Observations and Testing*: It is ***mandatory***, and per CBC 2019 Table 1705.6, that a representative of this firm perform observations and fill compaction testing during excavation operations to verify that the remedial operations are consistent with the recommendations presented in this report. All grading excavations resulting from the removal of soils should be observed and evaluated by a representative of our firm before they are backfilled.

B. ***Seismic Design Criteria***

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

9. *Seismic Design Criteria:* The proposed structure should be designed in accordance with the 2019 CBC, which incorporates by reference the ASCE 7-16 for seismic design. We have determined the mapped spectral acceleration values for the site based on a latitude of 32.8460 degrees and a longitude of -117.2778 degrees, utilizing a program titled "*Seismic Design Map Tool" and* provided by the USGS through SEAOC, which provides a solution for ASCE 7-16 utilizing digitized files for the Spectral Acceleration maps. See Appendix B.

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

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

TABLE I
Mapped Spectral Acceleration Values and Design Parameters

S_s	S_1	F_a	F_v	S_{ms}	S_{m1}	S_{ds}	S_{d1}
1.345g	0.472g	1.00	1.83	1.345g	0.864g	0.897g	0.575g

C. Foundation Recommendations

12. *Footings:* We recommend that the proposed structures be supported on conventional, individual-spread and/or continuous footing foundations bearing on formational or properly compacted fill material. No footings should be underlain by undocumented fill soils. All building footings should be built on formational soils or properly compacted fill prepared as recommended above in Recommendation Nos. 3, 4 and 5. All footings for structures two stories or taller should be founded at least 24 inches below the lowest adjacent finished grade. One-story structures may be embedded 18 inches below the lowest adjacent subgrade soils.

    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 utility trenches should be excavated farther from the footing locations.

    Footings located adjacent to the tops of slopes 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. See Figure No. VII, Foundation Requirements Near Slopes, for further details.

13. *Bearing Values:* At the recommended depths, footings on formational or properly compacted fill soils may be designed for allowable bearing pressures of 2,500 pounds per square foot (psf) for combined dead and live loads and 3,300 psf for all loads, including wind or seismic. The footings should, however, have a minimum width of 18 inches. An increase in soil allowable static bearing can be used as follows: 600 psf for each additional foot over 2

feet in depth, and 400 psf for each additional foot in width over 1.5 feet, to a total allowable static bearing pressure not exceeding 4,500 psf. The static soil bearing value may be increased one-third for seismic and wind load analysis. As previously indicated, all of the foundations for the building should be built on dense formational soils or properly compacted fill soils.

14. *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. All footings shall 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 forms.

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

15. *Lateral Loads:* Lateral load resistance for the structure supported on footing foundations may be developed in friction between the foundation bottoms and the supporting subgrade. An allowable friction coefficient of 0.35 is considered applicable. An additional allowable passive resistance equal to an equivalent fluid weight of 300 pounds per cubic foot (pcf) acting against the foundations

may be used in design provided the footings are poured neat against the dense formational or properly compacted fill materials. These lateral resistance values assume a level surface in front of the footing for a minimum distance of three times the embedment depth of the footing and any shear keys, but not less than 8 feet from a slope face, measured from effective top of foundation. Retaining walls supporting surcharge loads or affected by upper foundations shall consider the effect of those upper loads.

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

17. *Retaining and Shoring Walls:* Where temporary slope recommendations cannot be met due to limitations such as close proximity to property lines or existing structures, shoring will be required. Based on the location of the proposed multi-family apartment structure to the northeastern, southeastern and southwestern property lines, shoring will be required. Geologic observations by our firm will be mandatory for excavations over 3 feet in height. If our geologist considers that soil or geologic features show potential instability for temporary excavations, additional unanticipated shoring may be required.

    Retaining and shoring 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 unrestrained (cantilever) walls with level backfill be designed for an equivalent fluid pressure of 38 pcf.

We recommend that restrained walls (i.e., any walls with angle points that restrain them from rotation) with level backfill be designed for an equivalent fluid pressure of 38 pcf plus an additional uniform lateral pressure of 8H pounds per square foot, where H is equal to the height of backfill above the top of the wall footing, in feet (or 56 pcf if using a triangular soil pressure distribution).

Wherever walls will be subjected to surcharge loads, they should also be designed for an additional uniform lateral pressure equal to one-third the anticipated surcharge pressure in the case of unrestrained walls and one-half the anticipated surcharge pressure in the case of restrained walls. The active and at-rest restraining soil pressures used for shoring design should be at least 45 pcf for the unrestrained shoring walls and 64 pcf for restrained shoring walls.

For seismic design of unrestrained walls over 6 feet in retaining height, we recommend that the seismic pressure increment be taken as a fluid pressure distribution utilizing an equivalent fluid weight of 16 pcf. A kh value of 0.18 may be used when designing retaining walls with a computer program such as *Retain Pro*.

The preceding design pressures assume that the walls are backfilled with low expansion potential materials (Expansion Index less than 50) and that there is sufficient drainage behind the walls to prevent the build-up of hydrostatic pressures from surface water infiltration. We recommend that wall drainage be provided using J-Drain 200/220 and J-Drain-SWD. No gravel or separate pipe is required with the J-Drain system. The upper edge of 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.

Gravel should only be used behind retaining walls where space constraints prohibit the proper compaction of backfill soils. For more information, refer to Figure No. VIII, Schematic Retaining Wall Subdrain Recommendations.

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 specify if any retaining walls should be braced as soon as they are built, prior to backfill placement.

D. Concrete Slab on-grade Criteria

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

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

    In our opinion, new interior floor slabs should be at least 5 inches actual thickness and be reinforced with No. 4 bars on 18-inch centers, both ways, placed at mid-height in the slab. We also opine that the lower level (basement) garage slabs be at least 6 inches thick and reinforced similarly to the 5-inch-thick slab. Actual floor slab thickness and reinforcement recommendations may be upgraded by the project Structural Engineer. Soil moisture content should be kept above the optimum prior to waterproofing placement under the new concrete slab.

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

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

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

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

The following American Society for Testing and Materials (ASTM) and American Concrete Institute (ACI) sections address the issue of moisture transmission into and through concrete slabs: ASTM E1745-17 Standard Specification for Plastic Water Vapor Retarders Used in Contact Concrete Slabs; ASTM E1643-18a Standard Practice for Selection, Design, Installation, and Inspection of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs; ACI 302.2R-06 Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials; and ACI 302.1R-15 Guide to Concrete Floor and Slab Construction.

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 subparagraphs 7.1.1-7.1.5) should be less than 0.01 perms (grains/square foot/hour/per inch of Mercury) and comply with the ASTM E1745-17 Class A requirements. Installation of vapor barriers should be in accordance with ASTM E1643-18a. The basis of design is 15-mil Stego Wrap vapor barrier placed per the manufacturer's guidelines. Reef Industries Vapor Guard membrane has also been shown to achieve a permeance of less than 0.01 perms. We recommend that the slab be poured directly on the vapor barrier, which is placed directly on the prepared properly compacted smooth subgrade soil surface.

19.2 Common to all acceptable products, vapor retarder/barrier joints must be lapped at least 6 inches. Seam joints and permanent utility penetrations should be sealed with the manufacturer's recommended tape or mastic. Edges of the vapor retarder should be extended to terminate at a location in accordance with ASTM E1643-18a or to an alternate location that is acceptable to the project's structural engineer. All terminated edges of the vapor retarder should be sealed to the building foundation (grade beam, wall, or slab) using the manufacturer's recommended accessory for sealing the vapor retarder to pre-existing or freshly placed concrete. Additionally, in actual practice, stakes are often driven through the retarder material, equipment is dragged or rolled across the retarder, overlapping or jointing is not properly implemented, etc. All these construction deficiencies reduce the retarder's effectiveness. In no case should retarder/barrier products be punctured or gaps be allowed to form prior to or during concrete placement. Vapor barrier-safe screeding and forming systems should be used that will not leave puncture holes in the vapor barrier, such as Beast Foot (by Stego Industries) or equivalent.

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 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.

20. *Exterior Slab Thickness and Reinforcement:* As a minimum for protection of on-site improvements, we recommend that all exterior pedestrian concrete slabs be 4 inches thick and be founded on properly compacted and tested fill, with No. 3 bars at 15-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.

21. *Driveway and Garage Concrete Pavement:* The new driveway and parking garage pavement, consisting of Portland cement concrete at least 6.0 inches in thickness, may be placed on properly compacted, relatively smooth subgrade soils. The concrete should be at least 3,500 psi compressive strength, with control joints no farther than 12 feet apart and at re-entrant corners. Pavement joints should be properly sealed with a permanent joint

sealant, as required in sections 201.3.6 through 201.3.8 of the Standard Specifications for Public Work Construction, 2018 Edition. 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. If control joints are to be spaced farther than 12 feet apart, the driveway slab should be reinforced with a grid of No. 4 steel bars on 15-inch centers, although spacing should be limited to no greater than 20 feet apart.

All undocumented fills soils in proposed driveway areas should be removed down to dense formational materials 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.

If permeable pavers are used, they should be placed on 1 inch of No. 8 bedding sand, on 8 inches of No. 57 gravel base, on properly compacted subgrade soils. The base and subgrade material should be compacted to at least 95 percent relative compaction. The subgrade and surface of pavers should drain toward the street or to perforated collection subdrain pipes discharging in an approved drainage facility.

E. ***Slopes***

Temporary slopes or shoring, where needed, will be required during site preparation and construction. Shoring may be required adjacent to property lines where any excavation could cause instability of the adjacent property or improvements.

22. *Temporary Slopes*: Based on our subsurface investigation work, laboratory test results, and engineering analysis, temporary cut slopes up to 12 feet in

height in cohesive formational materials with a fines content greater than 35% should be stable from mass instability at an inclination of 0.75:1.0 (horizontal to vertical). Temporary cut slopes up to 12 feet in height in loose/cohesionless soils should be stable against mass instability at an inclination of 1.0:1.0.

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

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

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

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

depends on the inherent expansion and strength characteristics of the soil types comprising the slope, slope steepness and height, loosening of slope face soils by burrowing rodents, and irrigation and vegetation maintenance practices, as well as the quality of compaction of fill soils. Structures and other improvements could suffer damage due to these soil movement factors if not properly designed to accommodate or withstand such movement. The fills derived from on-site sources used on slope faces will be prone to raveling/erosion. We recommend that appropriate measures be taken to reduce these effects. The implementation and maintenance of proper drainage and landscaping should improve the performance of slope faces.

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

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

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

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

F. Site Drainage Considerations

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

Failure to observe this recommendation could result in undermining, soil expansion, and possible differential settlement of the structure or other improvements or cause other moisture-related problems. Currently, the 2019 CBC requires a minimum of 1 percent surface gradient for proper drainage of building pads unless waived by the building official. Concrete pavement may have a minimum gradient of 0.5-percent. The surface gradient adjacent to structures must drain away as indicated in the 2019 CBC.

27. *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.

28. *Planter Drainage:* Any planter areas adjacent to the wall structure should be provided with sufficient area drains to help with rapid runoff disposal. No water should be allowed to pond adjacent to the wall or other improvements.

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

G. General Recommendations

30. *Cal-OSHA*: Where not superseded by specific recommendations presented in this report, trenches, excavations, and temporary slopes at the subject site

should be constructed in accordance with Title 8, Construction Safety Orders, issued by Cal-OSHA.

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

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

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

X. GRADING NOTES

It is recommended that ***Geotechnical Exploration, Inc.*** be retained to verify that soil conditions revealed during grading for the project are as anticipated in this *"Report of Preliminary Geotechnical Investigation."* In addition, the compaction of any fill soils placed during grading must be observed and tested by our field representative.

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

XI. LIMITATIONS

Our conclusions and recommendations have been based on available data obtained from our field investigation, background review and laboratory analysis, as well as our experience with similar soils and natural ground materials located in the City of San Diego.

Of necessity, we must assume a certain degree of continuity between exploratory excavations and/or natural exposures. It is, therefore, necessary that all observations, conclusions, and recommendations be verified at the time excavation begins. In the event discrepancies are noted, additional recommendations may be issued, if required.

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

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

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

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

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

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

Once again, should any questions arise concerning this report, please feel free to contact the undersigned. Reference to our **Job No. 20-12787** will expedite a reply to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

Jaime A. Cerros, P.E.
R.C.E. 34422/G.E. 2007
Senior Geotechnical Engineer

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

Jason Meares
Staff Geologist

JAIME A. CERROS
GE 002007
Exp 9/30/21
GEOTECHNICAL

LESLIE D. REED
No. 999
Exp. 3/31/21
CERTIFIED ENGINEERING GEOLOGIST

REFERENCES

JOB NO. 20-12787

September 2020

2007 Working Group on California Earthquake Probabilities, 2008, The Uniform California Earthquake Rupture Forecast, Version 2 (UCERF 2), U.S Geological Survey Open-file Report 2007-1437 and California Geological Survey Special Report 203.

Association of Engineering Geologists, 1973, Geology and Earthquake Hazards, Planners Guide to the Seismic Safety Element, Association of Engineering Geologists, Southern California Section.

Berger, V. and Schug, D.L., 1991, Probabilistic Evaluation of Seismic Hazard in the San Diego-Tijuana Metropolitan Region, Environmental Perils, San Diego Region, Geological Society of America by the San Diego Association of Geologists, October 20, 1991, p. 89-99.

City of San Diego, 2008, Seismic Safety Study, Geologic Hazards and Faults, Development Services Department, Grid Tile No. 29.

Crowell, J.C., 1962, Displacement Along the San Andreas, Fault, California, Geological Society of America, Special Papers, no. 71.

Demere, T.A. 1997, Geology of San Diego County, California, San Diego Natural History Museum, http://archive.sdnhm.org/research/paleontology/sdgeol.html, accessed July 30, 2020.

Grant Ludwig, L.B. and Shearer, P.M., 2004, Activity of the Offshore Newport-Inglewood Rose Canyon Fault Zone, Coastal Southern California, from Relocated Microseismicity, Bulletin of the Seismological Society of America, 94(2), 747-752.

Greene, H.G., Bailey, K.A., Clarke, S.H., Ziony, J.I. and Kennedy, M.P., 1979, Implications of fault patterns of the inner California continental borderland between San Pedro and San Diego, *in* Abbott, P.L., and Elliot, W.J., eds., Earthquakes and other perils, San Diego region: San Diego Association of Geologists, Geological Society of America field trip, November, 1979, p. 21–28.

Greensfelder, R.W., 1974, Maximum Credible Rock Accelerations from Earthquakes in California, California Division of Mines and Geology.

Hart E.W. and Bryant, W.A., 1997, Fault-Rupture Hazard Zones in California, California Division of Mines and Geology, Special Publication 42.

Hart, E.W., Smith, D.P. and Saul, R.B., 1979, Summary Report: Fault Evaluation Program, 1978 Area (Peninsular Ranges-Salton Trough Region), California Division of Mines and Geology, Open-file Report 79-10 SF, 10.

Hauksson, E. and Jones, L.M., 1988, The July 1986 Oceanside (M_L=5.3) Earthquake Sequence in the Continental Borderland, Southern California Bulletin of the Seismological Society of America, v. 78, p. 1885-1906.

Hileman, J.A., Allen, C.R. and Nordquist, J.M., 1973, Seismicity of the Southern California Region, January 1, 1932 to December 31, 1972; Seismological Laboratory, Cal-Tech, Pasadena, California.

Kennedy, M.P. and Tan, S.S., 2008, Geologic Map of San Diego 30′x60′ Quadrangle, California, California Geological Survey, Department of Conservation.

Richter, C.F., 1958, Elementary Seismology, W.H. Freeman and Company, San Francisco, California.

Rockwell, T.K., 2010, The Rose Canyon Fault Zone in San Diego, Proceedings of the Fifth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Paper No. 7.06C.

Rockwell, T.K., Dawson, T.E., Young Ben-Horin, J. and Seitz, G., 2014, A 21-Event, 4,000-Year History of Surface Ruptures in the Anza Seismic Gap, San Jacinto Fault, and Implications for Long-term Earthquake Production on a Major Plate Boundary Fault, Pure and Applied Geophysics, v. 172, 1143–1165 (2015).

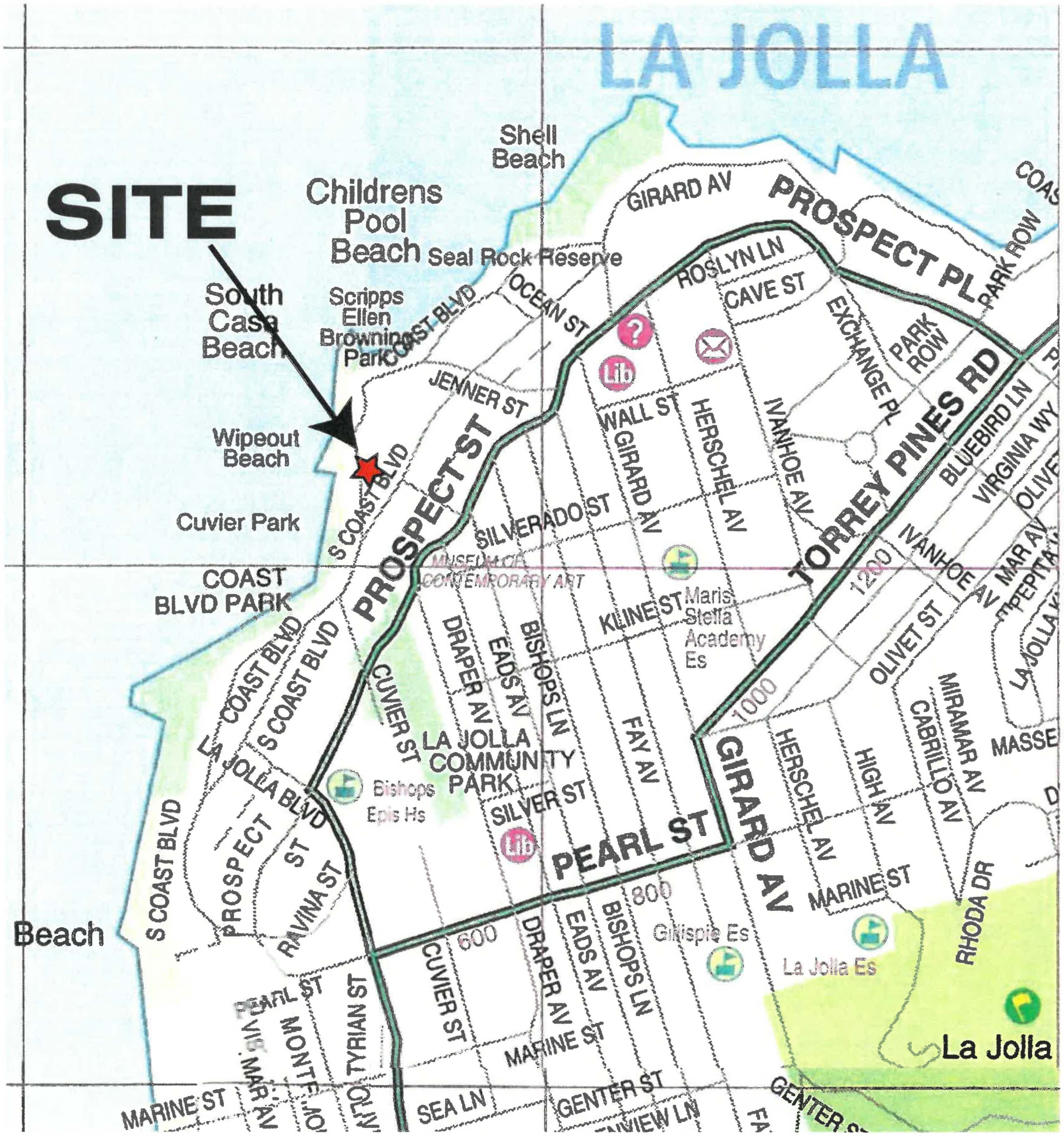
Rockwell, T.K., Millman, D.E., McElwain, R.S. and Lamar, D.L., 1985, Study of Seismic Activity by Trenching Along the Glen Ivy North Fault, Elsinore Fault Zone, Southern California: Lamar-Merifield Technical Report 85-1, U.S.G.S. Contract 14-08-0001-21376, 19 p.

Ross, Z.E., Hauksson E. and Ben-Zion Y., 2017, Abundant Off-fault Seismicity and Orthogonal Structures in the San Jacinto Fault Zone, Science Advances, 2017; 3(3): e1601946. Published 2017 Mar 15.

Toppozada, T.R. and Parke, D.L., 1982, Areas Damaged by California Earthquakes, 1900-1949, California Division of Mines and Geology, Open-file Report. 82-17.

U.S. Dept. of Agriculture, 1953, Aerial Photographs AXN-8M-89 and 90.

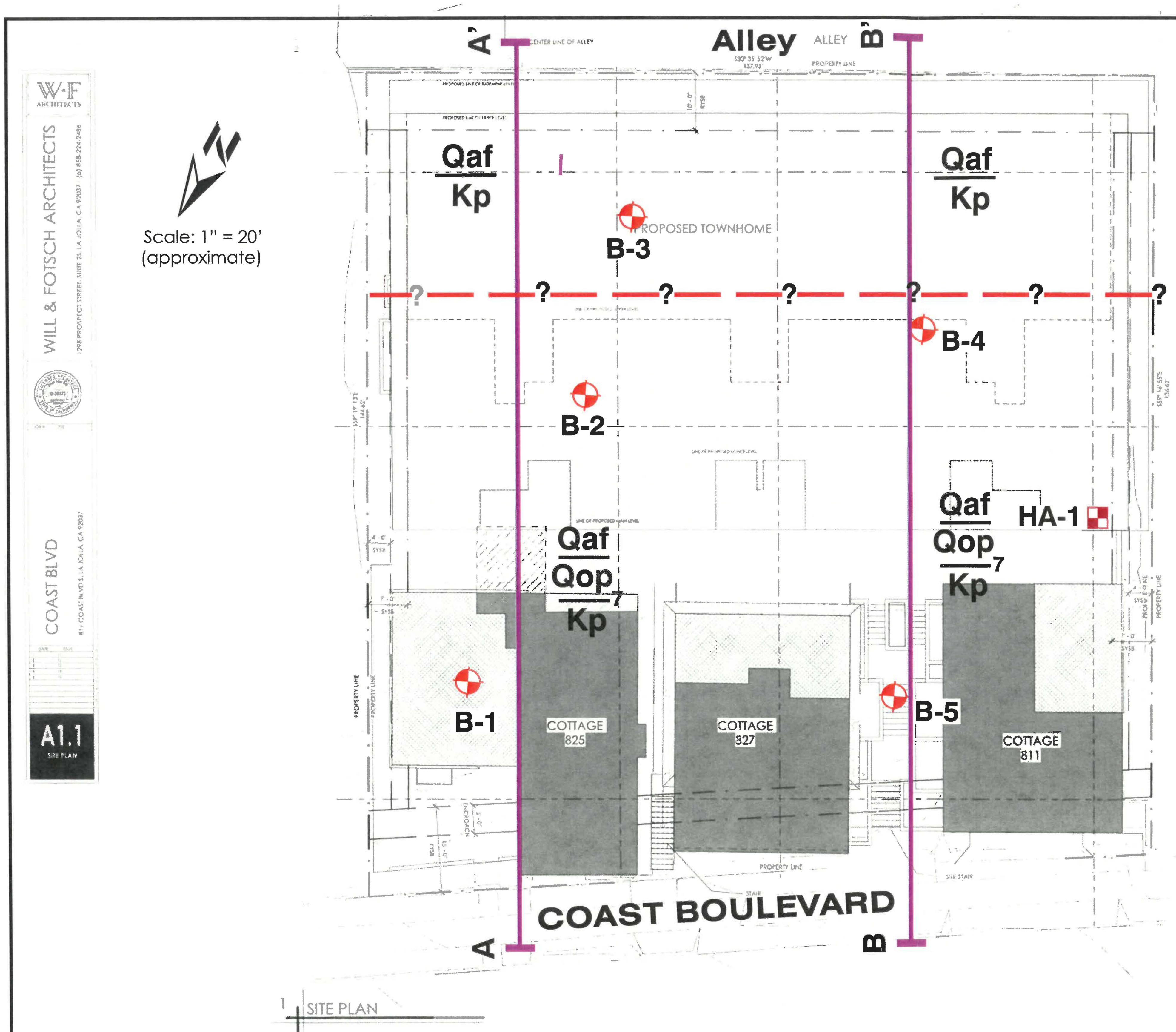
VICINITY MAP



Thomas Bros Guide San Diego County pg 1227-E6

Davidson Residential Development
811 - 827 Coast Boulevard South
La Jolla, CA.

Figure No. I
Job No. 20-12787



LEGEND

▭ PROPOSED NEW BUILDING AREA (TOWNHOMES)
▩ AREA OF EXISTING RESIDENCE
▨ AREA TO BE DEMOLISH
▨ AREA OF ADDICTION

LEGEND

🟥 HA-1	Approximate Location of Exploratory Hand Auger Boring
⊕ B-5	Approximate Location of Exploratory Boring
A ⊢⊣ A'	Approximate Location of Cross Section

GEOLOGIC LEGEND

Qaf	Artificial Fill
Qop_7	Old Paralic Deposits (unit 7)
Kp	Point Loma Formation
— ? —	Approximate Geologic Contact

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

REFERENCE: This PLOT PLAN was prepared from an existing undated SITE PLAN provided by WILL & FOTSCH ARCHITECTS and from on-site field reconnaissance performed by GEI.

PLOT PLAN

Davidson Residential Development
811 - 827 Coast Boulevard South
La Jolla, CA.
Figure No. IIa
Job No. 20-12787

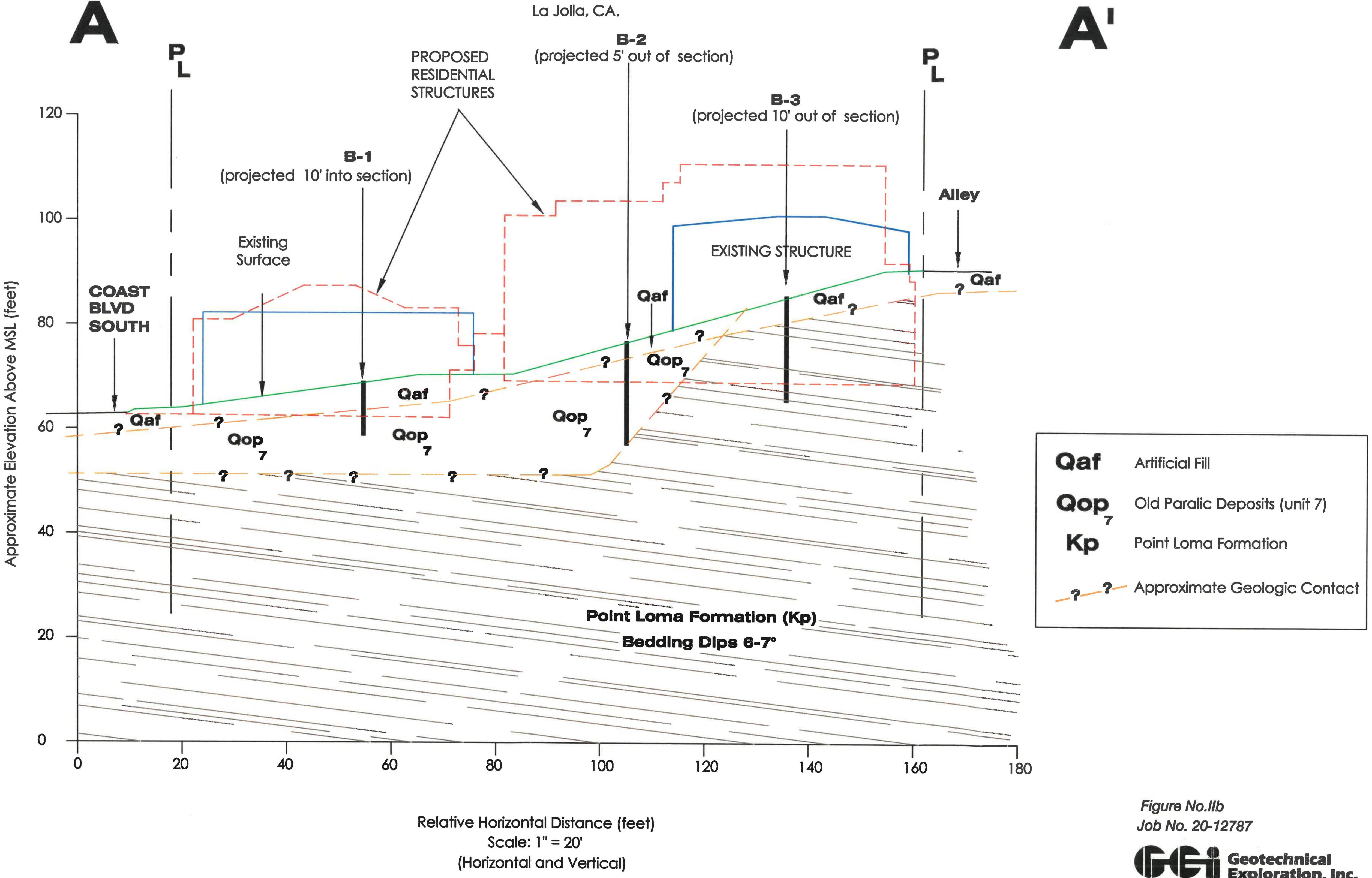
Geotechnical Exploration, Inc.

(September 2020)

20-12787-p2.ai

CROSS SECTION A-A'

Davidson Residential Developement
811 - 827 Coast Boulevard South
La Jolla, CA.



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

20-12787-AA2

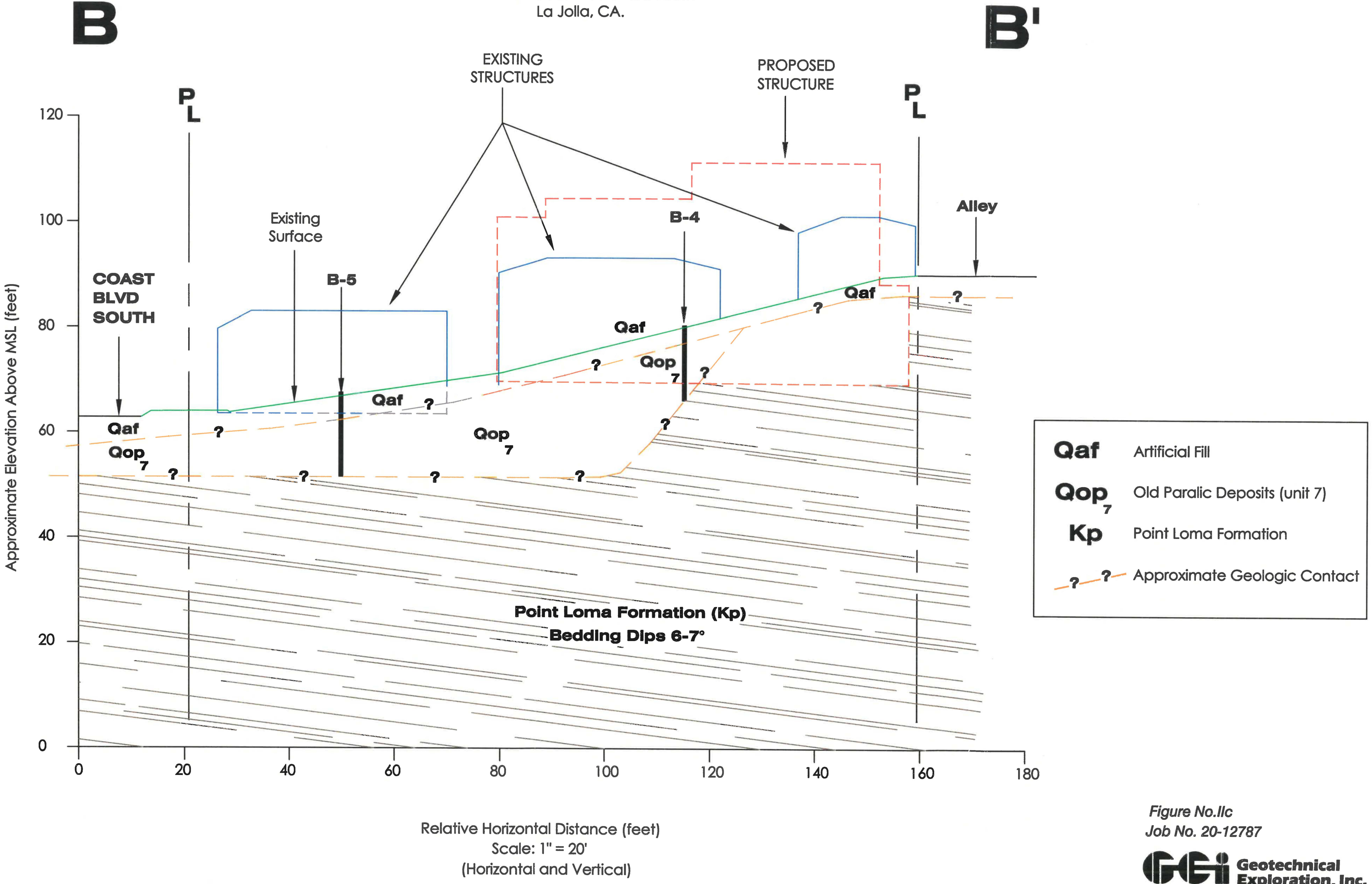
Figure No.IIb
Job No. 20-12787

Geotechnical Exploration, Inc.

September 2020

CROSS SECTION B-B'

Davidson Residential Developement
811 - 827 Coast Boulevard South
La Jolla, CA.



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

20-12787-BB2

Figure No.IIc
Job No. 20-12787

Geotechnical Exploration, Inc.

September 2020

Geotechnical Exploration, Inc.

EQUIPMENT: Limited access tripod drill rig

DIMENSION & TYPE OF EXCAVATION: 6-inch diameter boring

DATE LOGGED: June 25, 2020

LOGGED BY: JM

REVIEWED BY: JAC

SURFACE ELEVATION: ±70' Above Mean Sea Level

GROUNDWATER/SEEPAGE DEPTH: Not encountered

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain Size, Density, Moisture, Color)	U.S.C.S	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of MDD)	EXPAN (+%) CONSOL (-%)	EXPANSION INDEX	BLOW COUNTS / FT	SAMPLE O.D. (in)
2			**SILTY SAND,** fine- to medium-grained. Loose. Slightly moist. Brown to reddish brown. FILL (Qaf) -- Becomes dense and moist at 3 feet.	SM									
4		■										38	3"
6		▨	**SILTY SAND,** fine- to coarse-grained. Loose. Moist. Brown to reddish brown. -- 23% passing No. 200 sieve. OLD PARALIC DEPOSITS, UNIT 7 (Qop_7)	SM								6	2"
		☒					7.6	131.6					
8		■	**POORLY GRADED SAND WITH SILT,** medium- to coarse-grained. Medium dense. Slightly moist. Reddish brown. -- 8% passing No. 200 sieve. OLD PARALIC DEPOSITS, UNIT 7 (Qop_7)	SP-SM								18	3"
10		▨			5.7							17	2"
12			Bottom of boring at 11 feet.										
14													
16													
18													
20													
22													
24													

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

JOB NUMBER: 20-12787

JOB NAME:
Davidson Residential Development

SITE LOCATION:
811-827 Coast Boulevard South,
La Jolla, CA

LOG NO. **B-1**

FIGURE NO. **IIIa**

Geotechnical Exploration, Inc.

EQUIPMENT: Limited access tripod drill rig

DIMENSION & TYPE OF EXCAVATION:
6-inch diameter boring

DATE LOGGED: June 25, 2020

LOGGED BY: JM

REVIEWED BY: JAC

SURFACE ELEVATION: ±78' Above Mean Sea Level

GROUNDWATER/SEEPAGE DEPTH: Seepage at 20 feet.

FIELD DESCRIPTION AND CLASSIFICATION

DEPTH (feet)	SYMBOL	SAMPLE	DESCRIPTION AND REMARKS (Grain Size, Density, Moisture, Color)	U.S.C.S	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of MDD)	EXPAN (+%) CONSOL (-%)	EXPANSION INDEX	BLOW COUNTS / FT	SAMPLE O.D. (in)
			SILTY SAND, fine- to medium-grained. Loose. Slightly moist. Light brown to brown to reddish brown.	SM									
2			**FILL (Qaf)**										
		■	**CLAYEY SAND,** fine- to medium-grained. Medium dense. Slightly moist to moist. Brown to reddish brown.	SC								49	3"
4		▨	**OLD PARALIC DEPOSITS, UNIT 7 (Qop_7)**									21	2"
6		☒	**SILTY SAND,** fine- to medium-grained. Medium dense. Moist. Brown to yellowish brown.	SM									
8			**OLD PARALIC DEPOSITS, UNIT 7 (Qop_7)**										
		☒	**POORLY GRADED SAND WITH SILT,** coarse-grained. Medium dense. Slightly moist. Yellowish brown to reddish brown.	SP-SM									
10		■	-- Becomes dense at 10 feet.									60	3"
12		▨	**OLD PARALIC DEPOSITS, UNIT 7 (Qop_7)**									43	2"
14		☒	-- 8% passing No. 200 sieve.		5.7								
16													
18		☒											
20			-- Becomes wet at 20 feet.										
		▨	**SILTY SAND,** fine-grained. Very dense. Slightly moist. Bluish gray.	SM								50/5"	2"
22			**POINT LOMA FORMATION (Kp)**										
			Bottom of boring at 21.5 feet.										
24													

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

JOB NUMBER: 20-12787

JOB NAME:
Davidson Residential Development

SITE LOCATION:
811-827 Coast Boulevard South,
La Jolla, CA

LOG NO. **B-2**

FIGURE NO. **IIIb**

Geotechnical Exploration, Inc.

DATE LOGGED: June 25, 2020
LOGGED BY: JM
REVIEWED BY: JAC

EQUIPMENT: Limited access tripod drill rig
DIMENSION & TYPE OF EXCAVATION: 6-inch diameter boring
SURFACE ELEVATION: ±86' Above Mean Sea Level
GROUNDWATER/SEEPAGE DEPTH: Not encountered

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain Size, Density, Moisture, Color)	U.S.C.S	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of MDD)	EXPAN (+%) CONSOL (-%)	EXPANSION INDEX	BLOW COUNTS / FT	SAMPLE O.D. (in)
	⬜		**CLAYEY SAND,** fine- to medium-grained. Medium dense. Slightly moist. Dark brown to reddish brown.	SC									
2			**FILL (Qaf)**										
4	🟨		**CLAYEY SAND,** fine-grained. Loose. Moist. Olive.	SC									
6		☒	-- 43% passing No. 200 sieve.		18.3								
			POINT LOMA FORMATION (Kp)										
8		■											
	🟦	■	**SILTY SAND,** fine-grained. Very dense. Moist. Bluish gray with reddish brown lenses.	SM	12.3	117.4						58	3"
10			**POINT LOMA FORMATION (Kp)**										
12		☒	-- 40% passing No. 200 sieve.		13.2								
14													
16													
18													
		■										100/5"	3"
20			Bottom of boring at 19.5 feet.										
22													
24													

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

JOB NUMBER: 20-12787
JOB NAME: Davidson Residential Development
SITE LOCATION: 811-827 Coast Boulevard South, La Jolla, CA

LOG NO. **B-3**
FIGURE NO. **IIIc**

Geotechnical Exploration, Inc.

DATE LOGGED: June 26, 2020
LOGGED BY: JM
REVIEWED BY: JAC

EQUIPMENT: Limited access tripod drill rig
DIMENSION & TYPE OF EXCAVATION: 6-inch diameter boring
SURFACE ELEVATION: ± 81' Above Mean Sea Level
GROUNDWATER/SEEPAGE DEPTH: Not encountered

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain Size, Density, Moisture, Color)	U.S.C.S	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of MDD)	EXPAN (+%) CONSOL (-%)	EXPANSION INDEX	BLOW COUNTS / FT	SAMPLE O.D. (in)
2		☒	**SANDY CLAY.** Soft to stiff. Moist. Dark brown. FILL (Qaf)	CL									
4		☒	**CLAYEY SAND,** fine-grained. Medium dense. Moist. Yellowish brown. OLD PARALIC DEPOSITS, UNIT 7 (Qop_7)	SC									
6 8 10		■ ▨ ☒	**SILTY SAND,** fine-grained. Loose to medium dense. Moist. Olive brown. -- 38% passing No. 200 sieve. OLD PARALIC DEPOSITS, UNIT 7 (Qop_7)	SM	13.0	114.4	8.3	128.0	89			21 6	3" 2"
12 14		☒ ■ ▨	**POORLY GRADED SAND WITH SILT,** coarse-grained. Medium dense. Slightly moist. Olive yellow. OLD PARALIC DEPOSITS, UNIT 7 (Qop_7)	SP-SM	6.1	103.6						30 54	3" 2"
16		▨	**SILTY SAND,** fine-grained. Very dense. Slightly moist. Bluish gray. POINT LOMA FORMATION (Kp) Bottom of boring at 14.5 feet.	SM									
18 20 22 24													

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

JOB NUMBER: 20-12787
JOB NAME:
Davidson Residential Development
SITE LOCATION:
811-827 Coast Boulevard South,
La Jolla, CA

LOG NO. **B-4**

FIGURE NO. **IIId**

Geotechnical Exploration, Inc.

DATE LOGGED: June 06, 2020
LOGGED BY: JM
REVIEWED BY: JAC

EQUIPMENT: Limited access tripod drill rig
DIMENSION & TYPE OF EXCAVATION: 6-inch diameter boring
SURFACE ELEVATION: ± 67' Above Mean Sea Level
GROUNDWATER/SEEPAGE DEPTH: Seepage at 15.5 feet.

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 MDD)	EXPAN (+%) CONSOL (-%)	EXPANSION INDEX	BLOW COUNTS / FT	SAMPLE O.D. (in)
0–4			**SILTY SAND,** fine- to medium-grained. Loose. Moist. Brown to reddish brown. **FILL (Qaf)**	SM									
4		■	**CLAYEY SAND,** fine- to coarse-grained. Loose to medium dense. Moist. Brown to reddish brown.	SC	10.5	121.3						19	3"
6		▨	**OLD PARALIC DEPOSITS, UNIT 7 (Qop_7)**									7	2"
8			**POORLY GRADED SAND WITH SILT,** coarse-grained. Medium dense. Moist. Yellowish brown.	SP-SM									
10		☒	**OLD PARALIC DEPOSITS, UNIT 7 (Qop_7)**										
11			-- Becomes reddish brown at 11 feet.										
12		■										54	3"
13			-- Becomes dense at 13 feet.										
14		▨	-- Becomes yellow at 14 feet.									37	2"
15.5			-- Becomes wet at 15.5 feet.										
16		▨	**SILTY SAND,** fine-grained. Very dense. Slightly moist. Bluish gray to olive yellow. **POINT LOMA FORMATION (Kp)**	SM								50/3"	2"
16.75			Bottom of boring at 16.75 feet.										

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

JOB NUMBER: 20-12787
JOB NAME: Davidson Residential Development
SITE LOCATION: 811-827 Coast Boulevard South, La Jolla, CA

LOG NO. **B-5**

FIGURE NO. **IIIe**

Geotechnical Exploration, Inc.

EQUIPMENT: Hand auger

DIMENSION & TYPE OF EXCAVATION:
4-inch diameter hand auger boring

DATE LOGGED: June 06, 2020

LOGGED BY: JM

REVIEWED BY: JAC

SURFACE ELEVATION: ± 74' Above Mean Sea Level

GROUNDWATER/SEEPAGE DEPTH: Not encountered

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain Size, Density, Moisture, Color)	U.S.C.S	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of MDD)	EXPAN (+%) CONSOL (-%)	EXPANSION INDEX	BLOW COUNTS / FT	SAMPLE O.D. (in)
	▒		**SILTY SAND,** fine- to medium-grained. Loose. Slightly moist. Dark brown. -- Becomes moist and brown at 2 feet. **FILL (Qaf)**	SM									
2	▒												
4	⁘	☒	**CLAYEY SAND,** fine- to medium-grained. Medium dense. Moist. Brown. **OLD PARALIC DEPOSITS, UNIT 7 (Qop_7)**	SC									
6	⁘	☒	**SILTY SAND,** fine- to medium-grained. Medium dense. Moist to very moist. Brown. **OLD PARALIC DEPOSITS, UNIT 7 (Qop_7)**	SM									
8			Bottom of boring at 8 feet.										
10													
12													
14													
16													
18													
20													
22													
24													

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

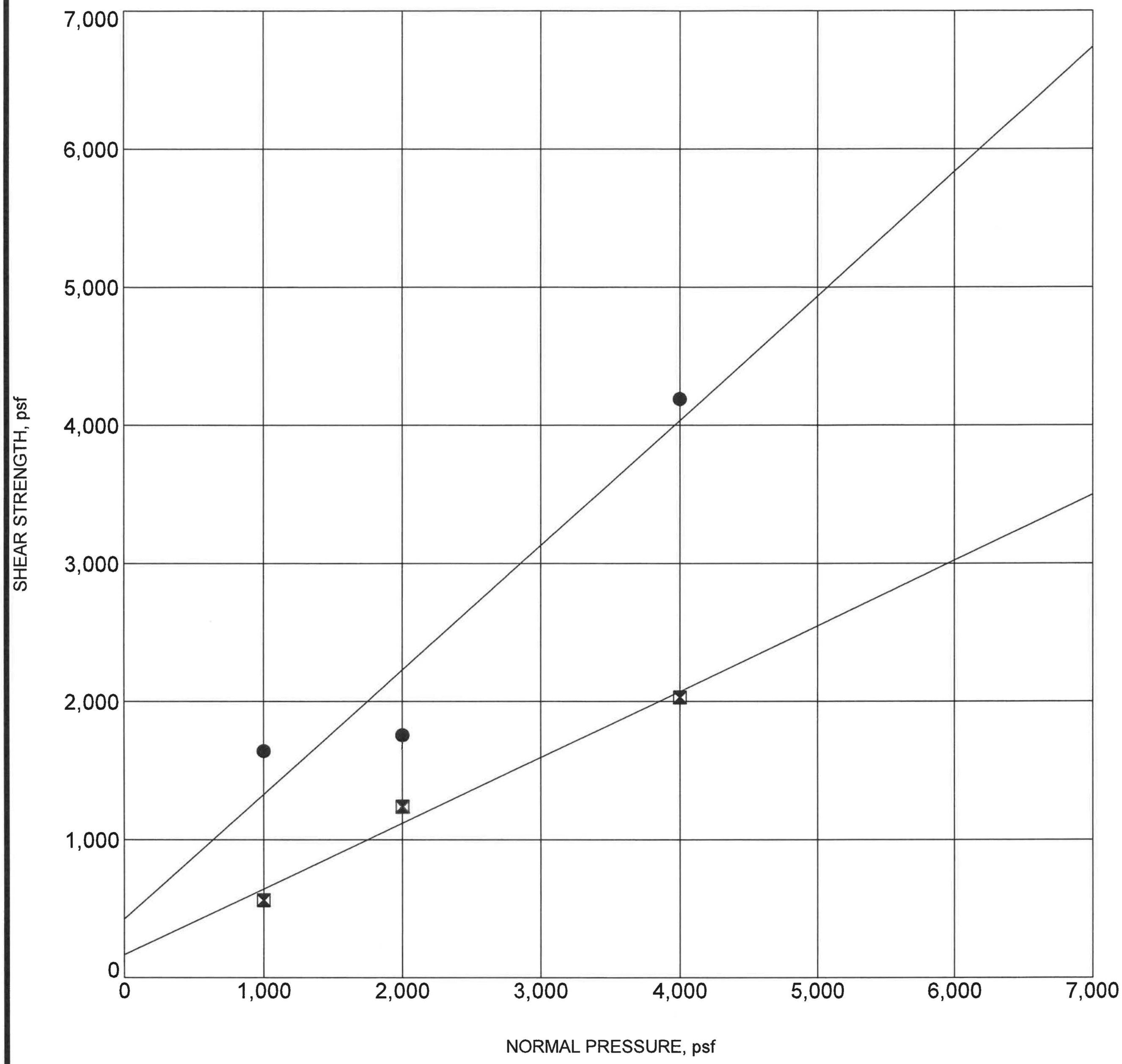
JOB NUMBER: 20-12787

JOB NAME:
Davidson Residential Development

SITE LOCATION:
811-827 Coast Boulevard South,
La Jolla, CA

LOG NO. **HA-1**

FIGURE NO. **IIIf**



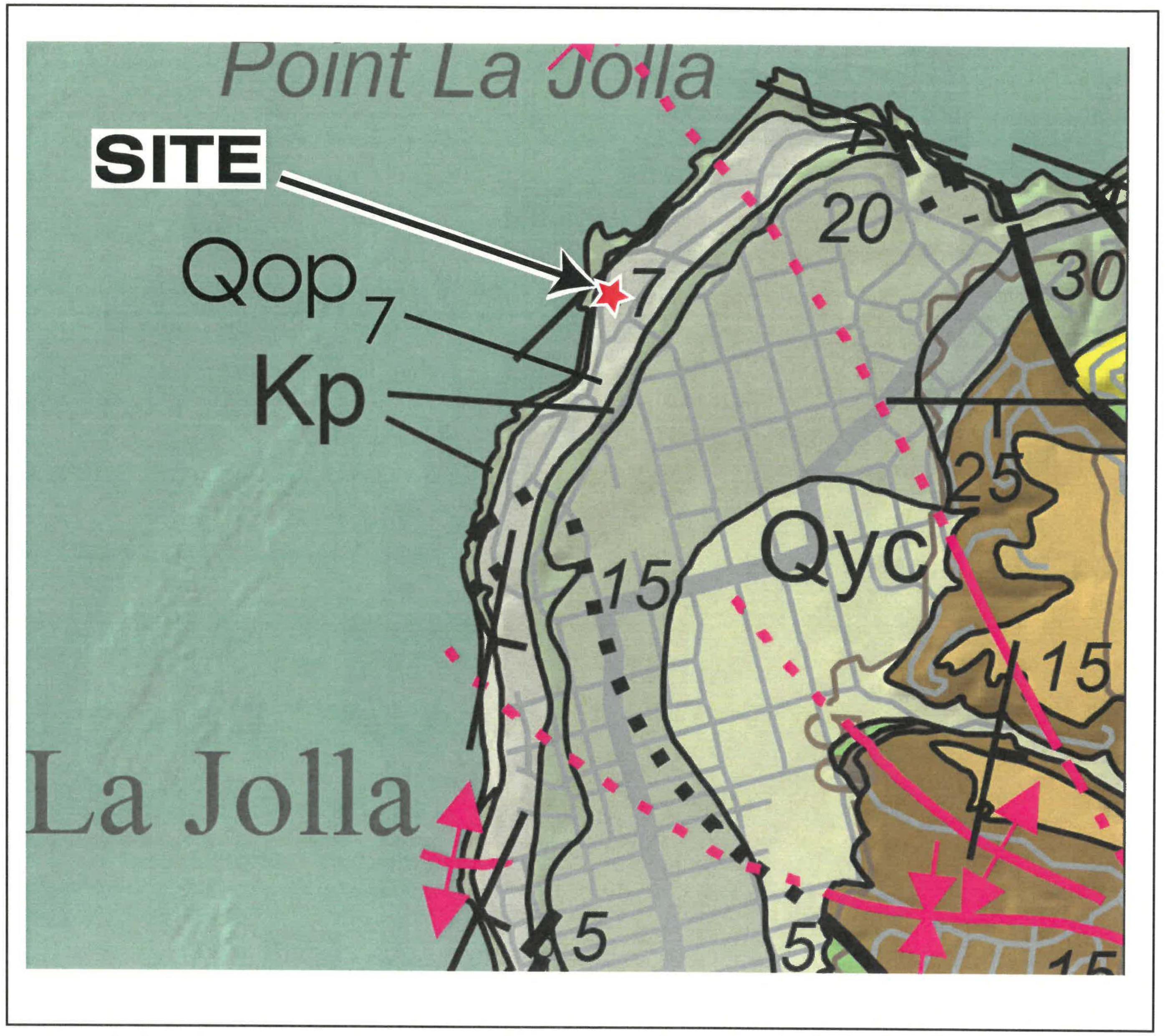
Specimen Identification		Classification	γ_d	MC%	c	ϕ
●	**B-3 @ 9.0'**	**SILTY SAND (SM), Light Olive Brown**			**425**	**42**
☒	**B-4 @ 12.5'**	**SILTY SAND (SP-SM), Light Olive Brown**			**166**	**25**

Geotechnical Exploration, Inc.

DIRECT SHEAR TEST

Figure Number: IV
Job Name: Davidson Residential Development
Site Location: 811-827 Coast Blvd. South, La Jolla, CA
Job Number: 20-12787

GEOLOGIC MAP
2008
compiled by Michael P. Kennedy
and Siang S Tan



Davidson Residential Development
811 - 827 Coast Boulevard South
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.

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

Anticline—Solid where accurately located; dotted where concealed.

Syncline—Solid where accurately located; dotted where concealed.

Kgd—granite pegmatite dike

Closed depression—Closed depression in Elsinore fault zone.

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

Strike and dip of beds
- Inclined
- Overturned
- Vertical
- Horizontal

Strike and dip of igneous foliation
- Inclined
- Vertical

Strike and dip of igneous joints
- Inclined
- Vertical

Strike and dip of metamorphic foliation
- Inclined

Strike and dip of sedimentary joints
- Vertical

Description of Units

Qop_7	Unit 7 Old paralic deposits, undivided
Kp	Point Loma Formation

Figure No. V
Job No. 20-12787

Geotechnical Exploration, Inc.
September 2020

Davidson-LJ-geo.ai

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

Development Services Department
DATE: 4/3/2008

Davidson Residential Development
811 - 827 Coast Boulevard South
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

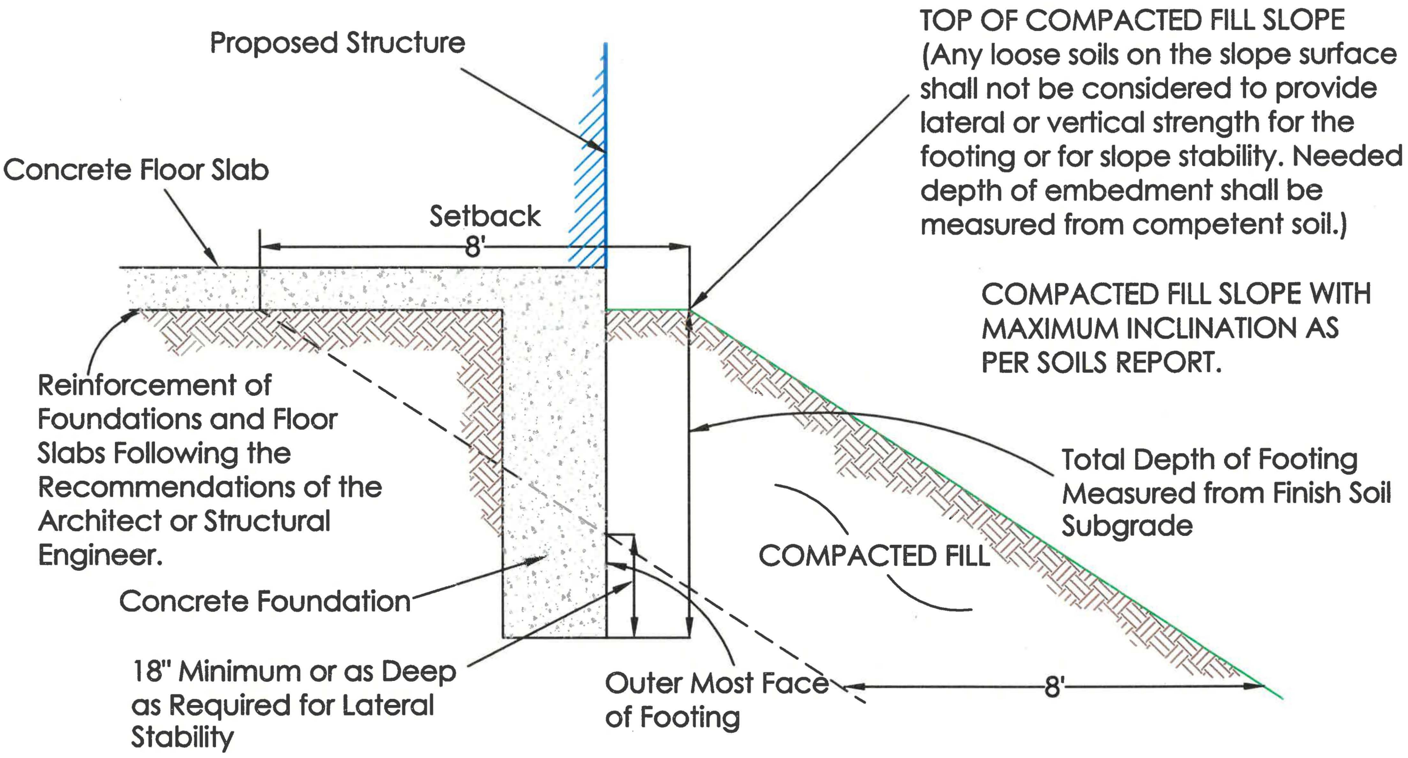
Figure No. VI
Job No. 20-12787

Geotechnical Exploration, Inc.

September 2020

Davidson-LJ-seis.ai

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

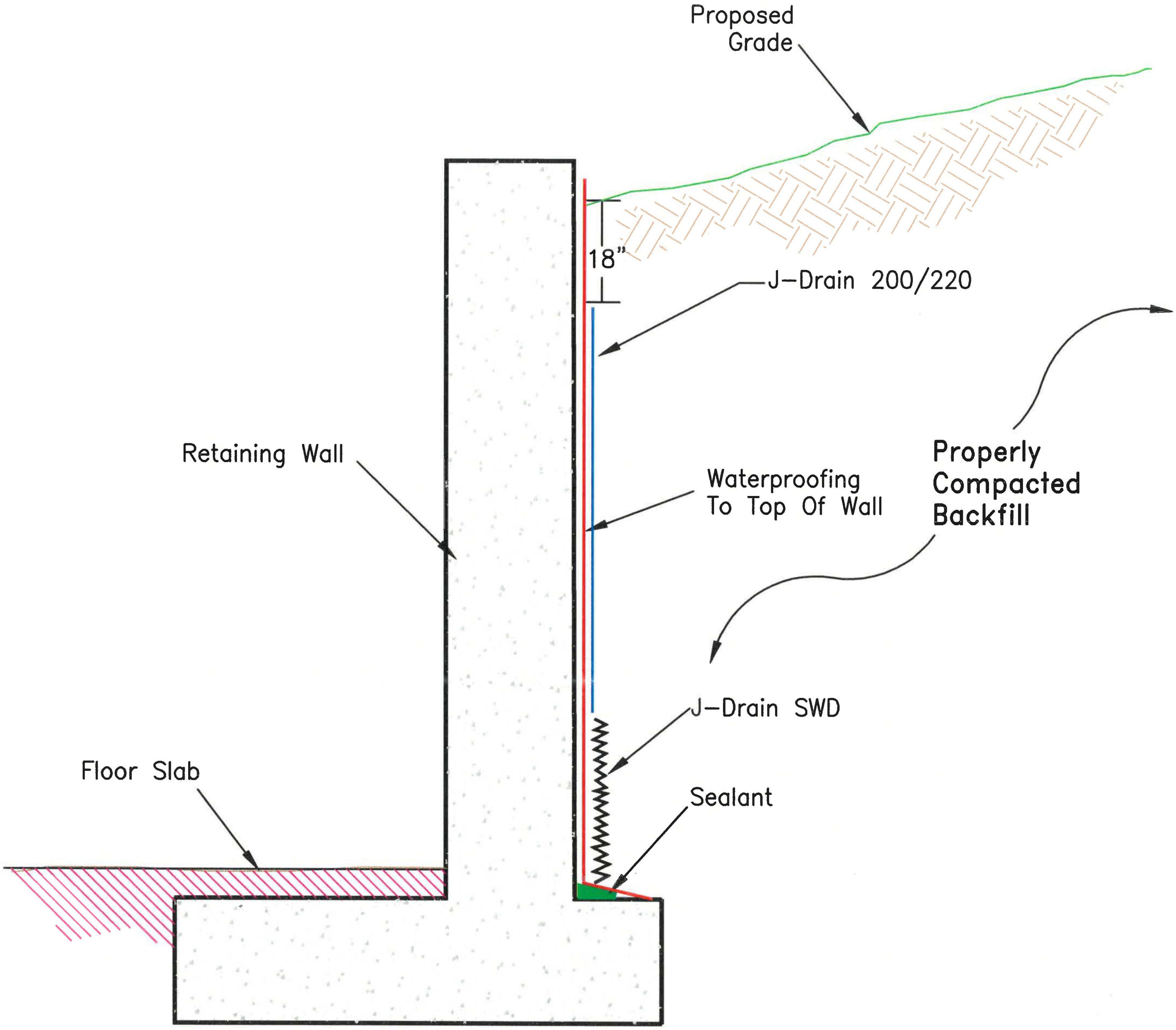
Figure No. VII
Job No. 20-12787

Geotechnical Exploration, Inc.

20-12787-VII.dwg

September 2020

SCHEMATIC RETAINING WALL
SUBDRAIN RECOMMENDATIONS



NOT TO SCALE

Figure No. VIII
Job No. 20-12787

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.

Geotechnical Exploration, Inc.

September 2020

20–12787–VI.dwg

APPENDIX A

APPENDIX A
UNIFIED SOIL CLASSIFICATION CHART
SOIL DESCRIPTION

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

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

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

SILTS AND CLAYS

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

APPENDIX B

APPENDIX B

811-827 Coast Blvd. South, La Jolla, CA

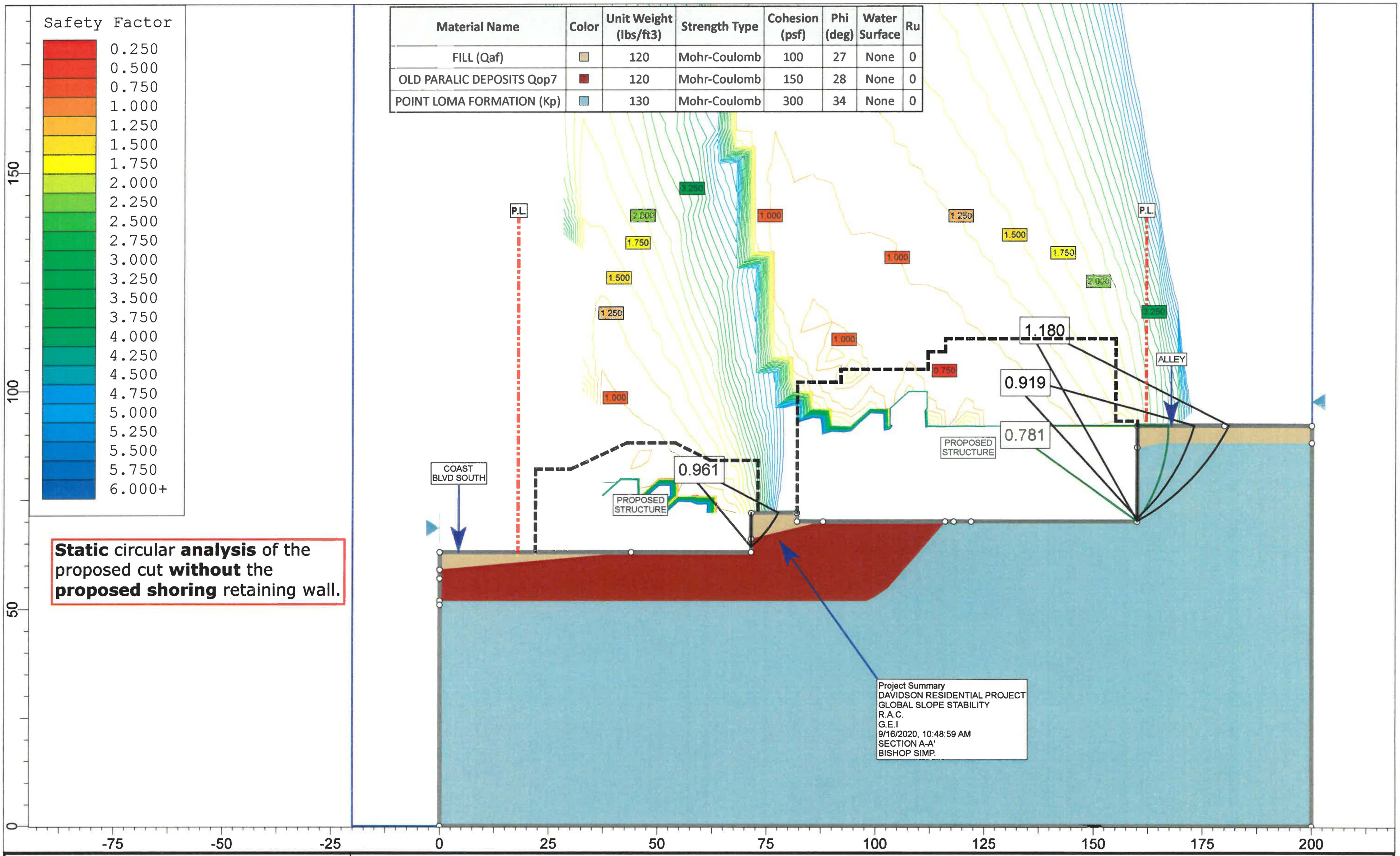
Latitude, Longitude: 32.8460, -117.2778

Date	9/22/2020, 11:37:24 AM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.345	MCE_R ground motion. (for 0.2 second period)
S_1	0.472	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.345	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8 0.864	Site-modified spectral acceleration value
S_{DS}	0.897	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8 0.575	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8 D	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8 1.83	Site amplification factor at 1.0 second
PGA	0.611	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.673	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.345	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.547	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.125	Factored deterministic acceleration value. (0.2 second)
S1RT	0.472	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.531	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.746	Factored deterministic acceleration value. (1.0 second)
PGAd	0.88	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.869	Mapped value of the risk coefficient at short periods
C_{R1}	0.888	Mapped value of the risk coefficient at a period of 1 s

APPENDIX C



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
FILL (Qaf)	🟫	120	Mohr-Coulomb	100	27	None	0
OLD PARALIC DEPOSITS Qop7	🟥	120	Mohr-Coulomb	150	28	None	0
POINT LOMA FORMATION (Kp)	🟦	130	Mohr-Coulomb	300	34	None	0

Static circular **analysis** of the proposed cut **without** the **proposed shoring** retaining wall.

Project Summary
DAVIDSON RESIDENTIAL PROJECT
GLOBAL SLOPE STABILITY
R.A.C.
G.E.I
9/16/2020, 10:48:59 AM
SECTION A-A'
BISHOP SIMP.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project DAVIDSON RESIDENTIAL PROJECT

Analysis Description GLOBAL SLOPE STABILITY

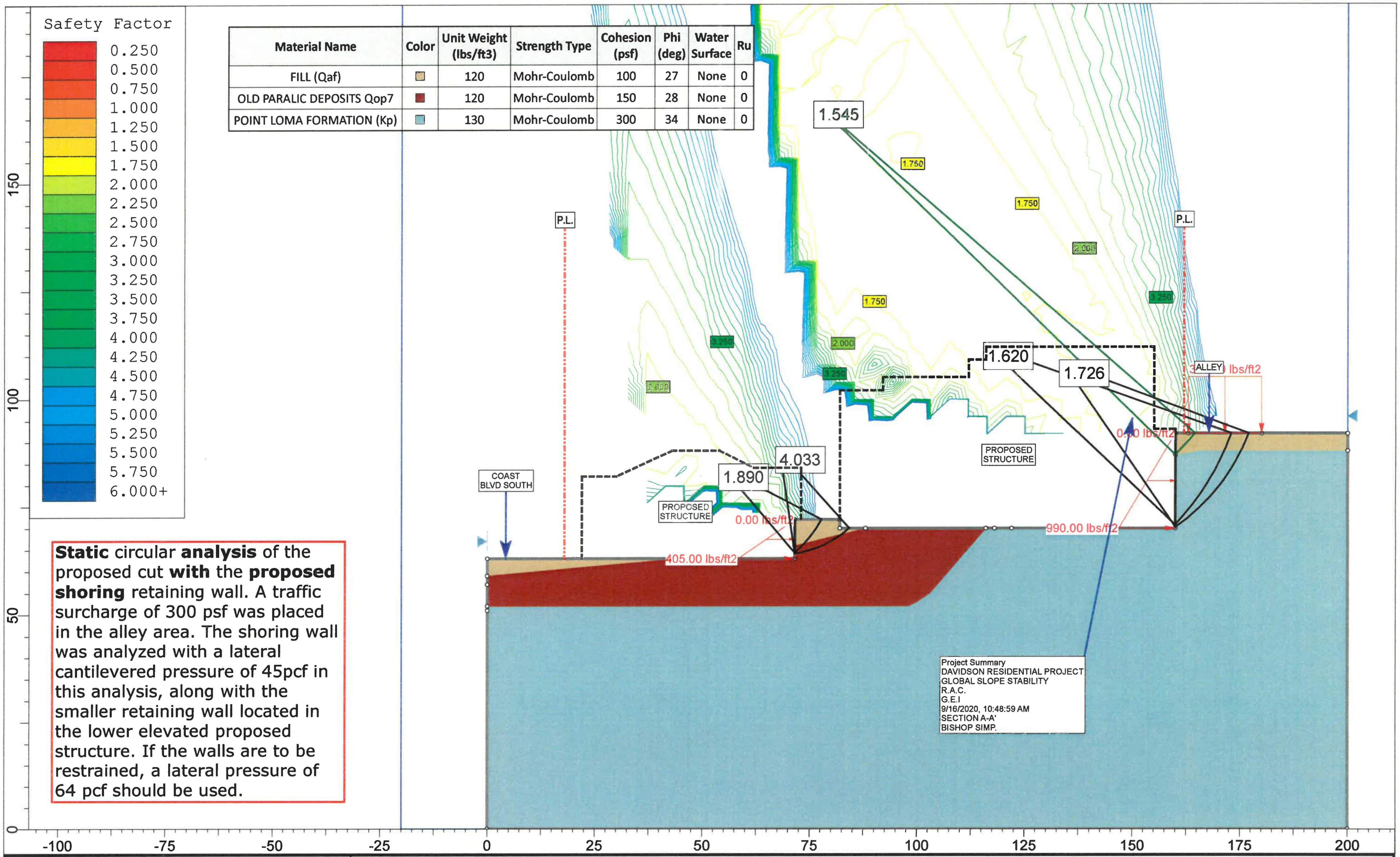
Drawn By R.A.C.

Scale 1:365

Company G.E.I

Date 9/16/2020, 10:48:59 AM

File Name JOB NO. 20-12787_S(A)_01.slim



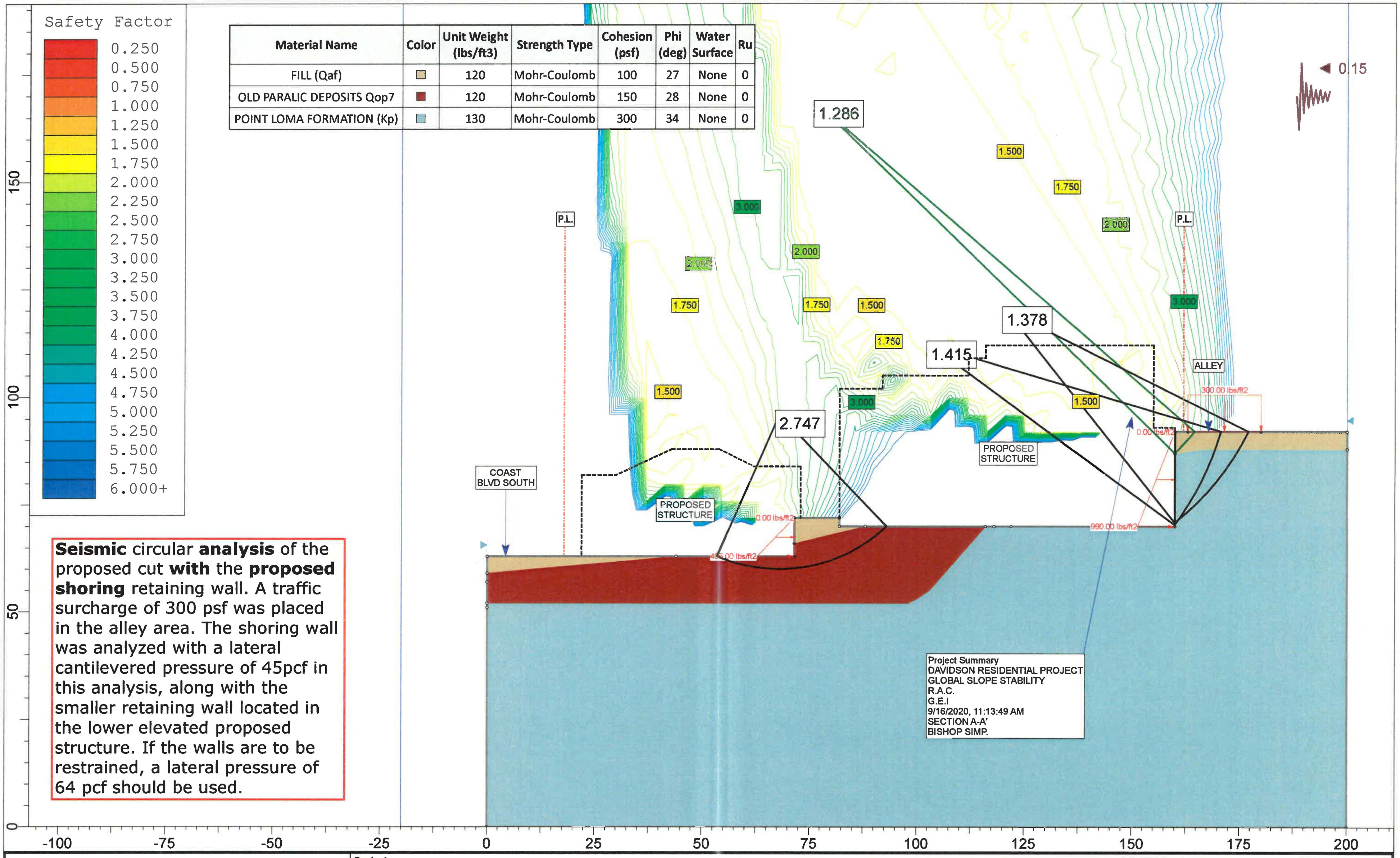
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
FILL (Qaf)	🟫	120	Mohr-Coulomb	100	27	None	0
OLD PARALIC DEPOSITS Qop7	🟥	120	Mohr-Coulomb	150	28	None	0
POINT LOMA FORMATION (Kp)	🟦	130	Mohr-Coulomb	300	34	None	0

Static circular **analysis** of the proposed cut **with** the **proposed shoring** retaining wall. A traffic surcharge of 300 psf was placed in the alley area. The shoring wall was analyzed with a lateral cantilevered pressure of 45pcf in this analysis, along with the smaller retaining wall located in the lower elevated proposed structure. If the walls are to be restrained, a lateral pressure of 64 pcf should be used.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	DAVIDSON RESIDENTIAL PROJECT **SECTION A-A'**
Analysis Description	GLOBAL SLOPE STABILITY
Drawn By	R.A.C.
Scale	1:370
Company	G.E.I
Date	9/16/2020, 10:48:59 AM
File Name	JOB NO. 20-12787_S(A)_02.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
FILL (Qaf)	🟫	120	Mohr-Coulomb	100	27	None	0
OLD PARALIC DEPOSITS Qop7	🟥	120	Mohr-Coulomb	150	28	None	0
POINT LOMA FORMATION (Kp)	🟦	130	Mohr-Coulomb	300	34	None	0

Seismic circular **analysis** of the proposed cut **with** the **proposed shoring** retaining wall. A traffic surcharge of 300 psf was placed in the alley area. The shoring wall was analyzed with a lateral cantilevered pressure of 45pcf in this analysis, along with the smaller retaining wall located in the lower elevated proposed structure. If the walls are to be restrained, a lateral pressure of 64 pcf should be used.

Geotechnical Exploration, Inc.

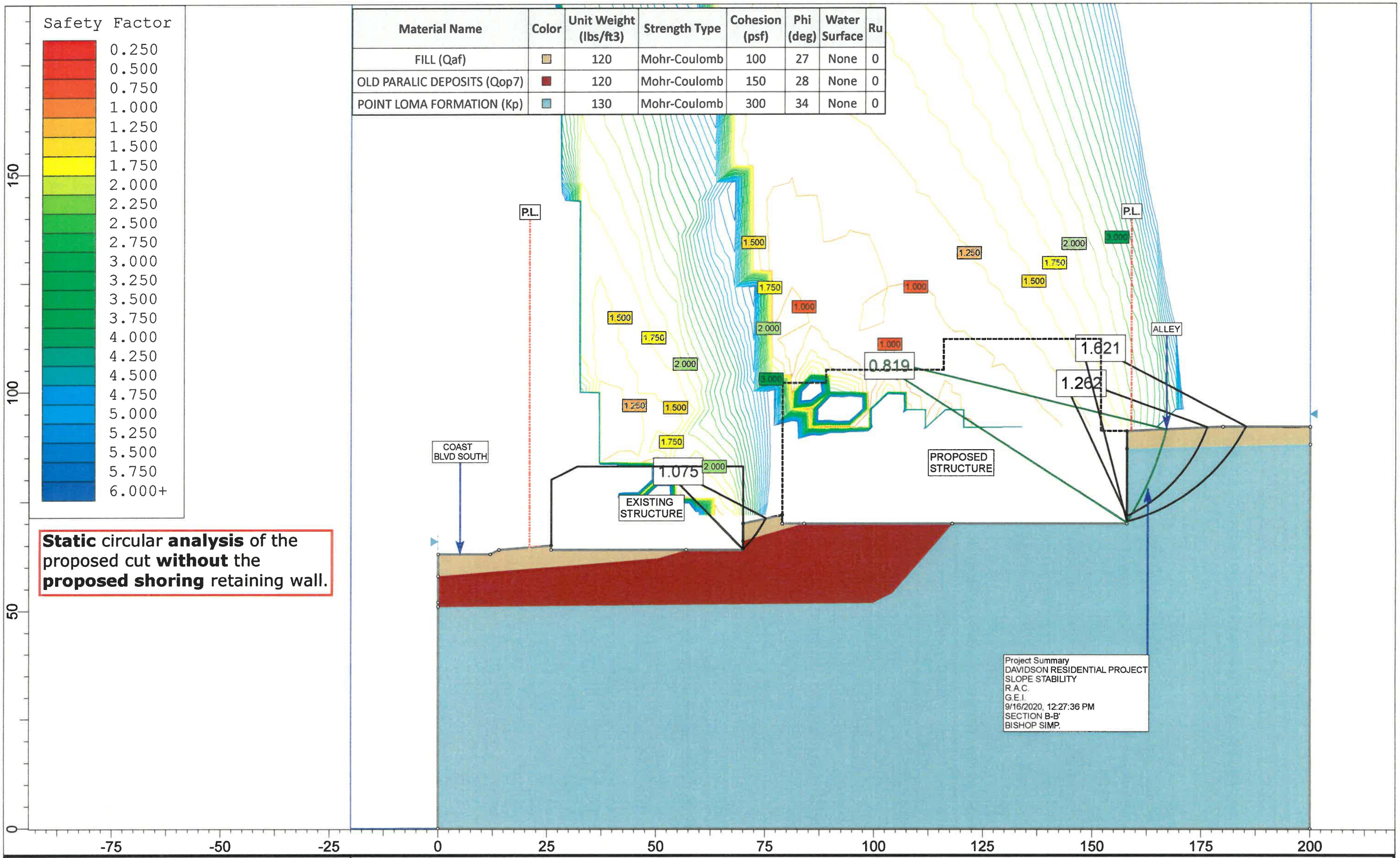
SLIDEINTERPRET 6.039

Project
DAVIDSON RESIDENTIAL PROJECT
SECTION A-A'

Analysis Description
GLOBAL SLOPE STABILITY

Drawn By R.A.C.
Scale 1:370
Company G.E.I

Date 9/16/2020, 11:13:49 AM
File Name JOB NO. 20-12787_S(A)_02w_0.15gSHAKE.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
FILL (Qaf)	🟫	120	Mohr-Coulomb	100	27	None	0
OLD PARALIC DEPOSITS (Qop7)	🟥	120	Mohr-Coulomb	150	28	None	0
POINT LOMA FORMATION (Kp)	🟦	130	Mohr-Coulomb	300	34	None	0

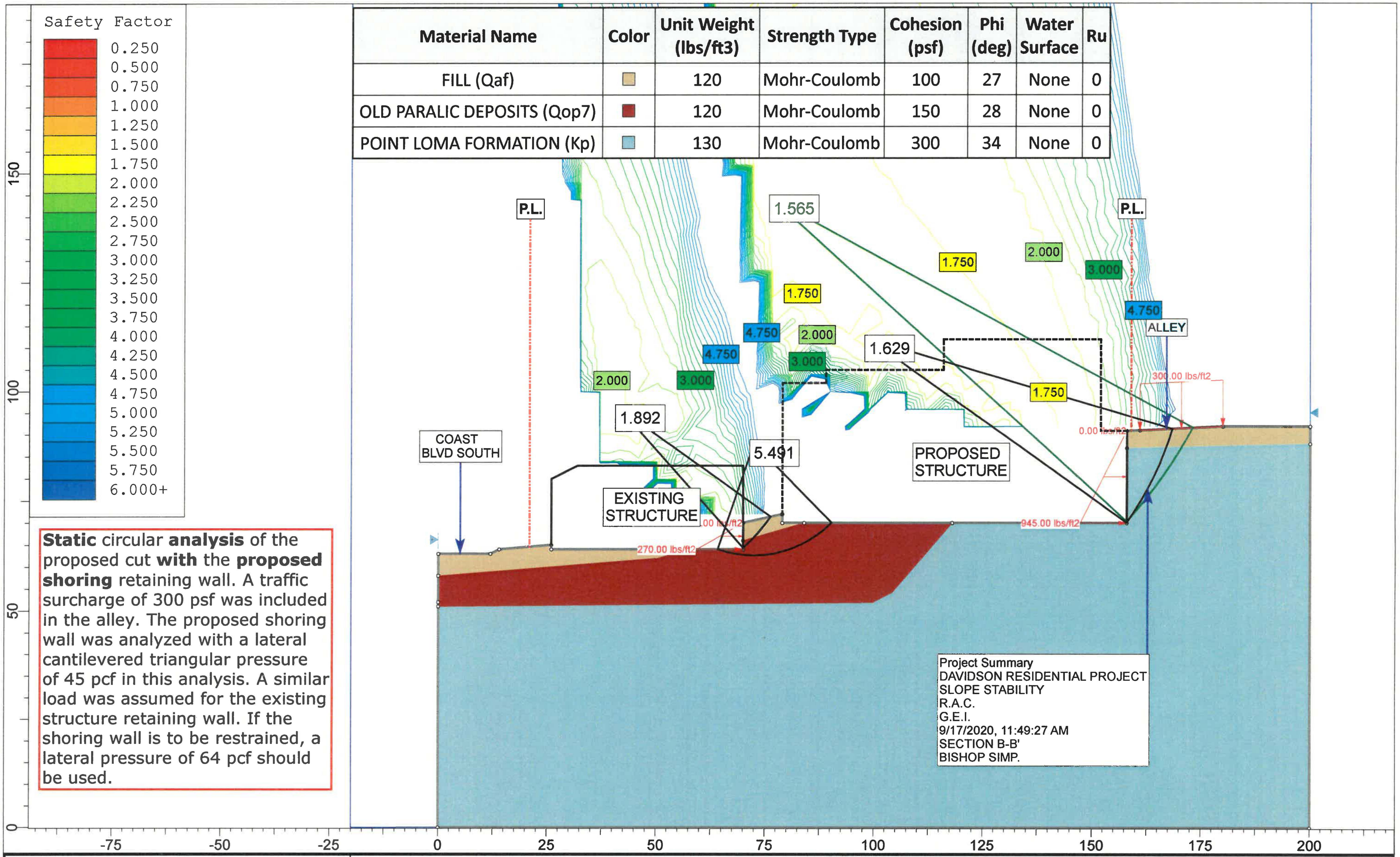
Static circular **analysis** of the proposed cut **without** the **proposed shoring** retaining wall.

Project Summary
DAVIDSON RESIDENTIAL PROJECT
SLOPE STABILITY
R.A.C.
G.E.I.
9/16/2020, 12:27:36 PM
SECTION B-B'
BISHOP SIMP.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	DAVIDSON RESIDENTIAL PROJECT **SECTION B-B'**
Analysis Description	SLOPE STABILITY
Drawn By	R.A.C.
Scale	1:365
Company	G.E.I.
Date	9/16/2020, 12:27:36 PM
File Name	JOB NO. 20-12787_S(B)_01.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
FILL (Qaf)	🟫	120	Mohr-Coulomb	100	27	None	0
OLD PARALIC DEPOSITS (Qop7)	🟥	120	Mohr-Coulomb	150	28	None	0
POINT LOMA FORMATION (Kp)	🟦	130	Mohr-Coulomb	300	34	None	0

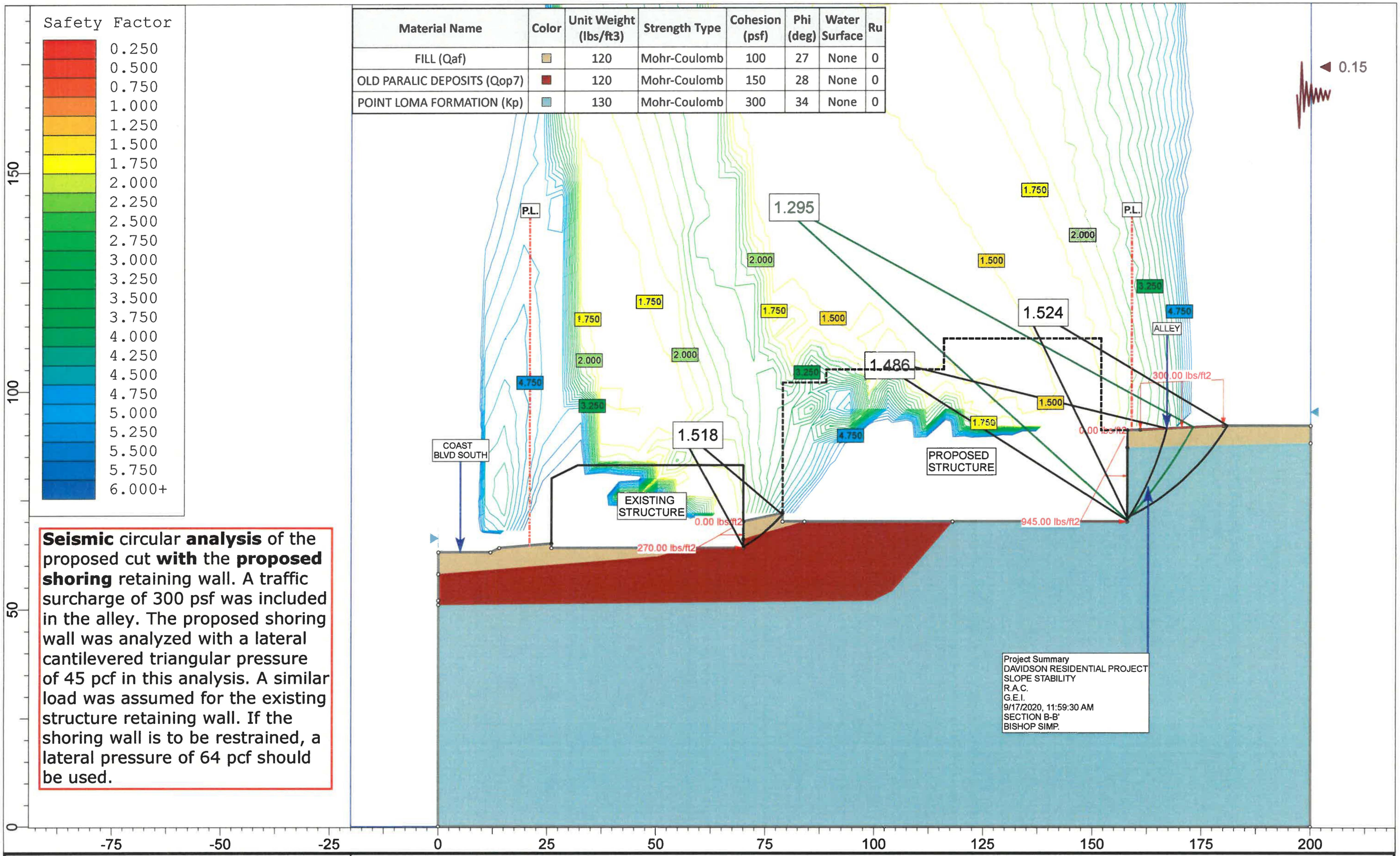
Static circular **analysis** of the proposed cut **with** the **proposed shoring** retaining wall. A traffic surcharge of 300 psf was included in the alley. The proposed shoring wall was analyzed with a lateral cantilevered triangular pressure of 45 pcf in this analysis. A similar load was assumed for the existing structure retaining wall. If the shoring wall is to be restrained, a lateral pressure of 64 pcf should be used.

Project Summary
DAVIDSON RESIDENTIAL PROJECT
SLOPE STABILITY
R.A.C.
G.E.I.
9/17/2020, 11:49:27 AM
SECTION B-B'
BISHOP SIMP.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	DAVIDSON RESIDENTIAL PROJECT	**SECTION B-B'**	
Analysis Description	SLOPE STABILITY		
Drawn By	R.A.C.	*Scale*	1:365
Company	G.E.I.		
Date	9/17/2020, 11:49:27 AM	*File Name*	JOB NO. 20-12787_S(B)_02.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru
FILL (Qaf)	🟫	120	Mohr-Coulomb	100	27	None	0
OLD PARALIC DEPOSITS (Qop7)	🟥	120	Mohr-Coulomb	150	28	None	0
POINT LOMA FORMATION (Kp)	🟦	130	Mohr-Coulomb	300	34	None	0

Seismic circular **analysis** of the proposed cut **with** the **proposed shoring** retaining wall. A traffic surcharge of 300 psf was included in the alley. The proposed shoring wall was analyzed with a lateral cantilevered triangular pressure of 45 pcf in this analysis. A similar load was assumed for the existing structure retaining wall. If the shoring wall is to be restrained, a lateral pressure of 64 pcf should be used.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project DAVIDSON RESIDENTIAL PROJECT **SECTION B-B'**

Analysis Description SLOPE STABILITY

Drawn By R.A.C. *Scale* 1:365 *Company* G.E.I.

Date 9/17/2020, 11:59:30 AM *File Name* JOB NO. 20-12787_S(B)_02w_0.15gSHAKE.slim