UPDATE REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION AND COASTAL BLUFF EDGE EVALUATION

Lowenthal Residential Project
1720 Torrey Pines Road
La Jolla, California

JOB NO. 01-8018
03 July 2024

Prepared for:

Mr. Richard Lowenthal

03 July 2024

Mr. Richard Lowenthal
1720 Torrey Pines Road
La Jolla, CA 92037

Job No. 01-8018

Subject: **Update Report of Preliminary Geotechnical Investigation and Coastal Bluff Edge Evaluation**
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La Jolla, California

Dear Mr. Lowenthal:

In accordance with your request, ***Geotechnical Exploration, Inc.*** has prepared this update to our preliminary geotechnical investigation, evaluation of the general geologic conditions, and coastal bluff edge location evaluation at the property located at 1720 Torrey Pines Road, La Jolla, California, per the current requirements of the City of San Diego. The field work was originally performed in 1999 for a prior owner. This report updates our geotechnical report issued in 2000 and our bluff evaluation report issued in 2001.

It is our understanding the existing residence is to be significantly remodeled including a new basement, new second-story and extensive exterior improvements including a swimming pool. In our opinion, if the conclusions and recommendations presented in this update report are incorporated into the design of the residential project and implemented during site preparation, the site will be suited for the proposed project and associated improvements.

This opportunity to be of service is sincerely appreciated. Should you have any questions concerning the following report, please do not hesitate to contact us. Reference to our **Job No. 01-8018** 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

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The following report presents the findings and recommendations of ***Geotechnical Exploration, Inc.*** for the subject project. For site location, refer to the Vicinity Map, Figure No. I.

I. PROJECT SUMMARY

It is our understanding, based on review of schematic plans prepared by Marengo Morton Architects, dated February 5, 2024, that the existing single-story residence at 1720 Torrey Pines Road is to be expanded including a basement-level addition, lateral additions, a second-story addition, and associated exterior improvements including a swimming pool. The following prior investigations have been performed by our firm and others on the subject property and the data has been utilized in preparation of this report. This report is an update to the 2000 and 2001 reports (see below) prepared by this firm (GEI). Copies can be provided upon request:

1. Addendum Update Report of Results of Historic Bluff Recession Evaluation and Stability Evaluations, Korevaar Property, 1720 Torrey Pines Road, prepared by Geotechnical Exploration, Inc., dated April 20, 2007 (GEI Job No. 01-8018).

2. Addendum Opinion of Effect of Sea Level Rise on Property, Korevaar Property, 1720 Torrey Pines Road, prepared by Geotechnical Exploration, Inc., dated July 29, 2002 (GEI Job No. 01-8018).

3. Response to City of San Diego Second Geotechnical (Geology) Review of Documents, Bluff Recession Evaluation, Korevaar Property, 1720 Torrey Pines Road, prepared by Geotechnical Exploration, Inc., dated May 29, 2002 (GEI Job No. 01-8018).

4. Results of Historic Bluff Recession Evaluation and Stability Evaluations, Korevaar Property, 1720 Torrey Pines Road, prepared by Geotechnical Exploration, Inc., dated October 3, 2001 (GEI Job No. 01-8018).

5. Letter of Opinion of Historic Bluff Rim Recession, Wallace Property, 1720 Torrey Pines Road, prepared by GEI, dated October 6, 2000 (GEI Job No. 99-7622).

6. Report of Geotechnical Investigation, Wallace Property, 1720 Torrey Pines Road, by Geotechnical Exploration, Inc. (GEI), dated February 9, 2000 (GEI Job No. 99-7622). This report includes the logs of 6 exploratory handpits placed by GEI on November 16, 1999 and the additional four borings (B-4, B-5, B-6 and B-7) placed by SCS&T on November 15, 1999. The 1999 GEI logs and laboratory data have been provided as Figure Nos. III and IV and have been utilized for this update report. The handpit locations are shown on Figure No. II, the Plot Plan with Site-specific Geology.

7. Bluff Photographs by GEI, dated December 23, 1999, and historic photographs (1928-1966).

8. Summary of Additional Subsurface Borings, Proposed Dammeyer Residence, 1720 Torrey Pines Road, prepared by SCS&T, dated November 15, 1999 (SCS&T Job No. 9911152.2). The results of four additional exploratory borings (B-4, B-5, B-6, and B-7) were presented in this 1999 SCS&T report and have been addressed in the subsequent reports prepared by GEI, including this 2024

update report. The additional 1999 SCS&T boring logs are provided in Appendix A of this report and are shown on Figure No. II, the Plot Plan with Site-specific Geology.

9. Preliminary Geotechnical Review of Southern California Soil and Testing Report No. 991152.1 dated 9/24/99, prepared by GEI, dated October 6, 1999 (GEI Job No. 99-7622).

10. Review of SCS&T Report of September 24, 1999, 1720 Torrey Pines Road, prepared by Criterium-Fennema Engineers, dated October 5, 1999.

11. Report of Feasibility Study and Geologic Reconnaissance, Proposed Dammeyer Residence, 1720 Torrey Pines Road, by Southern California Soil and Testing (SCS&T), dated September 24, 1999 (SVCS&T Job No. 991152.1). The results of three exploratory borings (B-1, B-2, and B-3) were included in this 1999 SCS&T report and have been utilized in the subsequent reports prepared by GEI, including this 2024 update report. The 1999 SCS&T boring logs are provided in Appendix A of this report and the boring locations are shown on Figure No. II, the Plot Plan with Site-specific Geology.

12. Limited Structural Inspection – June 15, 1999, 1720 Torrey Pines Road, prepared by Criterium-Fennema Engineers, dated June 21, 1999.

The proposed remodel and additions will be constructed with standard-type building materials utilizing conventional foundations and basement retaining walls. Foundation loads are expected to be typical for this type of relatively light construction. Construction plans have not been provided to us during the preparation of this report, however, when completed they should be made available for our review. Additional or modified recommendations will be provided at that time if warranted.

Based on our current understanding of the planned construction, it is our opinion that the proposed site development would not destabilize neighboring properties or induce the settlement of adjacent structures or right-of-way improvements if designed and constructed in accordance with our recommendations. It is also our explicit opinion, based on our field investigation, review of pertinent geologic literature and analysis of geological maps and aerial photographs, that the site is not underlain by an active fault. A spur of the northwest trending Mount Soledad Fault is mapped crossing the north corner of the subject property outside of the proposed habitable structure area.

A. <u>Prior Investigation Findings</u>

The prior subsurface investigations revealed that the lot is underlain at shallow depth by native soils consisting of medium dense to dense/hard, formational material consisting of Quaternary-age Old Paralic Deposits (Qop_6) and Cretaceous-age Point Loma Formation (Kp). Fill soils ranging in thickness from approximately 1 to 5 feet overlie the Old Paralic Deposits. Thicker fill soils were identified by SCS&T in their 1999 borings, however, these soils appear to have been mis-identified in the small diameter borings. Based on our field investigation most of the building pad is underlain by shallow-depth fills, natural colluvial soils and Old Paralic Deposits soils.

B. <u>General Site Preparation Recommendations</u>

It is recommended that the fill soils and any loose native soils be removed and recompacted as part of site preparation prior to the addition of any new fill or structural improvements. Construction of the proposed basement and lower-level rear yard improvements should result in the removal of most of the fill soils. New foundations should be founded into the underlying medium dense Old Paralic Deposits formational soils or properly compacted fill soils. In proposed secondary improvement areas, existing shallow fill soils will require removal and recompaction prior to placement of new fill or improvements.

C. General Opinion of Bluff Stability

The existing coastal bluff is considered stable in its current configuration and, in our opinion, will not be adversely affected or destabilized by the proposed residential construction. In addition, if the residence is constructed in accordance with our recommended 40-foot setback from the bluff edge, it is our opinion a sea wall or base of bluff protective structure would not be required throughout the anticipated 75-year useful life of the new home.

D. Necessity of Observations and Testing during Grading and Construction

Please be aware that the importance of thorough observation and testing during construction should be recognized by the client and the contractor(s) to provide appropriate documentation for any necessary as-graded reports. Recommendations for observation and testing are provided under the "Conclusions and Recommendations" section of this report.

II. SCOPE OF WORK

The scope of work performed for the prior subsurface investigation conducted in 1999 included a site reconnaissance and subsurface exploration program under the direction of our geologist, with placement, logging and sampling of six (6) exploratory handpit excavations. In addition, seven (7) exploratory borings were placed by

Southern California Soil and Testing (SCS&T) in September and November, 1999, and were utilized in our 2000 and 2001 reports (refer to Appendix A for SCS&T boring logs). Drone photography was performed on May 13, 2024 to obtain photographs of the current bluff and beach conditions.

The data obtained from the six 1999 GEI excavations and the seven 1999 SCS&T borings have been utilized in the preparation of this update report. The 2024 photographs were compared to historic photos to evaluate changes in the bluff conditions over time. Refer to Figure No. II, the Plot Plan and Site-Specific Geology map for the 1999 excavation and boring locations.

In addition, for this update report we reviewed available published information pertaining to the site geology, evaluated the bearing characteristics of the encountered surficial fill and formational material, performed geotechnical engineering analysis of the field data, performed slope stability analysis as it relates to the proposed construction, provided bluff recession analysis and locations of the 25-, 40- and 50-foot bluff edge setback lines as determined by prior and current field exploration and analysis, provided the location of the predicted 75-year bluff edge, and prepared this report.

III. SITE DESCRIPTION

The property is known as Assessor's Parcel No. 350-151-10-00, Lot 3 and a Portion of Lot 1 of Judkins Estates, per Map No. 3326, and is addressed as 1720 Torrey Pines Road, in the community of La Jolla, City and County of San Diego, State of California. Refer to Figure No. I, the Vicinity Map, for site location.

The site, consisting of approximately 0.91-acre, is located at 1720 Torrey Pines Road, in the La Jolla area of the City of San Diego. The property is currently developed with a 3,574-square-foot, single-story, single-family residence and associated improvements. It is bordered by developed residential property to the southeast, a densely-vegetated coastal canyon and developed residential property to the northeast, residential property and a coastal canyon to the southwest, and by a coastal bluff descending to the Pacific Ocean to the north and northwest. Access to the subject flag lot is via an asphalt driveway shared by the residential property at 1700 Torrey Pines Road.

For this update report, including preparation of cross sections and slope stability analysis, we have utilized a recent topographic survey of the property prepared by 23. We have also used for reference a topographic survey prepared by Precision Survey and Mapping, dated August 1999, that includes topographic data of the bluff and adjacent coastal canyons to the northeast and southwest of the subject property.

The site topography consists of a northwest-trending ridge descending from the northwest flank of Mount Soledad. The ridgeline and existing structures are bordered by a coastal bluff and beach to the north and northwest, and northwest-trending coastal canyons to the northeast and southwest. Slope gradients on the coastal bluff generally range from near-vertical to approximately 2.0:1.0 (horizontal to vertical), while gradients on the two canyon sidewall slopes generally range from approximately 1.2:1.0 to 2.0:1.0 (horizontal to vertical).

The ridge top and the upper portions of the canyon slopes have been altered by previous grading operations to form two relatively level pads. The lower pad in the rear yard is at an approximate elevation of 70 to 72 feet above mean sea level (MSL). The upper pad supports the existing residence and driveway and is at an approximate elevation of 83 to 84 feet above MSL. Overall topographic relief at the site is approximately 80 feet, with elevations ranging from approximately 10 feet above MSL at the base of the coastal bluff to 90 feet above MSL where the driveway joins Torrey Pines Road.

Vegetation on the site consists of a relatively large rear yard lawn, a moderate to dense growth of native coastal scrub shrubbery on the bounding canyon walls, ornamental shrubbery, small trees on landscaped slopes and in planter areas, and some mature trees on level lot areas and canyon slopes to the northeast.

IV. FIELD INVESTIGATION

The prior field investigation by GEI consisted of a surface reconnaissance of the site and bluff, and a subsurface exploration program utilizing hand tools to investigate and sample the subsurface soils on November 16, 1999. Six exploratory handpits (HP-1 to HP-6) were excavated across the site in areas of the currently proposed residential structure additions, basement, retaining walls, swimming pool and associated improvements. The exploratory handpits were excavated to depths ranging from 1.5 to 5 feet in order to define the soil profile across the site and to obtain representative soil samples. The soils encountered in the exploratory handpits were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (refer to Appendix B). The approximate locations of the GEI exploratory handpits and site-specific geology are shown on Figure No. II, Plot Plan with Site-Specific Geology.

In addition, seven exploratory borings were placed on the site by Southern California Soil and Testing in September and November 1999. The results of the SCS&T borings were utilized in the 2000 and 2001 reports prepared by GEI and have been utilized in the preparation of this 2024 update report. The 1999 SCS&T boring logs (B-1 through B-7) are provided in Appendix A of this report and the SCS&T boring locations are shown on Figure No. II, the Plot Plan with Site-specific Geology.

Representative soil samples were obtained by GEI personnel from the 1999 exploratory handpits at selected depths appropriate to the proposed development of the lot. Soil sampling included in-place samples and bulk samples collected from the exploratory handpits to aid in classification and for appropriate laboratory testing. All samples were returned to our laboratory for evaluation and testing. Exploratory handpit logs were prepared on the basis of our observations and laboratory test results and have been attached as Figure Nos. IIIa-f.

The exploratory GEI handpit logs, SCS&T boring logs, and related information, reveal subsurface conditions only at the specific locations shown on the plot plan and on the particular date designated on the handpit and boring logs. Subsurface conditions at other locations may differ from conditions occurring at the explored 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 soil samples retrieved in 1999 in order to evaluate their physical and mechanical properties and their ability to support proposed residential construction. Since the time of our 1999 exploratory work, ASTM test dates have been changed but the approved test methodology has not changed. We provide below both the test method date from 1999 and the current test method date for reference. The laboratory test results are presented at their respective depths on the excavation logs, Figure Nos. IIIa-f, and IVa-f. The following tests were conducted in 1999 on representative soil samples:

1. *Moisture Content (ASTM D2216-80) (2019)*
2. *Standard Test Method for Bulk Specific Gravity and Density of Compacted Bituminous Mixtures using Coated Samples (ASTM D1188-90) (2015)*
3. *Standard Test Method for Laboratory Compaction Characteristics of Soil using Modified Effort (ASTM D1557-91, Method A) (2012/2021)*
4. *Determination of Percentage of Particles Smaller than #200 Sieve (ASTM D1140) (2017)*
5. *Expansion Index (ASTM D4829) (2021)*
6. *Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions (ASTM D3080-90) (2023)*

Moisture content (ASTM D2216) and density measurements (ASTM D1188) were performed to establish the in-situ moisture and density of samples retrieved from the exploratory handpit excavations. Tests performed by ASTM method D1188 determined the bulk specific gravity utilizing paraffin-coated specimens and helps to establish in-situ density of chunk samples retrieved from the excavations. This information was also used to perform remolded direct shear tests (ASTM D3080).

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

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 expansion potential of the on-site soils was evaluated utilizing the Standard Test Method for Expansion Index of Soils (ASTM D4829). In accordance with the Standard (Table 5.3), potentially expansive soils are classified as follows:

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

Laboratory tests of representative samples of the colluvial soils yielded expansion indices of 93, 56 and 46. Based on the table presented above, the sampled colluvial soils have a low to high potential for expansion. Due to their limited thickness and areal extent, it is recommended that the colluvial soils be removed during grading of the site or be mixed with low expansive on-site sandy soils to produce low to medium expansive fill material.

The clayey sand materials of the underlying Old Paralic Deposits/Bay Point Formation have a tested expansion index of 32, which is considered low expansion potential. Based on our visual classification, our laboratory analysis of representative samples from the site, and our past experience with similar soils, it is our opinion the on-site soils, with the exception of the colluvial soils, can be classified as having a low potential for expansion. The test results are presented on the excavation logs at the appropriate sample depths.

Direct shear tests (ASTM D3080) were performed on remolded soil samples to evaluate strength characteristics of the on-site soils. 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. Assigned shear values were presented in our 2000 report on pages 15 and 16 and in the 1999 SCS&T report on page 10.

Based on the field and laboratory test data, our observations of the primary soil types, and our previous experience with laboratory testing of similar soils, our Geotechnical Engineer has assigned values for friction angle, coefficient of friction, and cohesion for those soils that will have significant lateral support or load bearing functions on the project. The assumed soil strength values have been utilized in determining the recommended bearing value as well as active and passive earth pressure design criteria for foundations and retaining walls.

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). For an extended discussion of Regional Geology, refer to Appendix D.

VII. SITE-SPECIFIC GEOLOGIC DESCRIPTION

A. Stratigraphy

Our field work, reconnaissance and review of the *"Geologic Map of the La Jolla Quadrangle"* contained within California Division of Mines and Geology (now the California Geological Survey) Bulletin 200 *"Geology of the San Diego Metropolitan*

Area, California" (Michael P. Kennedy, 1975) and the updated geologic map by Kennedy and Tan, 2008, *"Geologic Map of San Diego, 30'x60' Quadrangle, CA,"* indicate that the site is underlain at depth and to below the beach elevation by dense, Cretaceous-age Point Loma (Kp) formational soils. These bedrock materials are overlain by Quaternary-age Old Paralic Deposits (Qop_6), formerly identified as Bay Point Formation (Qbp) in our 2000 and 2001 reports. The encountered soil profile over the Old Paralic Deposits/ Bay Point Formation generally consists of a relatively shallow thickness of fill along the northeast edge of the site and surficial colluvium. The older, underlying formational units are exposed in the referenced sea cliffs. Refer to the excavation logs HP-1 through HP-6 (Figure Nos. IIIa-f) and the SCS&T boring logs B-1 through B-7 (Appendix A).

Figure No. V presents a plan view geologic map (Kennedy and Tan, 2008) of the general area of the site and Figure No. VI displays the geologic hazards of the area. Geologic cross sections from our current work that extend to the north from Torrey Pines Road to the base of the bluff and from the west property line to the east property line have been prepared and are included as Cross Sections F-F' and G-G', Figure Nos. VIIa-b. Cross Sections A-A' through E-E' presented in our prior 2000 and 2001 reports have been revised and are presented in Appendix C.

Fill Soils (Qaf): Portions of the lot are overlain by 1 to 5 feet of surficial fill soils encountered at all six handpit locations (HP-1 through HP-6). The fill soils appear to thicken from 1 to 2 feet along the northeastern side of the existing home to 4 to 5 feet along the northeast canyon edge. We note that previous investigators identified thicker fill soils underlying the building pad. In our opinion, based on our 1999 field investigation, the soil materials were mis-identified in the small diameter borings by SCS&T (B-4 through B-7) and the soils comprising most of the pad are natural colluvial soils and Old Paralic Deposits (Qop_6)/Bay Point formational soils. Very localized fill soils up to 18 inches in depth were also reported to exist in the western lawn/yard area where an artificial pond was filled in (and not demolished).

The fill soils consist of light brown to brown, silty sand, brown clayey silt and silty clay, and brown sandy clay. The fill soils are generally loose/soft and moist, with low expansion potential. They are not suitable in their current condition for support of loads from new structures, new exterior improvements, or additional fill and will require removal and recompaction where required to achieve final grades. Refer to Figure Nos. IIIa-f and IVa-f for details.

Colluvium/Weathered Old Paralic Deposits (Qop_6/Qbp): Colluvial soils/weathered old paralic deposits were encountered overlying and grading into the Old Paralic Deposits/Bay Point Formation. The colluvial soils/ weathered old paralic deposits consisted of gray-brown slightly sandy clay, medium dense orange-brown silty fine- to coarse-grained sand, and orange-brown clayey sandy silt. The SCS&T borings describe the material as dark brown silty clayey sand and clayey silty sand. The fine-grained clayey soils are firm to very stiff and of medium to high expansion potential. Refer to Figure Nos. IIIa-f, and IVa-f for details.

Old Paralic Deposits (Qop_6): Old Paralic Deposits, formerly identified as Bay Point Formation (Qbp) in our 2000 and 2001 reports (and in the SCS&T boring logs), were encountered in all the excavations below the fill soils and/or colluvium. These soils underlie the upper portions of the site, including the existing residential pad. The Old Paralic Deposits were encountered at relatively shallow depths and consist primarily of brown to red-brown, medium dense, clayey silty sand/sandy silt. They are generally medium dense to dense (stiff to very stiff), damp to moist, and are considered suitable for support of loads from structures or additional fill. The Old Paralic Deposits observed in the upper portion of the bluff face overlie the Point Loma (Kp) Formation. Refer to Figure Nos. IIIa-f, IVa-f, and VIIa-b for details.

Point Loma Formation (Kp): Cretaceous-age Point Loma Formation underlies the site at depth. The dense sandstone of the Point Loma Formation is best viewed in the sea cliff exposures at the northwest end of the subject property. Hard clayey silt/ silty clay of the Point Loma Formation was encountered in the SCS&T borings although at the time it was misidentified as Ardath Shale (refer to Appendix A for the SCS&T logs of Borings B-1 through B-7 and Figure No. II the Plot Plan for the SCS&T boring locations). The Point Loma formational materials as exposed in the bluff face can appropriately be referred to as siltstone and sandstone, and possess very good inherent strength and bearing characteristics.

B. Structure

The Cretaceous-age Point Loma Formation (Kp) underlies the site and extends in depth to below the beach elevation. The geologic structure underlying the site is relatively consistent as exposed in the face of the sea cliffs, at the bottoms of the small canyons, and as exposed on the sea floor during low tides. Bedding orientations of these Tertiary materials strike approximately N70 W and dip 30 degrees to the south-southwest (refer to Appendix E, Photo No. 1). In general, bedding observed in the overlying shallow-thickness Quaternary Old Paralic Deposits, and based on the mapped Paralic Deposits over Point Loma Formation contact, is horizontal.

Bluff observations and review of the geologic map by Kennedy and Tan, 2008 (Figure No. V), "*Geologic Map of San Diego, 30'x60' Quadrangle, CA,*" indicates that the structural orientation of the Point Loma formational materials nearest the site strike approximately west-northwest and dip approximately 30 degrees to the south/southwest. Refer to Photo No.1 (1999) of Appendix E that shows the bluff face as viewed from the northeast during low tide (bedding dipping to the right highlighted in yellow). Photo No. 2 (1999) of Appendix E shows the low tide-exposed Point Loma Formation bedding trending approximately N70 W and dipping 25 to 30 degrees southwest. These bedding dips are into the canyon bottom bounding the property to the southwest. This is considered favorable from a stability perspective except for the upper approximately 10 feet of the canyon wall/ bluff face where bedding surfaces may not be buttressed by canyon bottom Point Loma Formation. This localized structural condition is most likely responsible for the northwesterly bluff face failure on the northwest portion of the property. Refer to Photo Nos. 3 (1999) and 4 (1999) in Appendix E of the northwestern bluff failure with the failure outlined in yellow. Refer to Photo Nos. 5 (2024) and 6 (2024) of Appendix E with the failure outlined in yellow and showing the failure location relative to the rear yard.

It is our opinion, based upon our field observations and investigation, which are in agreement with the Kennedy and Tan geologic map, that the Point Loma Formation possesses favorable geologic structure at the location of the planned residential structure remodel, additions and improvements. It is therefore our opinion that the geologic structure of the site is stable and suitable to support the proposed remodel, additions, and associated improvements.

VIII. GENERAL GEOLOGIC HAZARDS

Our review of the City of San Diego Seismic Safety Study Geologic Hazards Map Sheet No. 29 (Figure No. VI) indicates that the site is not located in an *Alquist-Priolo Earthquake Fault Zone,* which determines zones of required fault investigation, nor in an active fault buffer zone. The references indicate that the area of study is located within Geologic Hazards Categories (GHC) 43 and 53 and Fault Zone 12.

GHC 43 is mapped on the northwestern portion of the lot (the bluff area) and is described as "generally unstable" coastal bluffs and further defined as "Unfavorable jointing, local high erosion." GHC 53 covers most of the southern/southeastern portion of the lot and is described as "Sloping terrain, unfavorable geologic structure, low to moderate risk." The proposed residential additions are to be constructed beyond the 40-foot bluff setback. Based on our prior field exploration, as well as the references cited, the bedding dipping into the Point Loma Formation canyon bottom that is shown on the geologic maps for the Point Loma Formation in this area by Kennedy and Tan (2008), and our observations of the into-canyon dip of the bedding, it is our opinion that the proposed residential additions will be located in an area of favorable geologic structure and of low to moderate risk. Due to the 40-foot setback of the proposed structure additions, the construction area is, in our opinion, of low risk.

Fault Zone 12 is described as "Potentially Active, Inactive, presumed inactive or activity unknown." A spur of the Mount Soledad Fault is mapped just off the north corner of the property and Fault Zone 12 is mapped crossing the north corner of the 0.91-acre property in a northwest to southeast direction. No development is proposed in the area of the mapped fault zone. In our opinion, the site is not underlain by an active fault and the proposed development is not planned for the north corner of the property at the top of the coastal bluff within the 25 and 40-foot setback areas. Excerpts of the Geologic Map and the Geologic Hazards Map with legends are presented as Figure Nos. V and VI.

A. Local and Regional Faults

The primary seismic considerations for improvements at the subject site are surface rupture of fault traces, damage caused by ground shaking during a seismic event, and seismically-induced ground settlement. The potential for any or all of these hazards depends upon the recency of fault activity and the proximity of nearby faults to the subject site. Our review of the proper literature (CGS, 2021a) indicates that the subject site lies outside the present Earthquake Fault Zones, described in the Alquist-Priolo Earthquake Fault Zoning Act as being placed along active faults.

The site, like most of southern California, is located in a seismically active area and regional faulting is present in San Diego County. The major active faults nearest to the site are all part of the San Diego section of the Newport-Inglewood-Rose Canyon Fault Zone. The following local and regional fault zones are mapped in southern California in general proximity to the site:

- Mount Soledad Fault: The Mount Soledad Fault is mapped approximately 370 feet northeast of the site. A spur of this fault is located just north of the north corner of the subject property. This fault is considered inactive.
- Rose Canyon Fault Zone: The Rose Canyon Fault Zone is mapped approximately 550 feet northeast of the site and is estimated to be capable of generating a M6.9 earthquake (EERI, 2021).
- Coronado Bank Fault Zone: Mapped approximately 12.2 miles southwest of the site and estimated to be capable of a M7.6 earthquake.
- San Diego Trough Fault Zone: Mapped approximately 21 miles southwest of the site. The most recent surface rupture is of Holocene age (SCEDC, 2022).
- Newport-Inglewood Fault: Mapped approximately 23.4 miles northwest of the site, estimated to be capable of producing a M6.0 to M7.4 earthquake (Grant Ludwig and Shearer, 2004; SCEDC, 2022).
- Elsinore Fault Zone: The Julian and Temecula sections of the Elsinore Fault Zone are mapped approximately 37 and 39 miles, respectively, northeast of the site and are estimated to be capable of a of a M6.5 to M7.5 earthquake (SCEDC, 2022).
- San Clemente Fault Zone: Mapped approximately 48 miles to the southwest of the site. The most recent surface rupture is of Holocene age (SCEDC, 2022).
- San Jacinto Fault Zone: Mapped approximately 60 to 67 miles northeast of the site. This fault is estimated to be capable of a M6.5 to M7.5 (SCEDC, 2022).

The potential for strong ground shaking from earthquakes on active southern California faults and active faults in northwestern Mexico should be anticipated 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.

B. Other Geologic Hazards

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

Ground Shaking: Structural damage caused by seismically induced ground shaking is a detrimental effect directly related to faulting and earthquake activity. Ground shaking is considered to be the greatest seismic hazard in San Diego County. The intensity of ground shaking is dependent on the magnitude of the earthquake, the distance from the earthquake, and the seismic response characteristics of underlying soils and geologic units. Earthquakes of M5.0 or greater are generally associated with notable to significant damage. It is our opinion that the most serious damage to the site would be caused by a large earthquake originating on a nearby strand of the Rose Canyon, Coronado Bank or Newport-Inglewood Faults. Although the chance of such an event is remote, it could occur within the useful life of the structure.

Landslides: Our site reconnaissance did not reveal indications of landsliding underlying the building pad or areas to receive structures and improvements. Based on our review of the *Geologic Map of San Diego, 30'x60' Quadrangle, CA* by Kennedy and Tan (2008), the *USGS US Landslide Inventory,* the City of San Diego Seismic Safety Study -- Geologic Hazards Map and aerial photographs (4-11-53, AXN-8M-88 and 89), there are no known or suspected large-scale ancient landslides located beneath the site. A small bluff face landslide is mapped on the adjacent property near the northwest corner of the subject property. The failure is limited to the bluff face as shown in Photo Nos. 3, 4, 5, and 6 of Appendix E.

Slope Stability: Based on our knowledge of the on-site soils and the anticipated grading work required for the proposed residential project, it is our opinion that the site would be adequately stable, and the proposed construction would not adversely affect the stability of the coastal bluff, coastal canyon slopes, or adjacent properties. Furthermore, our slope stability calculations for gross and shallow analysis yield factors of safety higher than the acceptable minimum of 1.5 and 1.15 with seismic loading (refer to Section XIII, Slope Stability Analysis and Appendix F for details).

Due to the favorable geologic structure, the inherent rock strength of the underlying formation materials, and the high rock strengths of the Point Loma Formation at the base of the bluff and in the canyon bottom, it is our opinion the subject project would not be adversely affected by deep-seated slope stability issues that would adversely affect the property beyond the bluff edge. Due to localized, over-steepened portions of the bluff face, periodic surficial bluff face failures should be expected.

<u>*Liquefaction*</u>: 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 very low due to the density of the underlying Point Loma Formation materials and lack of shallow static groundwater. The groundwater surface was encountered between 25 and 30 feet in depth as depicted on Cross Section A-A' of Appendix C. In our opinion, the site does not have a potential for soil liquefaction or soil strength loss to occur due to a seismic event.

<u>*Tsunami and Seiches*</u>: A review of the California Geological Survey's 2009 *"Tsunami Inundation Map for Emergency Planning, La Jolla Quadrangle, San Diego County."* indicates that the site is not mapped within a possible inundation zone. The limit of the tsunami inundation zone for this site is along the exposed beach to the northwest and below the residence. The risk of a tsunami affecting the site is considered low as the site is situated at an elevation of approximately 70 to 80 feet AMSL. In general, the orientation of the southern California coastline and the bathymetry of the offshore southern California borderland have, during historical times, combined to protect the shoreline from any large magnitude tsunami height increases, as shown by records of tsunami occurrences that have been observed and/or recorded along the southern California shoreline since 1810 (Lander et al., 1993). For this segment of the California coastline (south of Santa Monica), there is no evidence of any high magnitude tsunamis generated during the last 200 years by large-scale regional sea floor movements (Gayman, 1998).

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. There are no significant bodies of water located at higher elevation or in the general vicinity capable of producing a seiche and inundating the subject site.

Flooding: Review of FEMA Flood Insurance Rate Map number 06073C1582H, effective 12/20/2019, indicates the site is within Zone X, described as *"Areas determined to be outside the 0.2% annual chance floodplain."* It is our opinion that the risk of flooding does not exist at the site.

Sea Level Rise: Sea level rise for this area over the next 75 years (design life of the structure) is projected to range from between 1.25 feet and 4.75 feet (August 2015 California Coastal Commission Sea-Level Rise Policy Guidance). The building pad is at approximate elevation of 70 to 85 feet above Mean Sea Level. We consider the maximum design water elevation for sea level rise to be the historical water elevation of approximately 5.5 feet added to 4.75 feet (the maximum approximate sea level rise), or an approximate total elevation of 10.25 feet. Using an average predicted sea level rise of 3 feet (1.25 to 4.75 feet) would result in a design water elevation for sea level rise of 8.5 feet. Due to the elevation of the site at 70 to 85 feet AMSL, the residence and improvements will not be subject to flooding from ocean water levels. Furthermore, in our opinion, the improvements at the building pad elevation of approximately 82.5 feet AMSL would not be adversely affected by wave run up.

Summary: In our opinion, no significant geologic hazards are known to exist on the site that would prohibit the construction of the proposed residential additions and associated improvements. Periodic failures of localized sections of over-steepened bluff face should be expected. However, the 70-foot-high bluff is, in our opinion, grossly stable and would not adversely affect the proposed structure additions during the anticipated useful life of 75 years.

Ground shaking from earthquakes on active southern California faults and active faults in northwestern Mexico is the greatest geologic hazard at the property. Design of the residential structure additions in accordance with the current building codes will reduce the potential for injury or loss of human life. Structures constructed in accordance with current building codes may suffer significant damage but should not undergo total collapse. Refer to Section XV (subsection B) and Appendix G of this report for seismic design criteria.

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

IX. GROUNDWATER

Groundwater was not encountered in the exploratory handpits placed in the general area of the proposed residential additions and improvements and we do not anticipate significant shallow depth (perched) groundwater problems to develop in the future if the improvements are developed as proposed and proper drainage is implemented and maintained. Groundwater or "perched" groundwater was encountered at depths of 25 to 30 feet with a surface gradient to the north in the three borings placed by SCS&T in 1999. Refer to the SCS&T boring logs provided in Appendix A and cross section A-A' (Appendix C).

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

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

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

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

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

X. <u>MAP AND AERIAL PHOTO DATA SOURCES</u>

Review of a series of relatively low-quality historic aerial photographs depicting the site and adjacent properties was performed. These photographs have been utilized to examine and measure areal and coastal features around the Lowenthal property. These photographs include high-angle and oblique/low-angle photographs that have been enlarged where practical without loss of original low-quality resolution. Orthophotographic map information and site plans were also utilized to supplement the photographic research. The photograph sources include government agencies, the San Diego Historical Society, the La Jolla Historical Society, our area photographs, and the private firm of Landiscor Aerial Information that maintains an aerial photograph library. Copies of the historic photographs (1928-1966), in addition to photos taken in 1999 and recent photos (2024), are provided in Appendix E of this report (and were originally presented in Appendix C – Historic Photographs, in our 10/3/2001 report). A list of the historic photographs is as follows:

Date	Description/Type	Source
1928-29	N/A/high-angle/low resolution	Landiscor-Aerial Photobank
1/26/35	79:741-281/low altitude, low angle	San Diego Historical Society
11/3/36	79:741-152/high angle, low altitude	San Diego Historical Society
2/16/49	N/A/high angle, low resolution	Landiscor-Aerial Photobank
4/11/53	AXN-8M-88/high angle, high altitude	USDA/Landiscor-Aerial Photobank
4/11/53	AXN-8M-89/high angle, high altitude	USDA/Landiscor-Aerial Photobank
10/16/65	2552/low angle, low altitude	Landiscor-Aerial Photobank
12/9/66	47-4883/high angle, high altitude	Landiscor-Aerial Photobank
12/23/99	N/A/sea level-low tide bluff views	GEI
Various	California Coastal Records Project Aerial Photos	

The following map sources of information were also utilized in our analysis:

Date	Description/Type	Source
1901-02	USGS Quadrangle La Jolla/ Topographic Map 1:24000	US Dept. of the Interior
1930	USGS Quadrangle La Jolla/ Topographic Map 1:24000	US Dept. of the Interior
1943	USGS Quadrangle La Jolla/ Topographic Map 1:24000	US Dept. of the Interior
1975	USGS Quadrangle La Jolla/ Topographic Map 1:24000	US Dept. of the Interior
2/2/78	Topographic Map 246-1683, 1"=200' (Revised 1963)	City of San Diego
6/22/79	Orthophotographic Map 246-1683 1"=200'	City of San Diego
8/99	Topographic site survey, 1"=40'	Precision Survey & Mapping
2008	Geologic Map of San Diego, 30'x60' Quad	Kennedy and Tan
2008	City of San Diego Seismic Safety Study Geologic Hazards Map	
12/5/2023	Topographic Survey	San Diego Land Surveying

XI. BEACH AND COASTAL BLUFF DESCRIPTION

In general, geologic materials that comprise the northern portion of the site, including portions of the intertidal and supratidal beach as well as the coastal bluffs, consist of two types: Quaternary Old Paralic Deposits (Qop_6)/Bay Point Formation (Qbp) that underlie upper portions of the existing pad and portions of the upper northern part of the site; and the Cretaceous Point Loma Formation that underlies the Old Paralic Deposits/Bay Point Formation and forms most of the coastal bluff. It also forms the foreshore area of the coast along which a seasonal sand and/or cobble beach exists, as well as offshore intertidal and subtidal ledges.

This section of coastal La Jolla east of Point La Jolla and south of La Jolla Shores, is referred to as the La Jolla Cove area. It is characterized as rocky headlands and rocky, wave-cut low-tide terraces fronted by perched narrow sandy beaches or narrow cobble and boulder beaches. Single-family residences are commonly located along the bluff tops. The Quaternary Old Paralic Deposits/Bay Point Formation appears to be relatively shallow at the site. It comprises the very upper portion of the coastal bluff.

The Upper Cretaceous Point Loma Formation comprises the sea bluff and is visible as outcrops on the coastal portion of the site, as well as to the east and west along the coast in this area of La Jolla. It consists of interbedded, dense, fine- to medium-grained sandstone. It is well indurated in its lower portion. These materials generally strike northwesterly, with southwesterly dips up to 25 to 30 degrees in the vicinity of the site.

The Point Loma Formation has formed generally subvertical to 1.0:1.0 and shallower (3.0:1.0) bluffs 55 to 60 feet high at the site.

Recession of the coastal bluff at the site would, over time, be associated with erosion of the Cretaceous Point Loma Formation, which forms most of the bluff and beach terrace below.

Rates of erosion of the Cretaceous sandstone have been examined by various researchers. Emery (1941) determined the rate of erosion to be about 0.02-foot/year for sites along the northern La Jolla shoreline, and Kennedy (1973) determined rates of erosion in the Sunset Cliffs area to be 3 to 4 feet/century, or 0.03-0.04 feet/year.

The rate of gradual erosional undercutting and wearing away of the bluff is usually distinct from the block fall recession rate of the bluff. Elsewhere along this section of coastline, fractured or jointed materials comprising the bluff have receded more rapidly through block fall mechanisms than true erosion would yield, creating the irregular shoreline visible along this section of La Jolla today.

Rates of erosion inherent to the exposed site bedrock (the Point Loma Formation sandstone and shale) comprising the subject bluff were calculated utilizing surveyed topographic information from 1999 compared to earlier photographs that show the current position of observable outcrop and the bluff retreat relative to the known age of improvements at the site.

Measured erosional recession for the coastal bluff at the site range from 1 foot up to 5 feet over a 34-year period (since 1966). Field measurements were compared to current topographic measurements. Additionally, scaled measurements were approximated utilizing the referenced maps and photographs. Existing improvements were measured to older topographic contours and newer topographic contours. Utilizing the measured erosional recession, rates of bluff erosion for the site were calculated to range from 0.03 to 0.15 feet per year. An average erosion rate of 0.09 feet per year yields an estimated 6.75 feet of erosion over the past 75 years.

It is well known that block fall or mass wastage is usually the controlling factor in bluff recession along most of the San Diego County coastline. Undercut and blockfall retreat rates are anticipated to increase the overall recession of the bluff to no greater than 10 feet in the past 75 years. It is our opinion that the bluff face and site should be stable inland of the proposed 40-foot setback for a period of at least 75 years.

XII. COASTAL BLUFF EVALUATION

A. Project-Specific Bluff Descriptions

The bluff along the northwest side of the subject property, which includes the bluff face failure, extends approximately 65 to 70 down to the foreshore area of the coast along which a seasonal sand and/or cobble beach exists, as well as offshore intertidal and subtidal ledges. The exposed bedrock configuration ranges from moderately sloping surfaces in the upper terrace deposits, to steeply-inclined lower bluff faces to 25 to 30 feet in height. No out-of-slope dip components were noted that would adversely affect slope stability deeper than 10 to 15 feet into and parallel to the bluff face. For reference purposes we have included as Appendix I, the "*Coastal Bluffs and Beaches Guidelines*" from the City of San Diego Municipal Code (pages 16-20).

In general, the geologic materials that comprise the north and northwestern portion of the site, including portions of the intertidal and supratidal beach (seasonally overlain by sand and/or cobble and boulders), as well as the approximately 65 to 70-foot-high coastal bluff, consist of two types: terrace materials of the Quaternary Old Paralic Deposits (Qop_6) that underlie the building pad and are exposed in the upper portion of the bluff; and the Cretaceous Point Loma (Kp) that underlies the Old Paralic

Deposits and forms the sea cliff. The Point Loma Formation also forms the foreshore platform area of the coast along the upper edge of which a seasonal sand and/or cobble and boulder beach exists, as well as offshore intertidal and subtidal ledges. Refer to Photo 2 of Appendix E which reveals the consistency of the southwesterly Point Loma Formation bedding as well as its erosion resistance.

The Cretaceous Point Loma Formation is visible from the beach below the site, as well as to the north and south along the coast in this area of La Jolla. As mapped by Kennedy and Tan (2008), these materials generally strike east-west with easterly dips up to 30 degrees.

B. __Bluff Morphology__

The sea bluff bounding the northwestern edge of the property is approximately 65 to 70 feet high and rises at generally subvertical to 1.0:1.0 and shallower (3.0:1.0) from the gently-sloping beach to the landscaped backyard. The upper portion of the bluff above an elevation of approximately 65 feet (MSL) is composed of fill soils underlain by Old Paralic Deposits (Qop_6)/Bay Point Formation (Qbp) of Late Pleistocene age. Observation of the lower portion of the bluff reveals southerly-dipping Cretaceous Point Loma Formation. Refer to Photo 1 of Appendix E.

C. __Regional Point Loma/Cabrillo Formation Bluff Descriptions__

This section of coastal La Jolla, referred to as the La Jolla embayment, is characterized as rocky headlands and rocky, wave-cut low-tide terraces fronted by perched narrow sandy beaches or narrow cobble and boulder beaches. Single-family residences are usually located on top of the bluffs. The *Shoreline Erosion Assessment and Atlas of the San Diego Region, Volume II*, prepared by California Department of Boating and

Waterways and San Diego Association of Governments (1994) further profiles this area of the La Jolla coastline as having "*moderate risk.*"

The Point La Jolla and Point Loma shorelines bulge substantially to westward, with respect to the County shorelines to the north and south. The primary cause for this bulge is tectonic uplift that probably began more than one million years ago. However, a secondary cause is due to the resistant Cretaceous bedrock that outcrops at sea level everywhere along the shoreline from the Marine Room restaurant in La Jolla Shores to Bird Rock Bay in south La Jolla. If the La Jolla sea cliffs (primarily to the south of Point La Jolla) were as easily subject to erosion as the less resistant Tertiary formations north of Scripps Institution of Oceanography (SIO) located at the north end of La Jolla Shores, then one would expect the La Jolla cliffs to be much higher.

The sea cliffs north of SIO range from 70 to 350 feet in height, while the cliffs along the shoreline from the Marine Room restaurant in La Jolla Shores to the La Jolla Cove (the La Jolla embayment) range from 10 to 30 feet in height and the cliffs from Point La Jolla and Bird Rock Bay are generally less than 15 and 25 feet in height. South of Bird Rock, where the Cretaceous rock again gives way to Tertiary formations, the cliffs are higher (30 to 35 feet) and the shoreline recedes gradually eastward. Both the higher cliff shorelines north of SIO and the low cliff shorelines south of Point La Jolla have been exposed to wave erosion for equal periods of time, i.e., during the last 5,000 to 7,000 years. Before 6,000 to 7,000 years ago, sea level was too low (for a period of perhaps 20,000 to 30,000 years) to permit the erosion of the existing sea cliffs. Thus, the shoreline configuration and the cliff heights tend to support the considerable ability of the Cretaceous siltstone and sandstone to resist marine erosion.

D. *Upper Bluff Edge Location*

At the time of our work in 1999, we were provided with a topographic survey prepared by Precision Survey and Mapping, dated August 1999, that includes topographic data of the bluff and adjacent coastal canyons to the northeast and southwest of the subject property. For this update report, including preparation of cross sections and slope stability analysis, we utilized a recent topographic survey of the property prepared by San Diego Land Surveying and Engineering, Inc., dated December 5, 2023 and the 1999 topographic survey of the bluff and adjacent coastal canyons to the northeast and southwest of the subject property. They have been combined and utilized as Figure No. II of this report.

The topographically well-defined bluff edge, as well as the 25- and 40-foot setback, were presented in our 2001 report. In addition to the previously presented setback lines we have also included a 50-foot setback. Information regarding the bluff edge location and setbacks, as well as the location of borings and excavations placed to obtain soil samples and to define the bluff edge, are shown on Figure No. II.

The previously placed test pit excavations by GEI (HP-1 and HP-5) and SCS&T borings (B-2, B-3 and B-6) were placed in the lower rear yard area above the bluff top in an effort to confirm the bluff edge location. The excavations encountered shallow depths of fill soils and colluvium/weathered Old Paralic Deposits underlain by medium dense Old Paralic Deposits/Bay Point and dense Point Loma formation (Kp). The excavations confirmed that the clear topographic break in slope is, in fact, the bluff edge. We have, therefore, used this excavation information to show the bluff edge extending beyond the proposed building area and around the northwestern end of the landscaped yard area.

See Figure No. II for the location of the existing home, the location of the proposed additions, and the 1999 GEI handpit and SCS&T boring locations. Based on our site observations and excavation information, the bluff edge location is shown on Figure No. II. In summary, the bluff edge is well defined and, in our opinion, the 25- and 40-foot setbacks shown in this report are accurate.

Based on our field investigation, as well as our historic topographic map and aerial photo research, it is our opinion that the coastal bluff edge on the subject property is defined as described in the "*Coastal Bluffs and Beaches Guidelines*" by the point at the top of the approximately 65- to 70-foot-high coastal bluff *"where the downward gradient of the land surface begins to increase more or less continuously until it reaches the general gradient of the coastal bluff face."* (Refer to Appendix I for a presentation of this report *Coastal Bluffs and Beaches Guidelines,* Section I, D.)

Excavations placed in the rear yard revealed the bluff top break in slope and bluff edge to be as we have indicated on the Plot Plan, Figure No. II, as well as on Cross Sections F-F' and G-G' (Figure Nos. VIIa-b) and Cross Sections A-A' and B-B' of Appendix C.

E. ***Lower Bluff Geomorphology***

Base of bluff geomorphology must be considered in the determination of construction setbacks from the bluff edge. In cases where the landward bluff face undercut or "notching" due to wave impact erosional processes extends further inland than the lower bluff face or bluff-top bluff edge location, the greater landward extent of the two must be considered in slope stability and bluff recession evaluations.

The subject property toe of bluff has not undergone significant notching due to the cemented condition of the Point Loma Formation and northwest facing orientation of the bluff. Furthermore, the cobble- and boulder-covered beach and near shore shallow depth bedrock surface (refer to Photo Nos. 1 and 2 of Appendix E) provide significant protection from high tide and storm wave events.

F. ***Bedrock Strength and Erosion Resistance Factors***

As always with proposed coastal bluff top construction, bluff face geologic stability as well as bluff recession mechanisms and rates are significant factors to be considered in site development and determination of building setbacks. Evaluations must be made of inherent strengths of the Point Loma Formation and Old Paralic deposits (Bay Point/Marine terrace deposits), as well as their highly variable response to coastal erosion processes depending on lithologic variations and degrees of faulting and jointing.

Rock strength characteristics for the Point Loma Formation as it exists below and to the immediate north and south of the subject property are largely responsible for the favorable site stability. The cemented sandstone/siltstone possesses good strength characteristics. In addition, the coastal configuration in the area of the site is favorable to the relative long-term stability of the bluff (and is reflected in the bracketed rates of bluff erosion we have calculated and discussed below). The primarily westward orientation of the localized bluff face failure provides a degree of protection to that portion of the bluff from winter storms, which arrive from the northwest. The shallow depth wave-cut bedrock surface extending offshore also serves to reduce the impact of high tide and storm-generated waves. Relative risk from wave-generated bluff erosion is considered to be less for the subject property than at other La Jolla coastal locations with less cobble on the beach and less nearshore bedrock outcrop protection. A Google Earth image of the subject property and the adjacent easterly and westerly properties has been included as Figure No. VIII.

G. Historic and Measurable Erosion Rates of Sea-cliff Recession

Published rates of sea-cliff recession for the San Diego area such as by Kennedy (1973) indicate a wide variation in erosion rates that are primarily dependent on local geologic conditions. Kennedy's 1973 study concerning the rate of erosion of Cretaceous rocks in the Sunset Cliffs area is the most pertinent for this site since the bedrock types are similar in age and degree of cementation. In that study, Kennedy showed that approximately 75 percent of the San Diego County sea-cliffs studied underwent "retreat" of as much as 10 feet in 75 years (0.13 ft/year). The average rate of recession for the Sunset Cliffs area was determined to be approximately three feet for the 75-year period (0.04 ft/year).

As presented by SCS&T in their 1999 report for the subject property, review of several previously prepared reports for residences in this area of the La Jolla coast indicates that estimated bluff erosion rates for properties with similar geologic conditions varied from 0.5 to 2 inches per year (0.04 to 0.17 ft/year), or 3 to 12.75 feet per 75 years.

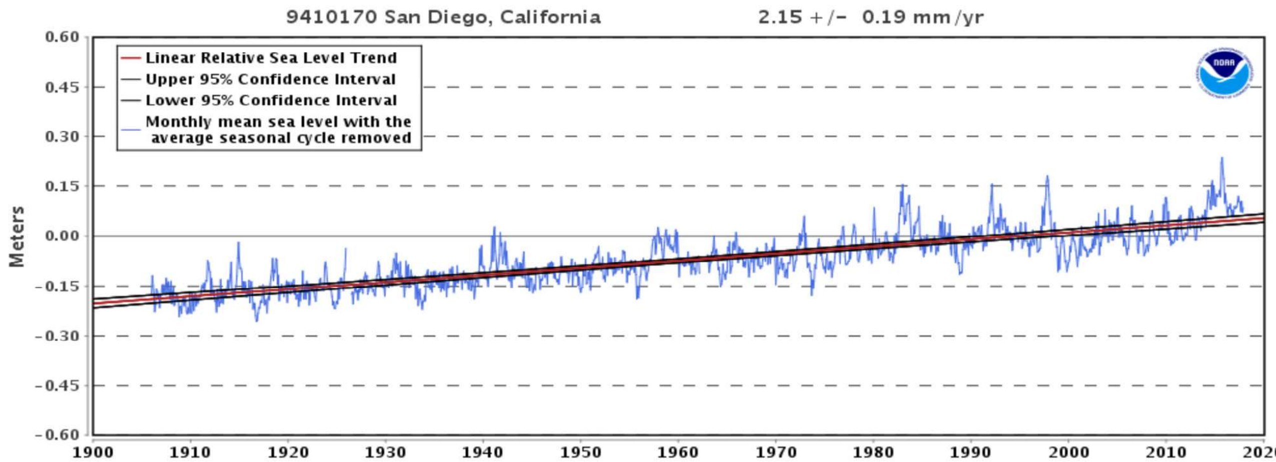
Although average rates of bluff recession are typically reported in feet per year or inches per year, the typical sea-cliff retreats in the form of block-falls several feet thick that are widely separated in time. Block-falls typically occur when a wave-cut notch at beach level extends into the bluff a distance sufficient to intercept joints in the rock that are parallel to the bluff face. Such joints may be spaced at intervals varying from two to approximately 10 or more feet dependent on rock type and proximity to faults or folds. In the case of the subject property, basal bluff erosion notching is nominal. The potential from upper bluff block falls of the weaker terrace deposits is due to upper bluff oversteepening due to the localized bluff failure and not basal bluff notching.

Measurable Rates of Sea Cliff Recession: Rates of erosion of the Cretaceous coastal bedrock have been examined by various researchers. Emery (1941) determined the rate of erosion to be about 0.02-foot/year for sites along the northern La Jolla shoreline, and, as mentioned previously, Kennedy (1973) determined rates of erosion in the Sunset Cliffs area to be 3 to 4 feet/century, or 0.03-0.04 foot/year.

The rate of gradual erosional undercutting and wearing away of the bluff base is distinctly different from block fall recession mechanisms and especially from upper bluff block falls due to oversteepening. Although localized periodic block falls give the appearance of more rapid recession, the long-term average is still controlled by the true rate of erosion and undercutting of the base of the bluffs.

H. Projected Future Bluff Edge Retreat and Sea Level Rise

Sea levels have been rising worldwide for the last 18,000 years or since retreat of Wisconsin-age glaciers approximately 18,000 years ago. At the glacial maximum, sea levels were approximately 400 feet lower, and since that time sea level data show relatively rapid rise of about one meter per century from about 18,000 years ago to about 8,000 years before present (Masters and Fleming, 1983). Then approximately 8,000 years ago, the rate of sea level rise slowed, ultimately to a nearly constant rate of approximately 10cm/century. The long-term historical sea level change record captured at the Scripps Institution of Oceanography shows that sea level in San Diego County since 1900 has been rising linearly up to the present at a mean rate of 2.15 mm/yr (± 0.19mm/yr) or approximately 8.5 inches since 1906.



Historic Sea Level Rise at Scripps Institution of Oceanography pier, 1906 to 2018.

The effect that sea level change will have on low-lying areas such as developed areas near river mouths will be significant. However, the effects of sea-level change on property situated high on coastal bluffs will be minimal. As stated in Chapter 5 of the USACE Coastal Engineering Manual (CEM) "*with some regional exceptions, sea level is not rising at a rate to cause undue concern for coastal development situated high above the ocean on resistant coastal bluffs*." The current standard of practice for coastal engineering contained in the 2004 USACE Coastal Engineering Manual (CEM) uses a 4.3-inch rise for the west US coast sea level for the next 100 years.

A paper by Cayan et al (2008) entitled "Climate Change Projection of Sea Level Extremes along the California Coast" provided a range of sea level rise from 4.3 inches to 28 inches over the next 100 years for the San Diego area. We note that a current project in Del Mar utilized the upper end of that range; a very conservative 24-inch rise in sea-level over the life of the project (Ben Benumof, personal communication, 2018). Benumof also concluded that a 2-foot rise in sea-level (concluded to be the worst-case scenario for sea-level change in the San Diego area) will simply shift the beach profile upwards and landward, but only landward if appreciable erosion occurs at the toe of the sea-cliff. Given the degree of cementation and shear strength of the Point Loma Formation and the lack of discernible basal bluff notching, it is our opinion that even the postulated worst-case sea-level rise will not result in appreciable accelerated erosion given the northwesterly- and westerly-facing bluff conditions below the subject property and the localized bluff face failure. For the purposes of this project, we are assigning a conservative predicted recession rate of 0.17 feet per year, or 12.75 feet in 75 years. The 75-year bluff edge location is presented on Figure No. II along with the assigned 25-, 40-, and 50-foot setback locations.

XIII. SLOPE STABILITY ANALYSIS

Slope stability analysis was performed along cross sections F-F' and G-G' through the property and coastal bluff. The cross sections are included as Figure Nos. VIIa-b. We performed the gross stability calculations using the *SLIDE 6* program by RocScience. The program is a limited equilibrium slope stability program that allows the use of several slope stability methods to calculate the factors of safety against shear failure. On this project we used the Bishop Simplified method as the basis for calculations when using both circular and a hypothesized block failure surfaces through the site geologic cross section. The graphic printouts of our slope stability analyses are provided in Appendix F. For performance of the slope stability analysis we utilized the following soil strength factors:

	Angle of Internal Friction	***Cohesion***	***Soil Weight***
Existing Fill Soils	32 degrees	150 psf	120 pcf
Qop6/Qbp	30 degrees	300 psf	125 pcf
Point Loma Formation	35 degrees	1,500 psf	128 pcf

We utilized cross sections F-F' and G-G' for our analysis because of their locations through the steepest most sensitive locations on the bluff face. We performed our analysis for both static and under seismic conditions.

As shown by the printouts provided in Appendix F, which address the relevant bluff face conditions, neither static or seismic circular or block failure analysis result in a factor of safety below 1.5 out to the existing bluff edge and face of the bluff. Bluff recession over a period of 75 years would, therefore, be limited to our conservatively assigned lower bluff face recession of 12.75 feet in 75 years and is well within the proposed project setback of 40 feet.

XIV. __BLUFF EDGE SETBACK SUMMARY CONCLUSIONS__

Based on the geotechnical and geologic work performed by our firm and SCS&T beginning in 1999 and extending to the date of this update report, the bluff edge location is topographically well-defined and has been confirmed by placement of handpit and boring excavations. The toe-of-bluff recession rate and erosion potential are nominal due to the lower bluff highly cemented Point Loma Formation reducing the impact of high tide and storm wave erosion. The high-strength Point Loma formational materials result in static and seismic loading factors of safety exceeding 1.5 and 1.15, respectively at the bluff edge and factors of safety below 1.5 and 1.15 do not extend beyond the bluff edge into the building pad. It is therefore, our opinion, that the proposed setback of 40 feet for the structure additions and improvements project is adequate.

As is the case with all bluff top construction projects, all surface water drainage systems must include the collection and transmission of collected water to street discharge. No surface water flow, should be allowed over the top of the bluff.

XV. PROJECT GEOTECHNICAL RECOMMENDATIONS

The following recommendations addressing the geotechnical aspects of the proposed residential construction are based upon the prior field investigations and laboratory tests conducted in 1999 and our review of current onsite conditions, in conjunction with our knowledge and experience with similar soils in the La Jolla area of the City of San Diego.

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

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

It is our opinion, based on our current understanding of the proposed construction, that the area of study is suitable for the planned residential additions and associated improvements as long as the recommendations herein are incorporated during design and construction. *Care must be taken in the performance of grading operations to protect the bluff face.*

A. Preparation of Soils for Site Development

The following site preparation recommendations are provided:

1. *General:* Grading should conform to the guidelines presented in the 2022 California Building Code (CBC) as well as the requirements of the City of San Diego. During earthwork, removal and reprocessing of fill materials, as well as general grading procedures of the contractor, should be observed and the fill selectively tested by representatives of the geotechnical engineer, ***Geotechnical Exploration Inc***. If any unusual or unexpected conditions are exposed in the field, they should be reviewed by the geotechnical engineer and, if warranted, modified and/or additional remedial recommendations will be offered. Specific guidelines and comments pertinent to the planned development are provided herein.

   The recommendations presented herein have been completed using the information provided to us regarding site development. If information concerning the proposed development is revised or any changes in the design and location of the proposed additions is modified after issuance of this report, this office should be notified and the changes should be evaluated to determine if the recommendations presented in this report still apply.

2. *Clearing and Stripping:* In areas to receive the new additions, basement foundations and improvements, all existing obstructions as well as any existing landscaping should be removed. This includes the complete removal of all surface and subsurface obstructions (concrete footings, existing utility lines and miscellaneous debris, irrigation systems etc.) that may exist in these areas. After clearing the entire ground surface, the site should be stripped of

existing vegetation within the area of proposed construction. This includes any roots from existing trees and shrubbery.

Once the required excavations are made down to suitable soils, 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 below. Prior to any filling operations, the cleared and stripped vegetation and debris should be disposed off-site.

3. *Excavation:* After the pertinent areas of the site have been cleared and stripped, all existing fill soils and colluvium/weathered paralic deposit soils in the area of the proposed additions and associated improvements should be removed. In the basement areas, the excavation is anticipated to be approximately 12 to 14 feet deep including foundation depths. Shoring will be required where temporary sloping excavations may not be implemented due to space restrictions. A 1:1 temporary slope may be used on the sides of the basement excavation away from adjacent existing improvements.

   It is anticipated that the depth of site preparation disturbances requiring removal and recompaction for exterior improvements will be approximately 3 to 5 feet below existing grade. Deeper excavation will be needed for construction of the basement and swimming pool down to the proposed grade.

   Based on our experience with similar materials, it is our opinion that the existing fill soils and formational 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. Contractors should also review this report along with the handpit and boring logs to understand the scope and quantity of grading required for this project. Variability in excavating the subsurface materials should be expected across the project area. Undercutting may be recommended at time of grading if shallow portions of fill removal are encountered in an area and deeper fills in any other areas or if medium expansion potential soils are encountered to be too low in moisture content that needs to be increased.

The areal extent required to remove the surficial soils and disturbed existing fill 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 bearing on fill soils and any areas to receive exterior improvements where feasible, or to the depth of excavation or fill at that location, whichever is greater.

4. *Temporary Slopes*: Temporary slopes needed for basement retaining wall construction and/or removal and recompaction during site grading should be stable for a maximum slope ratio of 1.0:1.0 (horizontal to vertical) to a maximum height of 12 to 14 feet for soils possessing a minimum cohesion of 50 psf. Some localized sloughing or raveling of the soils exposed on the temporary 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 his methods of operation. No soil stockpiles or surcharge may be placed within a horizontal distance of 10 feet from the excavation.

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.

5. *Slope Observations*: A representative of ***Geotechnical Exploration, Inc.*** must observe the northwesterly-descending bluff and any steep temporary slopes *during construction*. In the event that soils and formational material comprising a slope are not as anticipated, any required slope design changes would be presented at that time. Where not superseded by specific recommendations presented in this report, trenches, excavations, and temporary slopes at the subject site should be constructed in accordance with Title 8, Construction Safety Orders, issued by Cal-OSHA.

6. *Subgrade Preparation:* After the proposed residential structure area has been cleared, stripped, and the required excavations made, the exposed approved subgrade soils in areas to receive new fill and/or slab on-grade 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 soil materials in the building areas, they should be excavated out. On site medium expansive soils may be mixed with imported low expansive soils, and moisture conditioned to at least 3 percent for medium expansive soils.

7. *Material for Fill:* If encountered on-site medium 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. Existing on-site or imported 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 import materials for use as fill should be approved by our representative prior to importing to the site.

   Backfill material to be placed behind site retaining walls should be low expansive soils (E.I. less than 50), with rocks no larger than 3 inches in diameter. Though not anticipated, highly expansive clayey soils if encountered on-site within the upper 5 feet should be fully removed and replaced with imported granular sandy soils before placement and compaction. Where shoring is used, permanent retaining walls should be designed for existing soils, medium to highly expansive, as applicable. Imported fill material should have a low expansion potential.

   During building pad preparation and if encountered, any cobble over 6 inches in diameter should be removed from the excavated soils. In addition, imported (if necessary) 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.

8. *Structural Fill Compaction:* All structural fill, and areas to receive any associated improvements, should be compacted to a minimum degree of compaction of 90 percent based upon ASTM D1557-12e1. 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. Low expansive granular soils should be moisture conditioned at 2 percent of optimum moisture content, and when mixed with medium expansive soils to at least 3 percent above optimum moisture content.

   Soil compaction testing by nuclear method ASTM D6938-17a should be performed every 2 feet or less of fill placement by a representative of ***Geotechnical Exploration, Inc.*** Furthermore, our representative should perform necessary observation of fill placement during grading operations throughout the project.

   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 during placement and 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.

9. *Water-Soluble Sulfate and Chloride Testing*: We recommend that the water-soluble sulfate content and chloride content of the near-surface soils be tested at the completion of grading or foundation excavations. The test results should be evaluated by an engineer specializing in corrosivity. Cement type recommendations should be provided by the structural engineer based on the current edition of the CBC (2022) or the American Concrete Institute and the soluble sulfate and chloride test results.

10. *Trench Backfill:* All utility trenches should be backfilled with properly compacted imported fill or low expansive on-site soils, but capped (upper 8 inches) with properly compacted on-site soils. Imported 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.

11. <u>*Observations and Testing*</u>: As stated in CBC 2022, <u>*Section 1705.6 Soils*</u>*:* "*Special inspections and tests of existing site soil conditions, fill placement and load-bearing requirements shall be performed in accordance with this section and Table 1705.6 (see below). The approved geotechnical report and the construction documents prepared by the registered design professionals shall be used to determine compliance. During fill placement, the special inspector shall verify that proper materials and procedures are used in accordance with the provisions of the approved geotechnical report."*

A summary of Table 1705.6 "REQUIRED SPECIAL INSPECTIONS AND TESTS OF SOILS" is presented below:

a) Verify materials below shallow foundations are adequate to achieve the design bearing capacity;
b) Verify excavations are extended to proper depth and have reached proper material;
c) Perform classification and testing of compacted fill materials;
d) Verify use of proper materials, densities and thicknesses during placement and compaction of compacted fill prior to placement of compacted fill, inspect subgrade and verify that site has been prepared properly.

Section 1705.6 "Soils" statement and Table 1705.6 indicate that it is mandatory that a representative of this firm (responsible engineering 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.

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

The Geotechnical Engineer of Record, in this case ***Geotechnical Exploration, Inc.,*** cannot be held responsible for the costs and time delays associated with the lack of contact and requests for testing services by the client, general contractor, grading contractor or any of the project design team responsible for requesting the required geotechnical services. Request for services is to be made through our office telephone number (858) 549-7222 and the telephone number of the GEI personnel assigned to the project or via email at least 24 hours in advance prior to the needed service visit.

B. Seismic Design Criteria

12. *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.

13. *Seismic Design Criteria:* The proposed structure additions should be designed in accordance with the 2022 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 32.8493 degrees and a longitude of -117.2635 degrees, utilizing a program titled "S*eismic Design Map Tool" and* provided by the USGS through SEAOC, which provides a solution for ASCE 7-16 utilizing digitized files for the Spectral Acceleration maps. Refer to Appendix F for the ASCE Seismic Summary Report.

14. *Structure and Foundation Design*: The design of the new structures and foundations should be based on Seismic Design Category D, Risk Category II for a Site Class C Soils (Very Dense and Soft Rock).

15. *Spectral Acceleration and Design Values*: The structural seismic design, when applicable, should be based on the following seismic soil parameter values, which are based on the site location, soil characteristics, and seismic maps by USGS, as required by the 2022 CBC. Seismic design soil parameters were obtained with the SEAOC Seismic Design Map Tool and they are presented in summarized form below. A full computer printout is presented as Appendix F.

TABLE I
Mapped Spectral Acceleration Values and Design Parameters

S_S	S_1	S_{MS}	S_{M1}	S_{DS}	S_{D1}	Fa	Fv	PGA	PGA_M	SDC
1.399	0.49	1.399	0.887	0.932	0.591	1.0	1.809	0.639	0.702	D

C. Foundation Recommendations

16. *Footings*: We recommend that the new addition footings be supported on continuous spread or isolated foundations bearing on properly compacted fill or medium dense to dense, low to medium expansive formational soils.

All structures footings should be founded on formational soils or properly compacted fill prepared as recommended above in Recommendation Nos. 4, 5 and 6. All footings for one- to two-story structures should be founded at least 18 inches into medium dense to dense formational soils or properly compacted fill soils and be 15 inches in width. Footing depth should be measured from the lowest adjacent subgrade. Footings close to descending slopes shall be deepened to provide an 8-foot setback from their top.

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.

17. *Bearing Values*: At the recommended depths previously discussed, footings for the residential additions on compacted fill or formational soils may be designed for allowable bearing pressures of 2,500 pounds per square foot (psf) for combined dead and live loads and 3,325 psf for all loads including wind or seismic. An increase in soil allowable static bearing can be used as follows: 1,000 psf for each additional foot in depth and 600 psf for each additional foot in width to a total static bearing capacity not exceeding 5,000 psf.

18. *Footing Reinforcement*: All footings should be reinforced as specified by the structural engineer. However, based on our field investigation findings and laboratory testing, we provide the following minimum recommendations. All continuous footings should contain top and bottom reinforcement to provide structural continuity and to permit spanning of local irregularities. We recommend that, as a minimum, four No. 5 reinforcing bars be provided in the continuous footings (two at the top and two at the bottom). A minimum clearance of 3 inches should be maintained between steel reinforcement and the bottom or sides of the footing. Isolated square footings should contain, as a minimum, a grid of three No. 4 steel bars on 12-inch centers, both ways.

    In order 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.

19. *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 250 pounds per cubic foot (pcf) acting against the foundations may be used in design provided the footings are poured neat against medium dense to dense soils 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.

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

D. Concrete Slab On-Grade Criteria

Slabs on-grade may only be used on new, properly compacted fill or when bearing on medium dense to dense formational soils.

21. *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 sufficient 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.

    Actual floor slab thickness and reinforcement recommendations should be provided by the project Structural Engineer. However, based on our investigation and laboratory data, new interior floor slabs should be at least 5 inches thick and be reinforced with a minimum of No. 4 steel bars spaced no farther than 18 inches apart in both directions. Shrinkage control and isolation joints should be specified by the structural engineer, but the maximum spacing should not exceed 20 feet and at reentrant corners.

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.

22. *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 are providing as Appendix G a discussion regarding minimum protection for slabs. 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. As a minimum moisture barrier, a 15-mil StegoWrap is recommended on 4 inches of crushed rock gravel ½-inch in maximum diameter on compacted subgrade. The project architect should discuss with the owner if a higher degree against soil moisture intrusion is desired in the basement area.

23. *Exterior Slab Thickness and Reinforcement:* Exterior slab reinforcement and control joints should be designed by the project Structural Engineer. As a minimum for protection of on-site improvements, we recommend that all exterior concrete slabs be at least 4 inches thick, reinforced with No. 3 bars at 15-inch centers, both ways at the center of the slab, and contain adequate isolation and control joints. Control joints should be spaced no farther than 10 feet apart and at reentrant corners.

The performance of on-site improvements can be greatly affected by soil base preparation and the quality of construction. It is therefore important that all improvements are properly designed and constructed for the existing soil conditions. The improvements should not be built on loose soils or fills placed without our observation and testing. The subgrade of exterior improvements should be verified as properly prepared within 48 hours prior to concrete placement. A minimum thickness of 3 feet of properly recompacted soils should underlie the exterior slabs on-grade or they should be constructed on dense formational soils.

E. Retaining Wall Recommendations

It is our understanding that a basement is proposed that will require retaining walls 10 to 12 feet in height. The new retaining wall design parameters are presented below. The basement excavation must satisfy OSHA guidelines.

24. *Design Parameters – Unrestrained:* The active earth pressure to be utilized in the design of any cantilever retaining walls utilizing low-expansive [EI less than 50] or imported very low- to low-expansive soils as backfill should be based on an Equivalent Fluid Weight of 38 pcf (for level backfill only). For 2.0:1.0 sloping backfill, the cantilever retaining walls should be designed with an equivalent fluid pressure of 52 pcf. On-site soils ranged from medium to highly expansive. If on-site soils are used as backfill, the retaining wall should be designed for soil pressures above should be increased by a factor of 1.70.

    Unrestrained site retaining walls should be backfilled with properly compacted very low to low-expansive soils. A conversion factor of 0.31 pcf may be used to convert vertical uniform surcharge loads to lateral uniform loads behind an unrestrained retaining wall with level backfill. If on-site soils are used as backfill the surcharge conversion factor should be 0.49 pcf.

25. *Design Parameters – Restrained:* Permanent restrained building retaining walls supporting compacted selected low-expansive on-site or imported level backfill may utilize a triangular pressure increasing at a rate of 56 pcf for wall design (78 pcf for sloping 2.0:1.0 backfill). If medium or highly expansive on-site soils are retained by restrained walls, the design soil pressure shall be 79 pcf. The soil pressure produced by any footings, improvements, or any other surcharge placed within a horizontal distance equal to the height of the retaining portion of the wall should be included in the wall design pressure. For on-site backfill soils, a conversion factor of 0.66 pcf may be used to convert vertical uniform surcharge loads to lateral uniform pressure behind a restrained retaining wall with level backfill (conversion factor of 0.90 pcf if supporting a 2.0:1.0 sloping backfill).

The recommended lateral soil pressures are based on the assumption that no loose soils or unstable soil wedges will be retained by the retaining wall. Backfill soils should consist of low-expansive soils and be placed from the heel of the foundation to the ground surface within the wedge formed by a plane at 30° from vertical, and passing by the heel of the foundation and the back face of the retaining wall when using proper temporary inclined excavations. Where the soils to be retained are supported by shoring, then assume soil pressures for medium to high expansive soils.

26. *Retaining Wall Seismic Design Pressures:* For seismic design of unrestrained walls over 6 feet in exposed height, we recommend that the seismic pressure increment be taken as a fluid pressure distribution utilizing an equivalent fluid weight of 20 pcf. This seismic increment is waived for restrained retaining

walls. If the walls are designed as unrestrained walls, the seismic load should be added to the static soil pressure.

27. *Retaining Wall Drainage:* The preceding design pressures assume that the walls are backfilled with properly compacted, imported low expansion potential materials (Expansion Index less than 50) or on-site stiff expansive soils behind shoring walls and that there is sufficient drainage behind the walls to prevent the build-up of hydrostatic pressures from surface water infiltration. We recommend that drainage be provided by a composite drainage material such as J-Drain 200/220 and J-Drain SWD, or equivalent. No perforated pipes or gravel are required with the J-Drain system.

The drain material at the top should terminate 12 inches below the exterior finish surface where the surface is covered by slabs or 18 inches below the finish surface in landscape areas. Waterproofing should extend from the bottom to the top of the wall. Refer to Figure No. IX, Retaining Wall Drainage Schematic. Basement walls should be provided with a sump area to collect water to be pumped out.

Geotechnical Exploration, Inc. will assume no liability for damage to structures or improvements that is attributable to poor drainage. The architectural plans should clearly indicate that subdrains for any below grade walls be placed at an elevation at least 1 foot ***below*** the bottom of the lower-level slabs. A water stop should be used at the joint between the basement wall and the foundation.

F. Swimming Pool Recommendations

27. Swimming pool geotechnical criteria will be provided upon receipt of detailed pool conceptual design information.

G. Pavement

28. *Concrete Pavement:* We recommend that new driveways subject only to automobile and light truck traffic be 5.5 inches thick and be supported directly on properly prepared/compacted on-site subgrade soils. The upper 6 inches of the low-expansive subgrade below the slab should be compacted to a minimum degree of compaction of 95 percent just prior to paving. The concrete should conform to Section 201 of The Standard Specifications for Public Works Construction, 2021 Edition, for Class 560-C-3250.

    In order to control shrinkage cracking, we recommend that saw-cut, weakened-plane joints be provided at about 12-foot centers both ways and at reentrant corners. The pavement slabs should be saw-cut as soon as practical but no more than 24 hours after the placement of the concrete. The depth of the shrinkage control joint should be one-quarter of the slab thickness and its width should not exceed 0.02-foot. Reinforcing steel is not necessary unless it is desired to increase the joint spacing recommended above.

29. *Interlocking Permeable Pavers*: If desired, we recommend that permeable pavement pavers for the driveway (subject only to automobile and light truck traffic) or rear yard (with only foot traffic) be supported on a 1.5-inch-thick section of bedding No. 8 sand on 8 inches of crushed miscellaneous base conforming to Section 200-2 of the Standard Specifications for Public Works Construction (2021 Edition) or 8 inches of No. 57 crushed rock gravel per ASTM

D448 gradation. The upper 6 inches of the pavement low-expansive subgrade soil, as well as the aggregate base layer, should be compacted to a minimum degree of compaction of 95 percent. Preparation of the subgrade and placement of the base materials should be performed under the observation of our representative. If the area of pavers will be used as an infiltration or filtration basin, the bottom gravel layer may be thicker, as specified by the project civil engineer.

H. Site Drainage Considerations

30. *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.

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

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

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

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

I. General Recommendations

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

observed problem (i.e., deepening the footing excavation, recompacting soil in the bottom of the excavation, etc.).

35. *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.

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

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

XVI. <u>GRADING NOTES</u>

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

XVII. <u>LIMITATIONS</u>

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

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

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

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

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. 01-8018** will expedite a reply to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

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

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

Cathy K. Ganze, Project Coordinator
Senior Project Geologist

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JOB NO. 01-8018
July 2024

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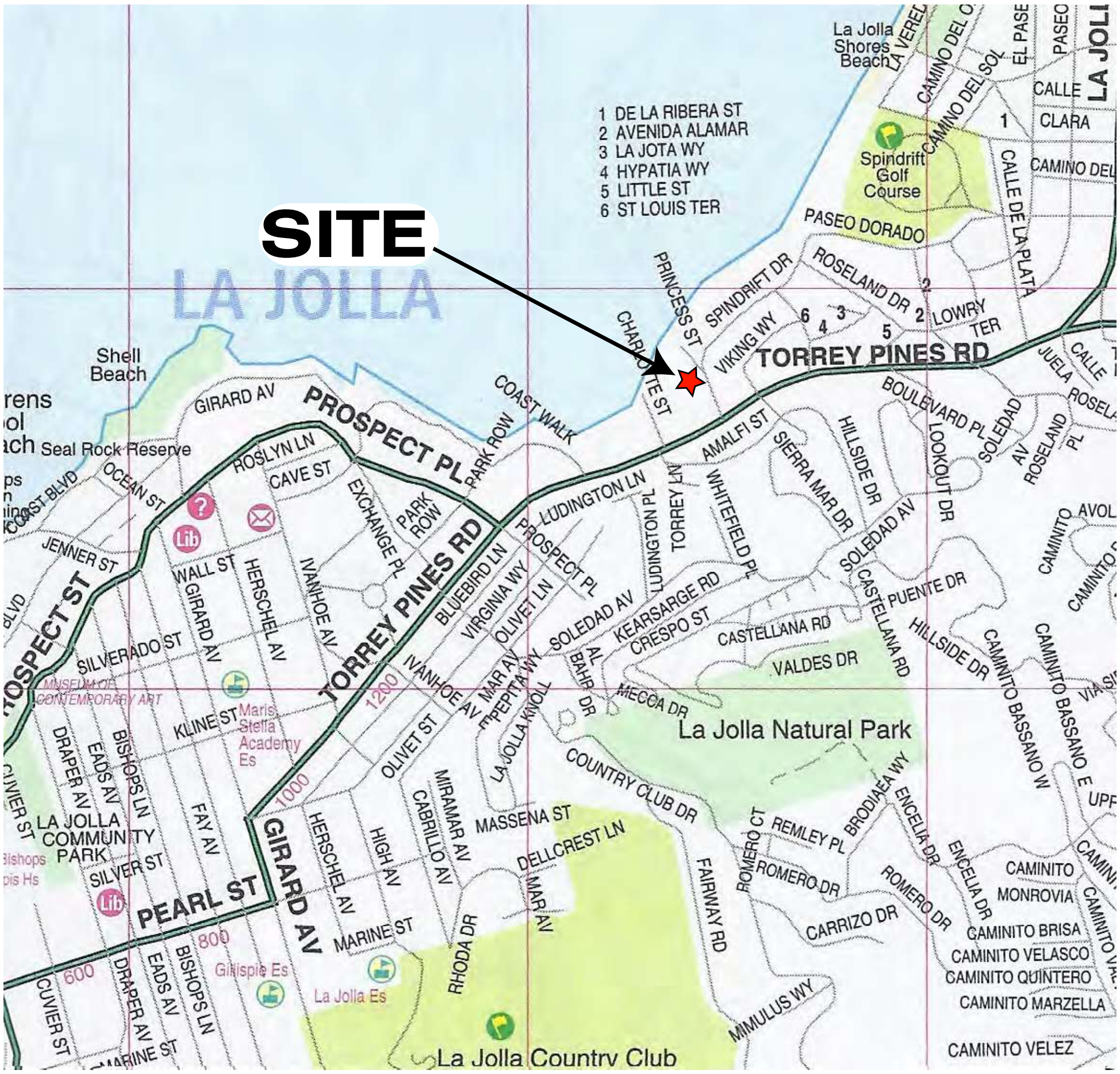
VICINITY MAP



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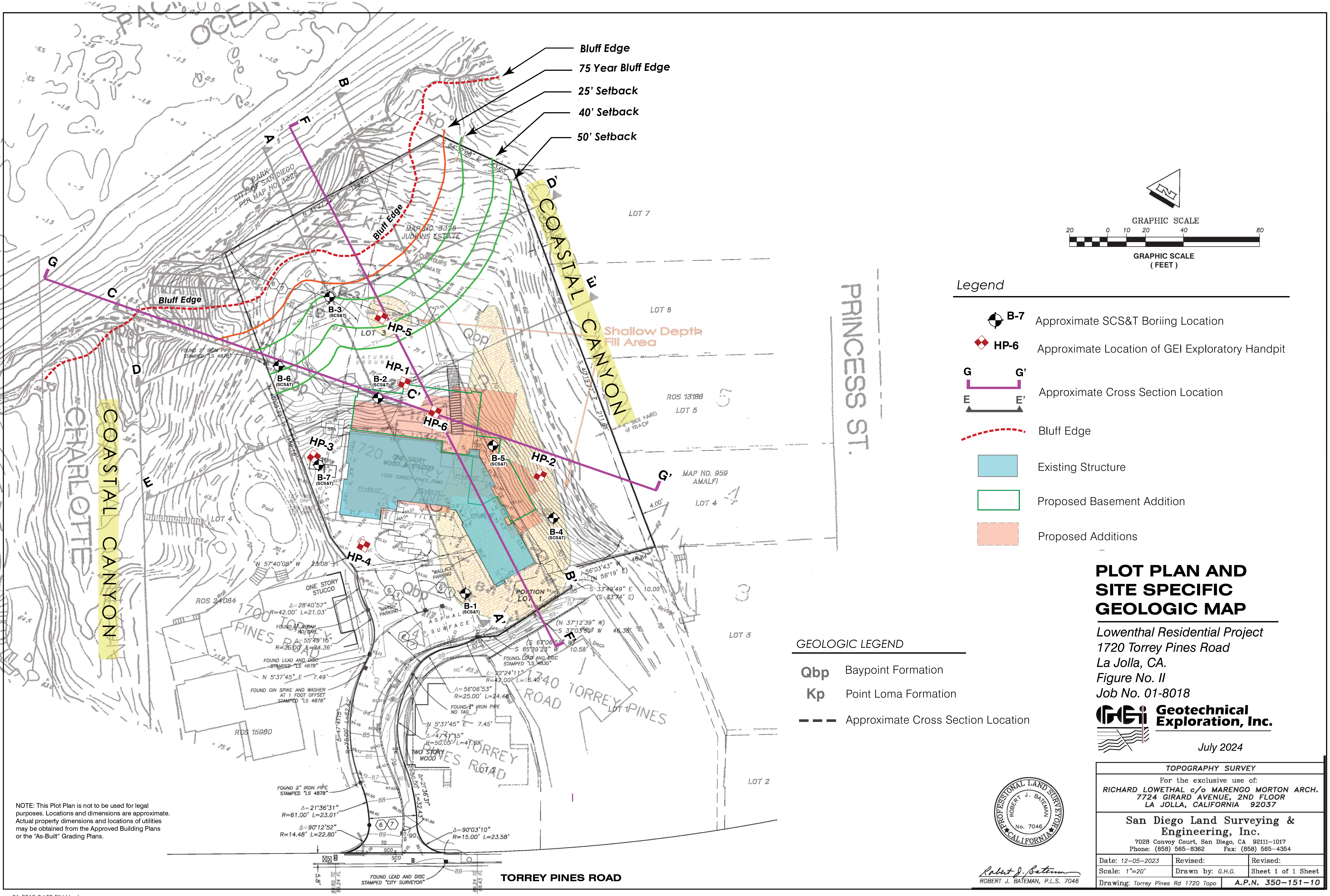
Lowenthal Residential Project
1720 Torrey Pines Road
La Jolla, CA.

Figure No. I
Job No. 01-8018

Geotechnical Exploration, Inc.



Bluff Edge
75 Year Bluff Edge
25' Setback
40' Setback
50' Setback
COASTAL CANYON
COASTAL CANYON
PRINCESS ST.
CHARLOTTE
TORREY PINES ROAD
Shallow Depth Fill Area
B-1
B-2
B-3
B-4
B-5
B-6
B-7
HP-1
HP-2
HP-3
HP-4
HP-5
HP-6
Qbp
Kp
GRAPHIC SCALE
GRAPHIC SCALE (FEET)
Legend
B-7 Approximate SCS&T Boriing Location
HP-6 Approximate Location of GEI Exploratory Handpit
G G' E E' Approximate Cross Section Location
Bluff Edge
Existing Structure
Proposed Basement Addition
Proposed Additions
GEOLOGIC LEGEND
Qbp Baypoint Formation
Kp Point Loma Formation
Approximate Cross Section Location
PLOT PLAN AND SITE SPECIFIC GEOLOGIC MAP
Lowenthal Residential Project
1720 Torrey Pines Road
La Jolla, CA.
Figure No. II
Job No. 01-8018
Geotechnical Exploration, Inc.
July 2024
TOPOGRAPHY SURVEY
For the exclusive use of:
RICHARD LOWETHAL c/o MARENGO MORTON ARCH.
7724 GIRARD AVENUE, 2ND FLOOR
LA JOLLA, CALIFORNIA 92037
San Diego Land Surveying & Engineering, Inc.
7028 Convoy Court, San Diego, CA 92111-1017
Phone: (858) 565-8362 Fax: (858) 565-4354
Date: 12-05-2023
Revised:
Revised:
Scale: 1"=20'
Drawn by: G.H.G.
Sheet 1 of 1 Sheet
Drawing: Torrey Pines Rd 1720 Topo
A.P.N. 350-151-10
ROBERT J. BATEMAN, P.L.S. 7046
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.
01-8018-BASE-FINAL.ai

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Hand Tools	5' x 4' x 3' Pit	11-16-99
SURFACE ELEVATION	GROUNDWATER DEPTH	LOGGED BY
Not Available	Not encountered	DV

DEPTH FT.	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
			SANDY CLAY. Soft, moist, gray-brown with roots. LANDSCAPE TOPSOIL	CL								
1			SLIGHTLY SANDY CLAY. Stiff to very stiff, moist, gray brown.	CL								
		2			12.1	100.8			83			
2			COLLUVIUM/Qbp									
							11.0	121.5		93*		
3												
			CLAYEY SAND, Medium dense, moist, gray with caliche.									
4		1			11.4	117.2	9.6	128.7	91	32*		
				SC								
			FORMATION (Qbp)									
			- becomes sandier.									
5			Bottom of Excavation @ 5'									
			East Wall of Pit.									
6			* Expansion Index									

WATER TABLE

LOOSE BAG SAMPLE

IN-PLACE SAMPLE

DRIVE SAMPLE

SAND CONE/F.D.T.

STANDARD PENETRATION TEST

JOB NAME		
Lowenthal Residential Property		
SITE LOCATION: 1720 Torrey Pines Road, La Jolla, Ca.		
JOB NUMBER: 01-8018	REVIEWED BY: GEOTECHNICAL EXPLORATION INC.	LOG No. HP-1
FIGURE NUMBER: IIIa		

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Hand Tools	2' x 2.5' x 4' Pit	11-16-99
SURFACE ELEVATION	**GROUNDWATER DEPTH**	**LOGGED BY**
Not Available	Not encountered	DV

DEPTH FT.	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION — DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–2			SANDY CLAY. Stiff-very stiff, damp, dark gray/brown.	CL								
		1	LANDSCAPE TOPSOIL		6.2	89.3	9.8	124.0				
2–3.5		☒	SILTY FINE TO COARSE SAND. Medium dense, damp, gold brown/orange brown.	SM								
			COLLVIUM									
3.5–4			SANDY SILT. Stiff, damp, orange brown.	ML								
			FORMATION (Qbp)									
4			Bottom of Excavation @ 4'									
			South Wall of Pit									

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

Legend	
▽	WATER TABLE
☒	LOOSE BAG SAMPLE
1	IN-PLACE SAMPLE
■	DRIVE SAMPLE
S	SAND CONE/F.D.T.
▨	STANDARD PENETRATION TEST

JOB NAME		
Lowenthal Residential Project		
SITE LOCATION		
1720 Torrey Pines Road, La Jolla, Ca.		
JOB NUMBER	**REVIEWED BY**	**LOG No.**
01-8018	GEOTECHNICAL EXPLORATION INC.	**HP-2**
FIGURE NUMBER		
IIIb		

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Hand Tools	2' x 2.5' x 3.5' Pit	11-16-99
SURFACE ELEVATION	GROUNDWATER DEPTH	LOGGED BY
Not Available	Not encountered	DV

DEPTH FT.	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION — DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–1.7			SILTY SAND. Loose, dry-damp, light brown. FILL	SM								
1.7–3.5		Loose bag; 1	SANDY CLAY. Firm to stiff, damp, brown/ gray brown, grades into clay. COLLUVIUM/Qbp	CL	13.8	99.5	10.6	121.8		56*		
			Bottom of Excavation @ 3'-6"									
			* Expansion Index									

WATER TABLE

LOOSE BAG SAMPLE

1 IN-PLACE SAMPLE

DRIVE SAMPLE

S SAND CONE/F.D.T.

STANDARD PENETRATION TEST

JOB NAME		
Lowenthal Residential Project		
SITE LOCATION: 1720 Torrey Pines Road, La Jolla, Ca.		
JOB NUMBER: 01-8018	REVIEWED BY: GEOTECHNICAL EXPLORATION INC.	LOG No.
FIGURE NUMBER: IIIc		**HP-3**

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Hand Tools	1.5' x 2' x 2.5' Pit	11-16-99
SURFACE ELEVATION	GROUNDWATER DEPTH	LOGGED BY
Not Available	Not encountered	DV

DEPTH FT.	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. − (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
			3-4" SILTY SAND. Loose, moist-wet dark brown. LANDSCAPE TOPSOIL	SM								
1			CLAYEY SANDY SILT. Stiff to very stiff, damp, moist, gold brown/orange brown.	CL								
2		⊠	COLLUVIUM/Qbp				10.1	121.8		46 *		
			Bottom of Excavation @ 2.5'									
3			* Expansion Index									
4												
5												
6												

WATER TABLE
LOOSE BAG SAMPLE
IN-PLACE SAMPLE
DRIVE SAMPLE
SAND CONE/F.D.T.
STANDARD PENETRATION TEST

JOB NAME		
Lowenthal Residential Project		
SITE LOCATION: 1720 Torrey Pines Road, La Jolla, Ca.		
JOB NUMBER: 01-8018	REVIEWED BY: GEOTECHNICAL EXPLORATION INC.	LOG No. HP-4
FIGURE NUMBER: IIId		

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Hand Tools	1' x 2' x 1'7" Pit	11-16-99
SURFACE ELEVATION	GROUNDWATER DEPTH	LOGGED BY
Not Available	Not encountered	DV

DEPTH FT.	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. − (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–1'7"			SANDY SILT/SILTY SAND. Loose-moist-wet dark brown, micaceous. over 3-4" grass lawn **LANDSCAPE TOPSOIL** Bottom of Excavation @ 1'-7"	SM								
1'7"–2			cement pond liner (unknown thickness and reinforcement)									

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

Legend	
▽	WATER TABLE
☒	LOOSE BAG SAMPLE
[1]	IN-PLACE SAMPLE
■	DRIVE SAMPLE
[S]	SAND CONE/F.D.T.
▨	STANDARD PENETRATION TEST

JOB NAME		
Lowenthal Residential Project		
SITE LOCATION		
1720 Torrey Pines Road, La Jolla, Ca.		
JOB NUMBER: 01-8018	REVIEWED BY: GEOTECHNICAL EXPLORATION INC.	LOG No.
FIGURE NUMBER: IIIe		**HP-5**

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Hand Tools	2' x 2' x 1'8" Pit	11-16-99
SURFACE ELEVATION	GROUNDWATER DEPTH	LOGGED BY
Not Available	Not encountered	DV

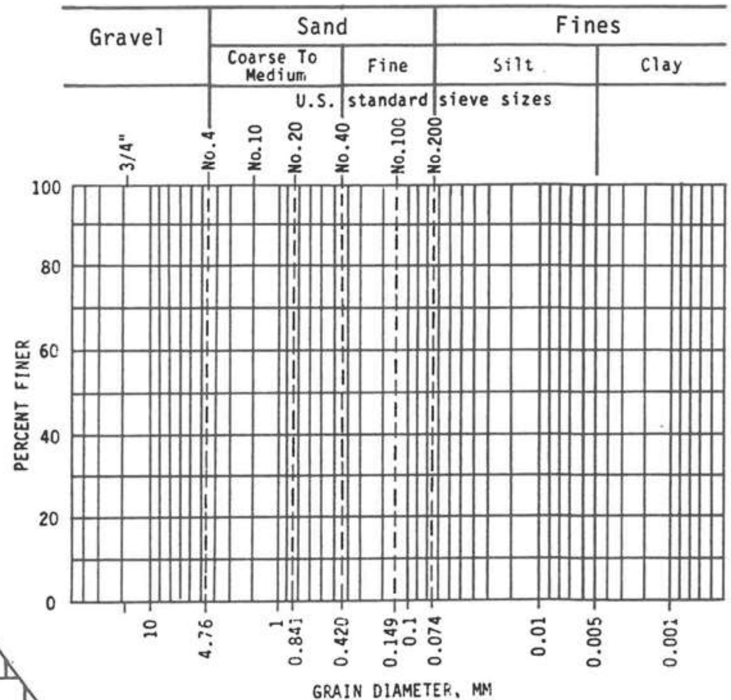
DEPTH FT.	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION: DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
0–1.5			SILTY SAND. Loose, dry-damp, light brown. **FILL**	SM								
1.5–1.67			SANDY CLAY. Stiff-very stiff, damp, brown/gray brown. **COLLUVIUM/Qbp**	CL								
			Bottom of Excavation @ 20"									

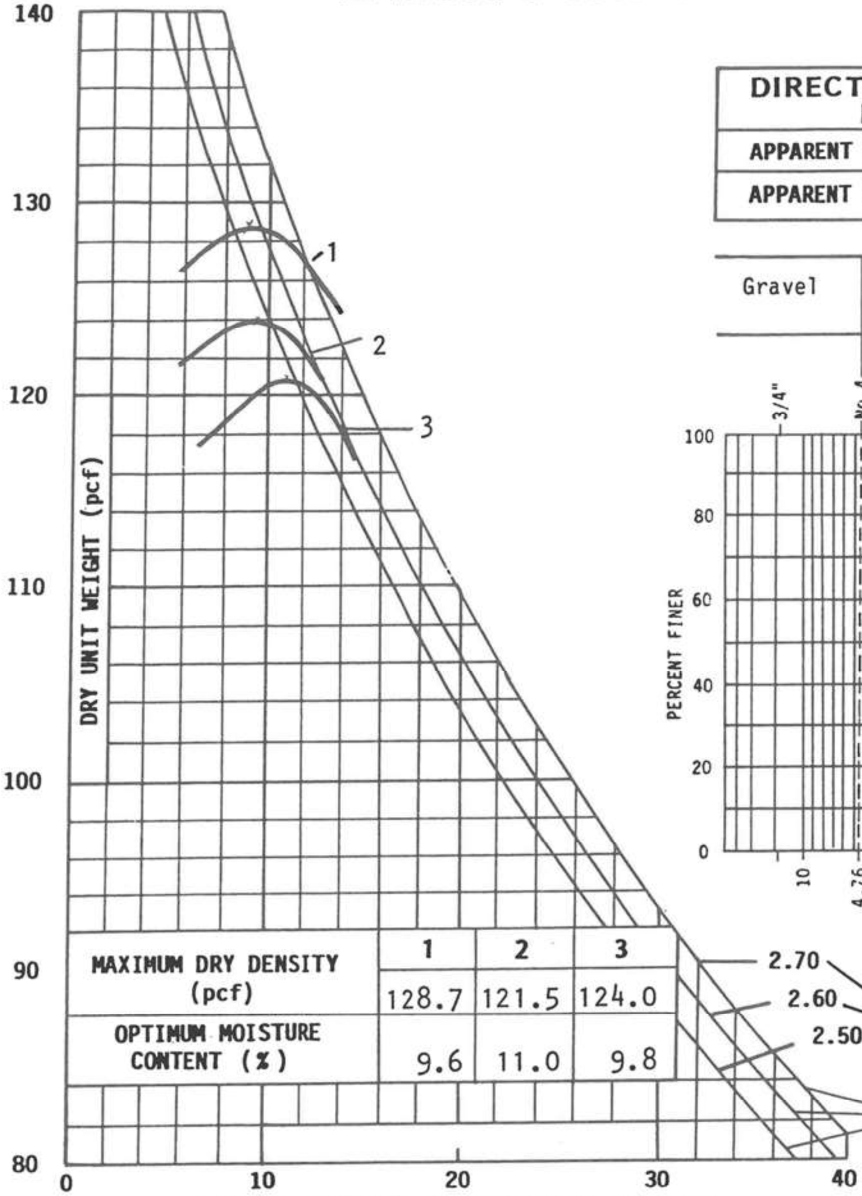
Depth scale: 1, 2, 3, 4, 5, 6

LEGEND			
WATER TABLE	JOB NAME: Lowenthal Residential Project		
LOOSE BAG SAMPLE	SITE LOCATION: 1720 Torrey Pines Road, La Jolla, Ca.		
IN-PLACE SAMPLE	JOB NUMBER: 01-8018	REVIEWED BY	LOG No.
DRIVE SAMPLE		GEOTECHNICAL EXPLORATION INC.	**HP-6**
SAND CONE/F.D.T.	FIGURE NUMBER: IIIf		
STANDARD PENETRATION TEST			

LABORATORY SOIL DATA SUMMARY

DIRECT SHEAR TEST DATA	1	2	3
APPARENT COHESION (psf)			
APPARENT FRICTION ANGLE			





	1	2	3
MAXIMUM DRY DENSITY (pcf)	128.7	121.5	124.0
OPTIMUM MOISTURE CONTENT (%)	9.6	11.0	9.8

LABORATORY COMPACTION TEST

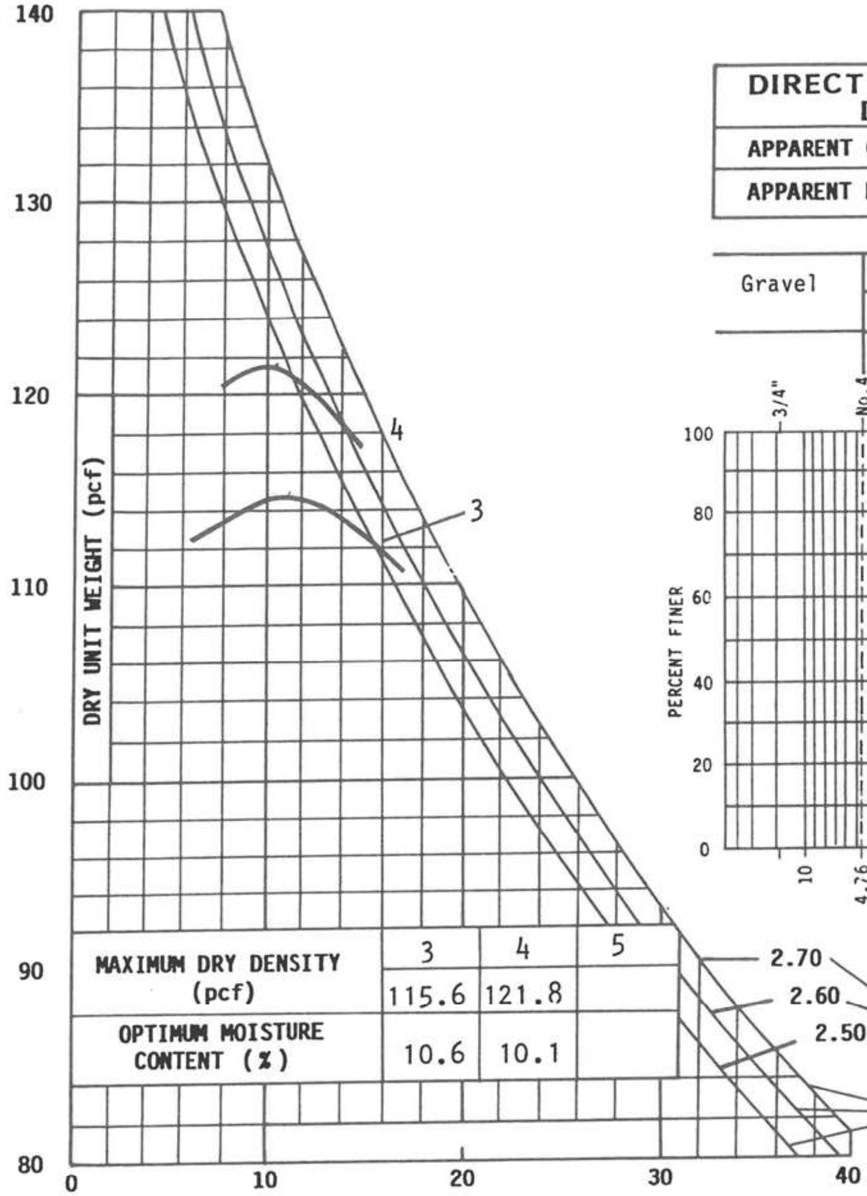
SOIL TYPE	SOIL CLASSIFICATION	BORING No.	TRENCH No.	DEPTH
1	Gray clayey sand.		HP-1	3'
2	Gray brown silty sandy clay		HP-1	2'
3	Gray brown sandy clay.		HP-2	2'

SWELL TEST DATA	1	2	3
INITIAL DRY DENSITY (pcf)			
INITIAL WATER CONTENT (%)			
LOAD (psf)			
PERCENT SWELL			

Figure No. IVa
Job No. 01-8018

Geotechnical Exploration, Inc.

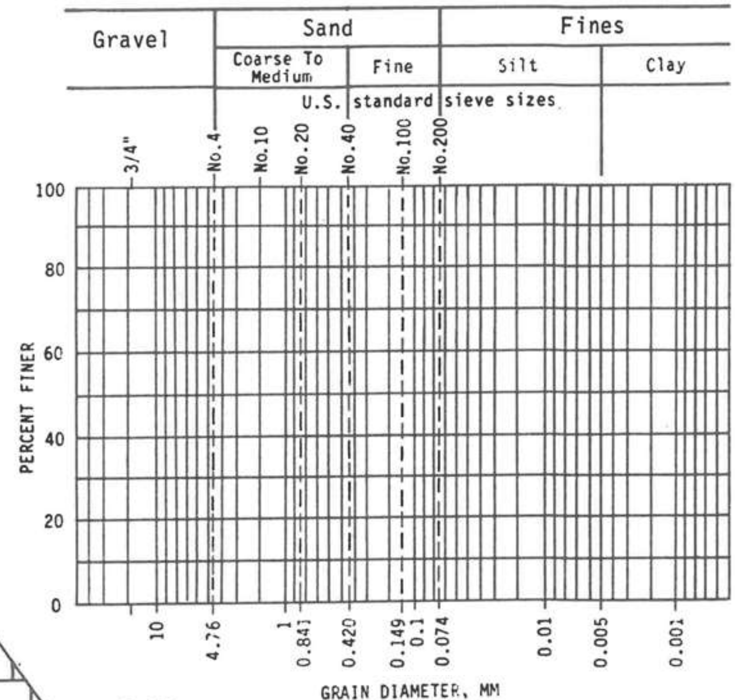
LABORATORY SOIL DATA SUMMARY



MAXIMUM DRY DENSITY (pcf)	3	4	5
	115.6	121.8	
OPTIMUM MOISTURE CONTENT (%)	10.6	10.1	

LABORATORY COMPACTION TEST

DIRECT SHEAR TEST DATA	3	4	5
APPARENT COHESION (psf)			
APPARENT FRICTION ANGLE			



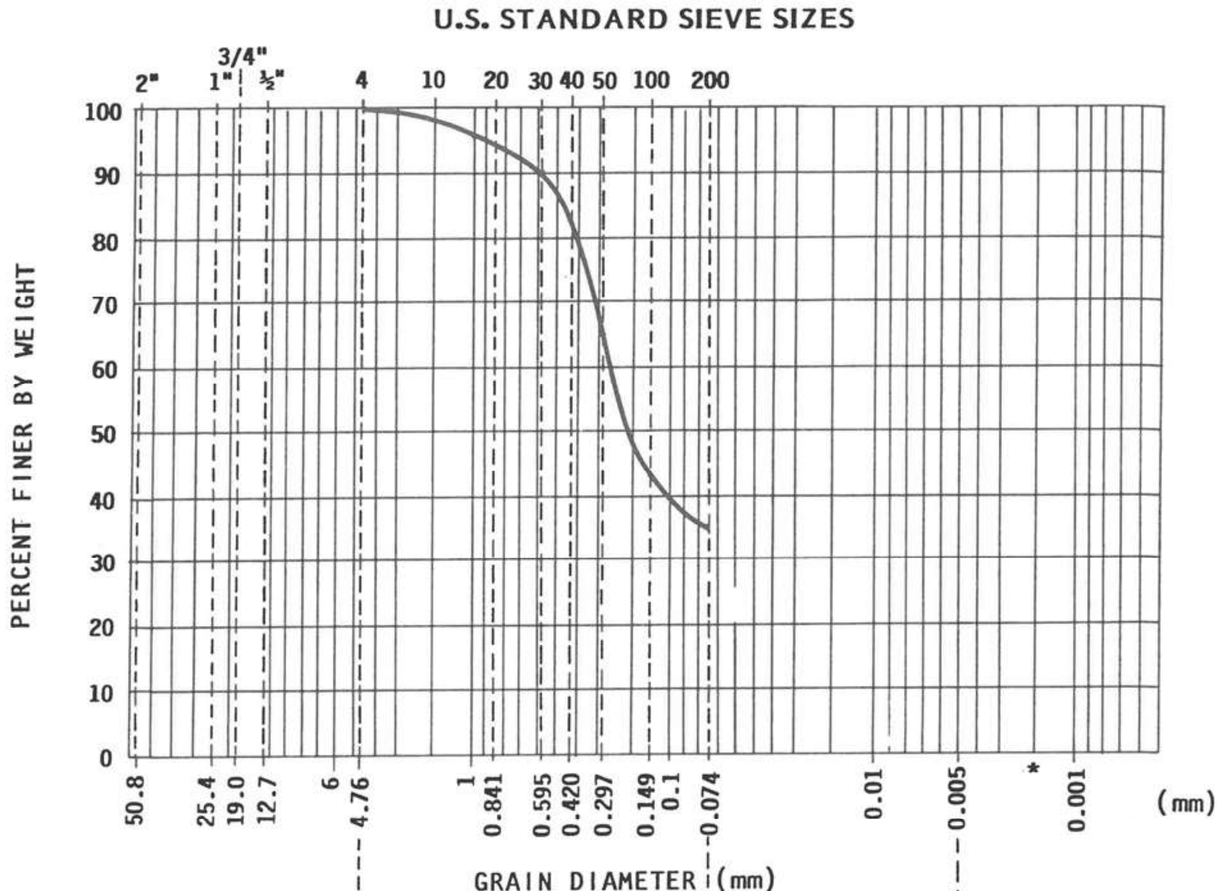
SOIL TYPE	SOIL CLASSIFICATION	BORING No.	TRENCH No.	DEPTH
3	Gray brown sandy clay		HP-3	2'
4	Gold brown/orange brown clayey sandy silt.		HP-4	2'

SWELL TEST DATA			
INITIAL DRY DENSITY (pcf)			
INITIAL WATER CONTENT (%)			
LOAD (psf)			
PERCENT SWELL			

Figure No. IVb
Job No. 01-8018

Geotechnical Exploration, Inc.

PARTICLE-SIZE ANALYSIS OF SOILS



GRAVEL	SAND			FINES	
	COARSE	MEDIUM	FINE	SILT	CLAY

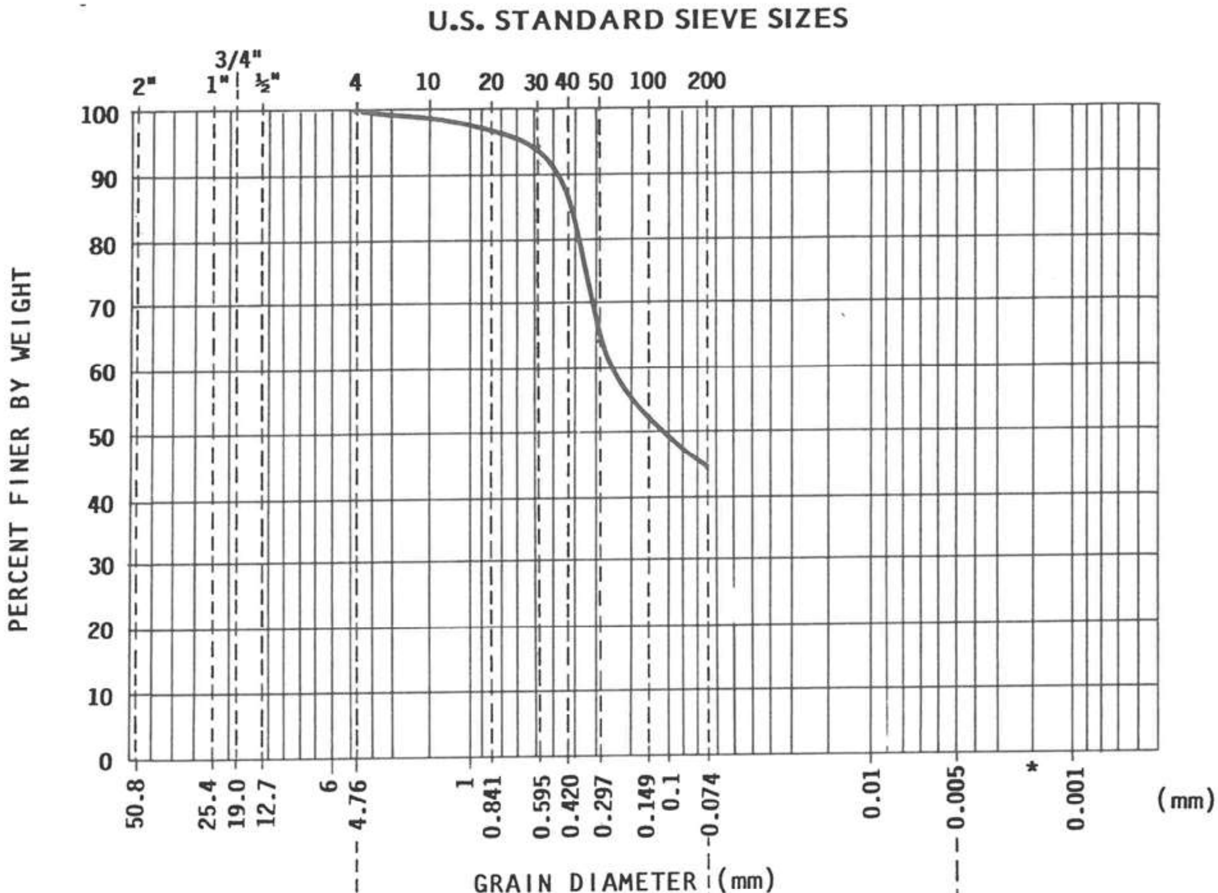
* **NOTE:** FOR THE UNIFIED SOIL CLASSIFICATION SYSTEM CLAY MATERIALS ARE 0.002 (mm) AND FINER.

BORING NO.	SAMPLE DEPTH	DESCRIPTION OF SOIL
HP-1	3'	Gray clayey sand

Figure No. IVc
Job No. 01-8018

Geotechnical Exploration, Inc.

PARTICLE-SIZE ANALYSIS OF SOILS



GRAVEL	SAND			FINES	
	COARSE	MEDIUM	FINE	SILT	CLAY

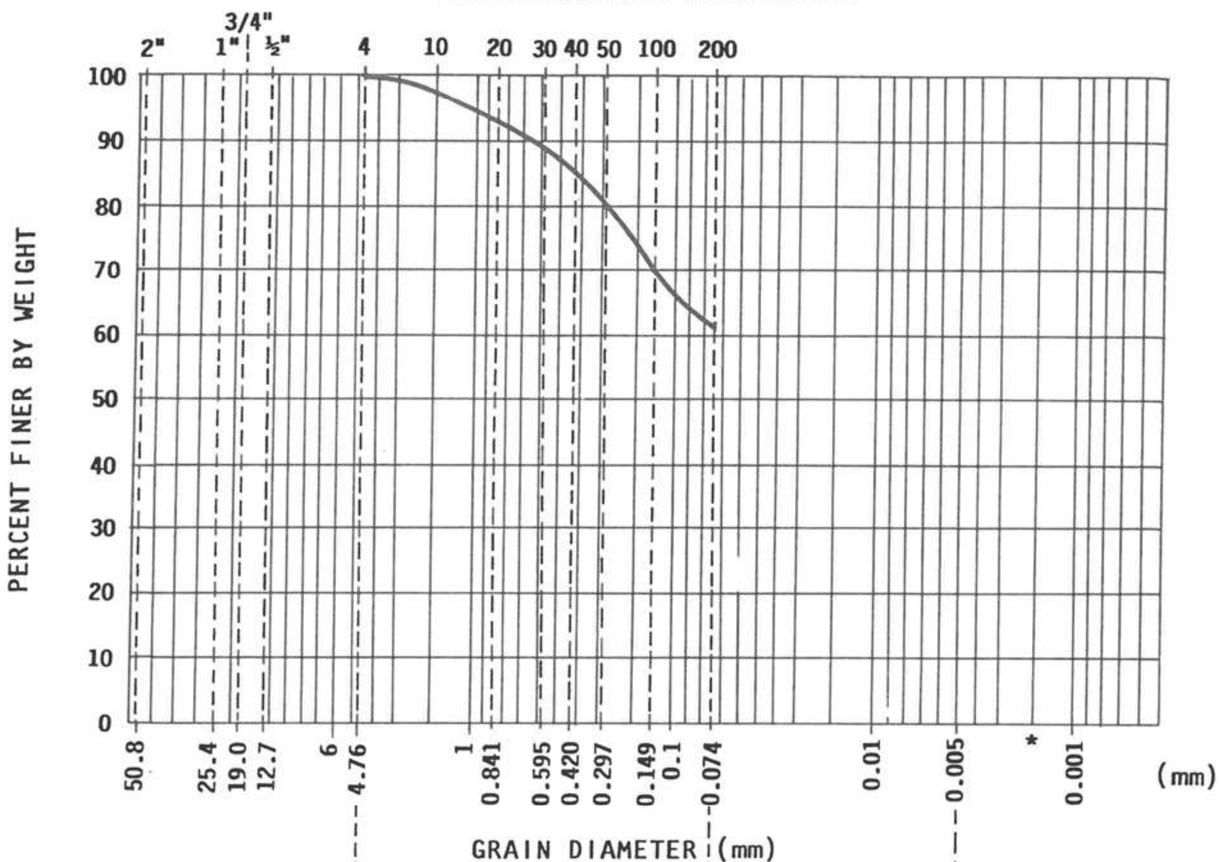
* **NOTE:** FOR THE UNIFIED SOIL CLASSIFICATION SYSTEM CLAY MATERIALS ARE 0.002 (mm) AND FINER.

BORING NO.	SAMPLE DEPTH	DESCRIPTION OF SOIL
HP-2	2'	Gold brown/orange brown silty sand.

Figure No. IVd
Job No. 01-8018

Geotechnical Exploration, Inc.

PARTICLE-SIZE ANALYSIS OF SOILS



GRAVEL	SAND			FINES	
	COARSE	MEDIUM	FINE	SILT	CLAY

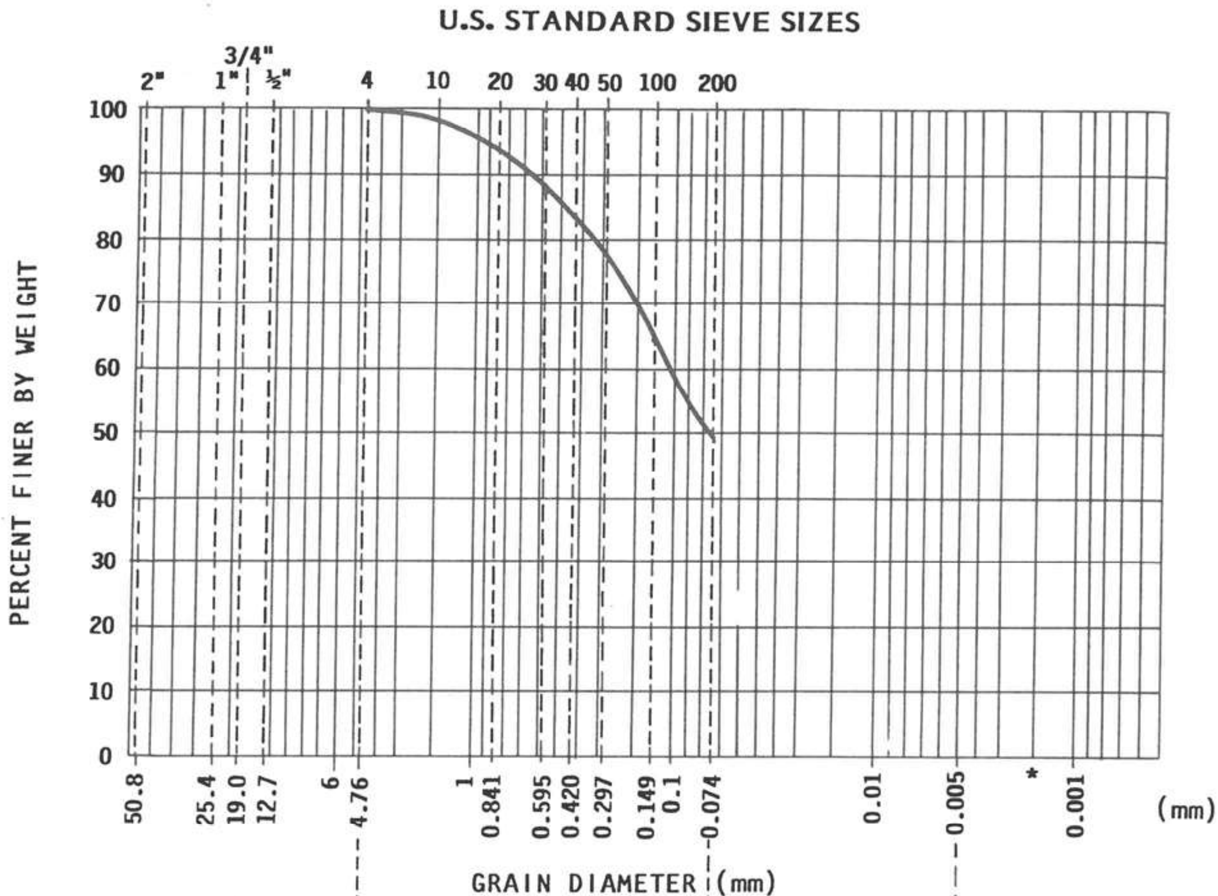
* **NOTE:** FOR THE UNIFIED SOIL CLASSIFICATION SYSTEM CLAY MATERIALS ARE 0.002 (mm) AND FINER.

BORING NO.	SAMPLE DEPTH	DESCRIPTION OF SOIL
HP-3	2'	Brown sandy clay.

Figure No. IVe
Job No. 01-8018

Geotechnical Exploration, Inc.

PARTICLE-SIZE ANALYSIS OF SOILS



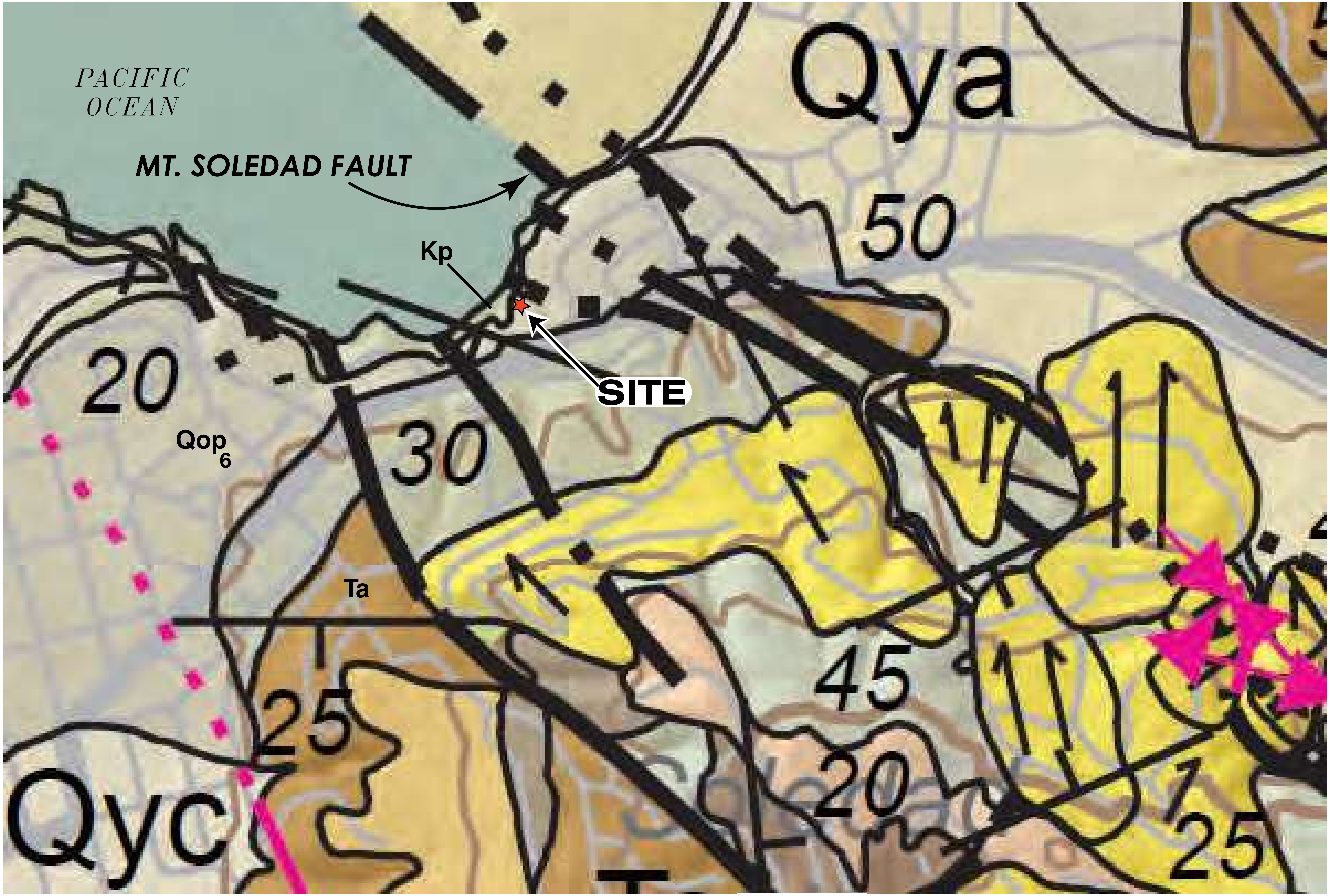
GRAVEL	SAND			FINES	
	COARSE	MEDIUM	FINE	SILT	CLAY

* **NOTE:** FOR THE UNIFIED SOIL CLASSIFICATION SYSTEM CLAY MATERIALS ARE 0.002 (mm) AND FINER.

BORING NO.	SAMPLE DEPTH	DESCRIPTION OF SOIL
HP-4	2'	Gold brown clayeysandy silt.

Figure No. IVf
Job No. 01-8018

Geotechnical Exploration, Inc.



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

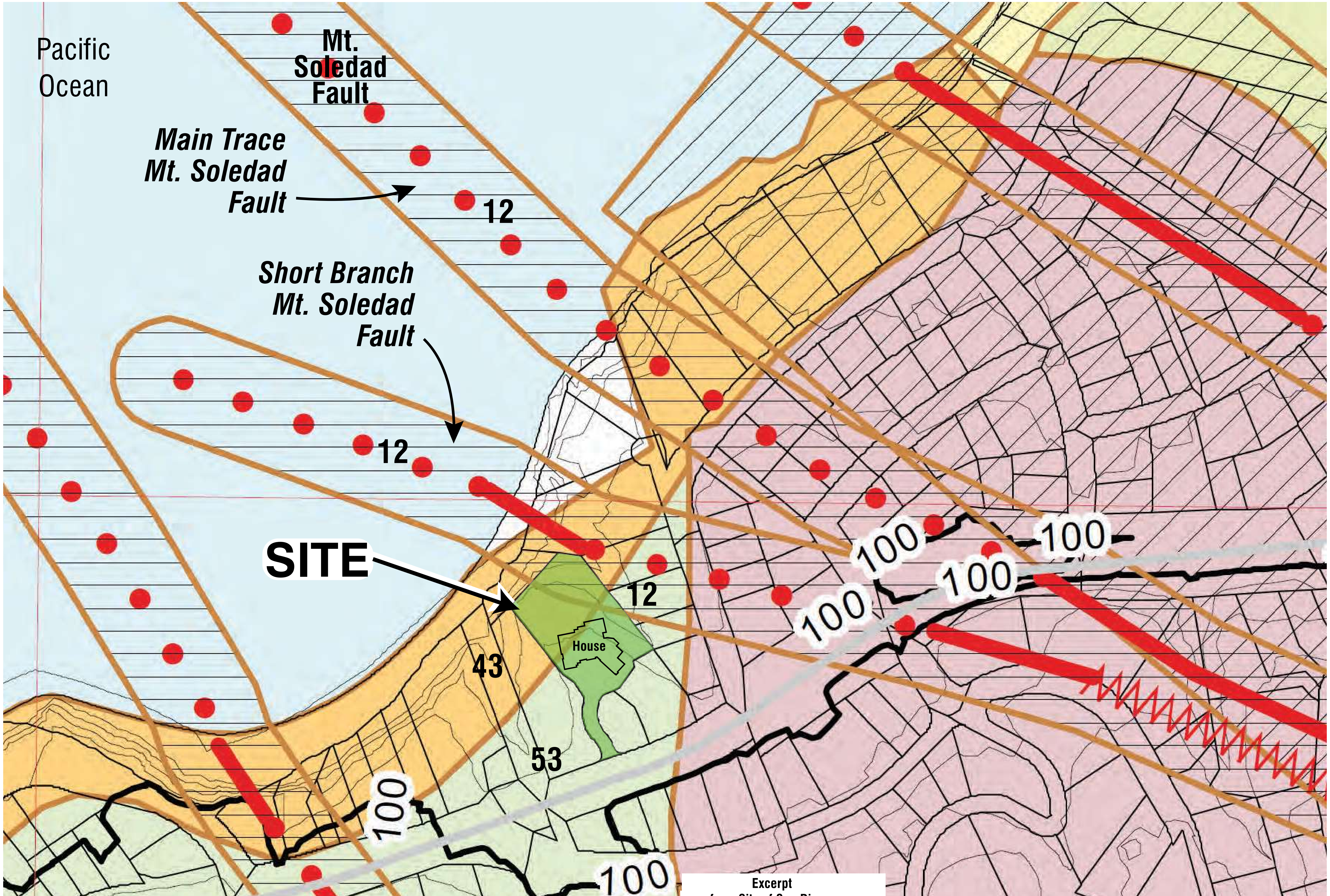
Kelly R. Bovard[1], Anne G. Garcia[1], Diane Burns[1], and Carlos I. Gutierrez[1]

Lowenthal Residential Project
1720 Torrey Pines Road
La Jolla, CA.
Figure No. V
Job No. 01-8018

Geotechnical Exploration, Inc.

July 2024

01-8018-V.ai



Excerpt
from City of San Diego
Seismic Safety Study
Geologic Hazards and Fault Map
Sheet 29
Development Services Department
DATE: 4/3/2008

Lowenthal Residential Project
1720 Torrey Pines Road
La Jolla, CA.
Figure No. VI
Job No. 01-8018

Geotechnical Exploration, Inc.

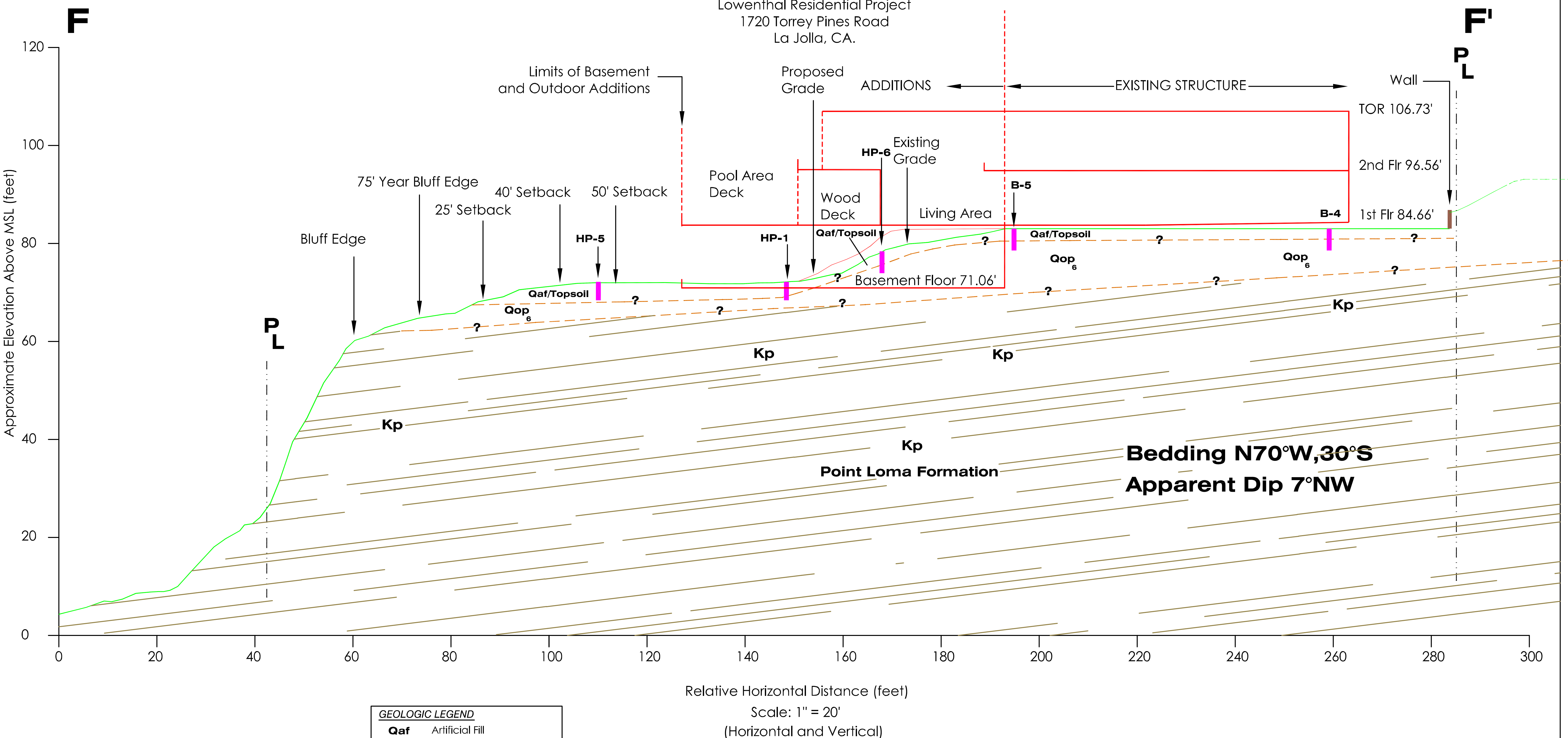
July 2024

01-8018-VI.ai

GEOLOGIC CROSS SECTION F-F'

N58°W

Lowenthal Residential Project
1720 Torrey Pines Road
La Jolla, CA.



GEOLOGIC LEGEND

Qaf	Artificial Fill
Qop_6	Old Paralic Deposits Unit 6
Kp	Point Loma Formation
? – ?	Approximate Geologic Contact

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

Figure No. VIIa
Job No. 01-8018

Geotechnical Exploration, Inc.

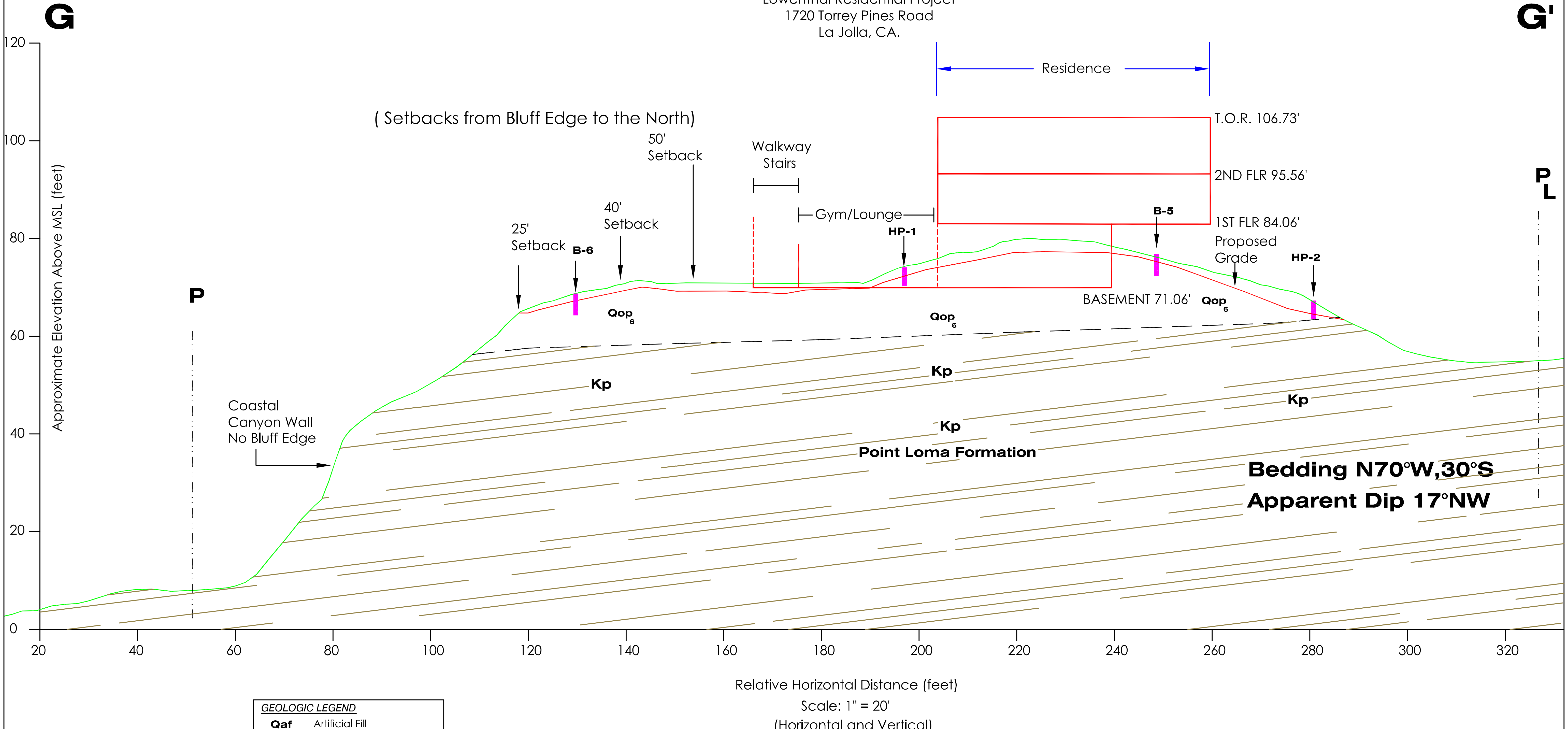
July 2024

01-8018-FF.dwg

GEOLOGIC CROSS SECTION G-G'

N78°E

Lowenthal Residential Project
1720 Torrey Pines Road
La Jolla, CA.



GEOLOGIC LEGEND

Qaf	Artificial Fill
Qop6	Old Paralic Deposits Unit 6
Kp	Point Loma Formation
-?- -?-	Approximate Geologic Contact

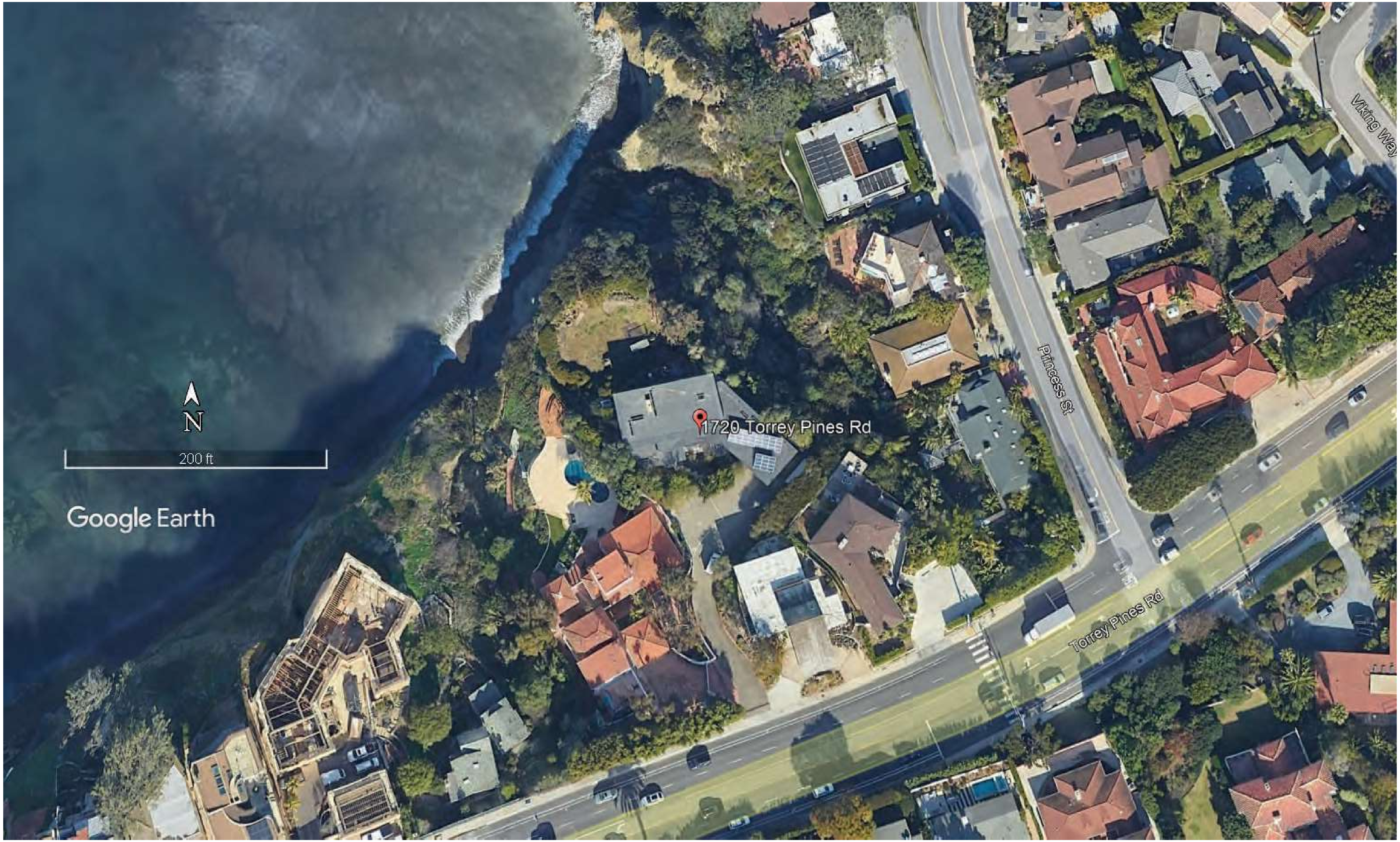
NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

01-8018-GG.dwg

Figure No. VIIb
Job No. 01-8018

Geotechnical Exploration, Inc.

July 2024



Reference: This photo was prepared
from an existing GOOGLE EARTH
image dated 11-16-2022

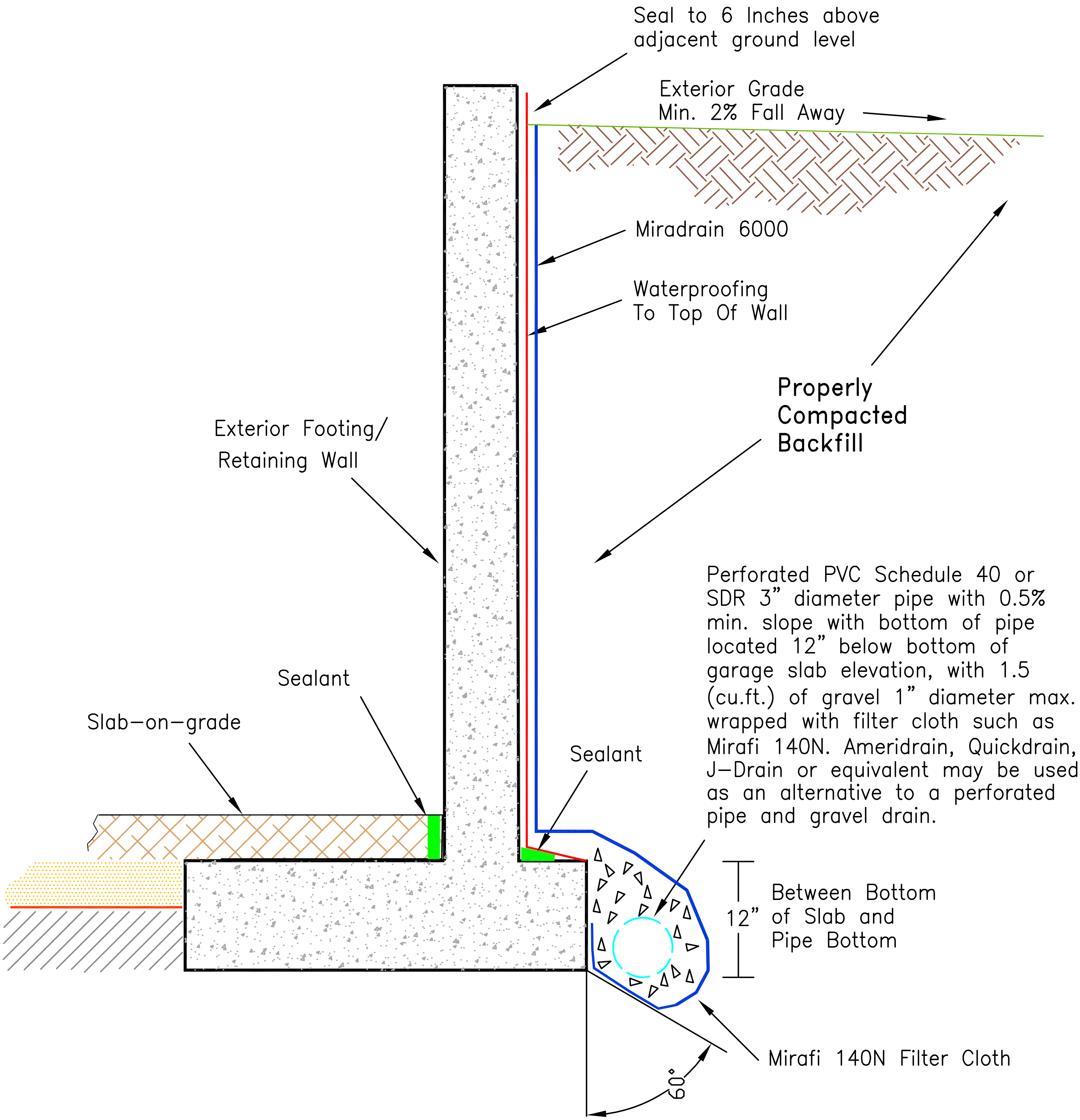
Lowenthal Residential Project
1720 Torrey Pines Road
La Jolla, CA.
Figure No. VIII
Job No. 01-8018

Geotechnical Exploration, Inc.

July 2024

01-8018-VIII.ai

RECOMMENDED RETAINING WALL DRAINAGE SCHEMATIC



NOT TO SCALE

Figure No. IX
Job No. 01-8018

Geotechnical Exploration, Inc.

July 2024

01-8018--IX.dwg

APPENDIX A

SCS&T Boring Logs B-1, B-2 and B-3, from SCS&T report dated 9/24/1999, and SCS&T Boring Logs B-4, B-5, B-6 and B-7, dated 11/15/1999, provided in GEI report dated 2/9/2000

LOG OF TEST BORING NUMBER B-1

Date Excavated: 9/10/99
Equipment: BUCKET RIG
Surface Elevation (ft): 83.5 +/-

Logged by: MF
Project Manager: DBA
Depth to Water (ft): 28.5

DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: UNDISTURBED	SAMPLES: BULK	PENETRATION (blows/ 6" of drive)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
0–2	SM/ SC	4" of Asphalt Concrete FILL (Qaf) - Brown, Very Moist, Medium Dense to Dense, CLAYEY SILTY SAND (Very Fine)	CK					
4	ML/ CL	FILL/PROCESSED TOPSOIL - Brown, Very Moist, Stiff, CLAYEY SILT/ SILTY CLAY						
6	SM	BAYPOINT FORMATION (Qbp) - Brown to Tan to Light Orange, Moist to Very Moist, Medium Dense, SILTY SAND (Fine-Medium), Occasional Silt/Clay Lenses	US		1/2			
8	SM/ SC	BAYPOINT FORMATION (Qbp) - Gray to Tan to Rust, Moist, Medium, Dense, CLAYEY SILTY SAND (Fine-Medium), Numerous Silt/Clay Lenses						
10–20	ML/ CL	Contact Subhorizontal, Irregular ARDATH SHALE (Ta) - Gray to Brown to Rust, Very Moist, Hard, SILTY CLAY/ CLAYEY SILT (Mudstone), Highly Fractured , Bedding Generally Indistinct @ 12.5' : Becomes Moderately to Highly Fractured @ 19.0' : Bedding Plane with Sheared Silt/Clay Dips 30 degrees/ 240 degrees	US		2/4			

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

DAMMEYER RESIDENCE

BY: DBA/MF/TSW

DATE: 9/23/99

JOB NUMBER: 9911152.1

PLATE NO.: 4

LOG OF TEST BORING NUMBER B-1

Date Excavated: 9/10/99
Equipment: Bucket Rig
Surface Elevation (ft): 83.5 +/-

Logged by: MF
Project Manager: DBA
Depth to Water (ft): 28.5

DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES UNDISTURBED	SAMPLES BULK	PENETRATION (blows/ 6" of drive)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		ARDATH SHALE (Ta) - No Change						
			CK					
22	ML / CL							
24								
			US		3/5			
26								
		Down-Hole Logging Ended @ 26 ft. Due to Standing Water @ 28.5 ft. (Artesian Effect)						
28								
30								
		@ 30 +/- ft. : Becomes Moderately Fractured						
		@ 31.0 +/- ft. : Heavy Groundwater Seepage	CK					
32								
34								
36			US		4/5			
38								
40								

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

DAMMEYER RESIDENCE

BY: DBA/MF/TSW	DATE: 9/23/99
JOB NUMBER: 9911152.1	PLATE NO.: 5

LOG OF TEST BORING NUMBER B-1

Date Excavated: 9/10/99
Equipment: BUCKET RIG
Surface Elevation (ft): 83.5 +/-

Logged by: MF
Project Manager: DBA
Depth to Water (ft): 28.5

DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES UNDISTURBED	SAMPLES BULK	PENETRATION (blows/ 6" of drive)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
42 44 46	ML/ CL	ARDATH SHALE (Ta) - No Change	US		10/15			
48 50 52 54 56 58 60		PRACTICAL REFUSAL @ 46 FT. (Drill Rig Binding Due to Groundwater/Clay) Kelley Weights: 0'-27' : 4,500 lbs 27'-52' : 3500 lbs						

SOUTHERN CALIFORNIA SOIL & TESTING, INC.	DAMMEYER RESIDENCE
BY: DBA/MF/TSW	DATE: 9/23/99
JOB NUMBER: 9911152.1	PLATE NO.: 6

LOG OF TEST BORING NUMBER B-2

Date Excavated: 9/13/99
Equipment: PORTABLE BUCKET RIG
Surface Elevation (ft): 73.0 +/-

Logged by: MF
Project Manager: DBA
Depth to Water (ft): 26.0
140 lb. Hammer/ 30 inch Drop

DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES UNDISTURBED	SAMPLES BULK	PENETRATION (blows/ 6" of drive)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
0–4	ML / CL, SM+ SP	FILL (Qaf)/ DISTURBED TOPSOIL - Brown to Gray, Wet, Soft/ Loose, CLAYEY SILT/ SILTY CLAY, SILTY SAND + SAND						
4–7.5	SM	BAYPOINT FORMATION (Qbp) - Grayish Brown to Rust, Very Moist, Medium Dense, Slightly CLAYEY SILTY SAND (fine),	US		7/17			
			CK					
		Cobbles Present @ Contact						
7.5–20	CL/ ML	Contact Subhorizontal, Irregular						
		ARDATH SHALE (Ta) - Gray to Grayish Brown to Rust, Moist, Hard, CLAYEY SILT/ SILTY CLAY (Mudstone), Highly Fractured, Bedding Generally Indistinct, Occasional Random, Discontinuous Shears						
			US		15/55			
		@ 17' - 18.5' : 1 Joint Dips 80-85 degrees/ 030 degrees 1 Joint Dips 80-85 degrees/ 090 degrees						
		@ 18' : Becomes Moderately to Highly Fractured	US		12/50			
		@ 18.5' : Discontinuous Sheared/ Disturbed Zone Appears to Dip 20 degrees/ NE						
		@ 19.5' : Possible Bedding Dips 20-25 degrees/ NE (Very Faint)						

Depth scale: 2, 4, 6, 8, 10, 12, 14, 16, 18, 20

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

DAMMEYER RESIDENCE	
BY: DBA/MF/TSW	DATE: 9/23/99
JOB NUMBER: 9911152.1	PLATE NO.: 7

LOG OF TEST BORING NUMBER B-2

Date Excavated: 9/13/99	Logged by: MF
Equipment: Portable Bucket Rig	Project Manager: DBA
Surface Elevation (ft): 73.0 +/-	Depth to Water (ft): 26.0
	140 lb Hammer / 30" Drop

DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: UNDISTURBED	SAMPLES: BULK	PENETRATION (blows/ 6" of drive)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
	CL/ ML	ARDATH SHALE (Ta) - No Change	US		15/40			
22								
24		@ 24' - 25' : 12" Discontinuous Concretionary Lens						
		* Down- Hole Logging Ended @ 25' Due To Standing Water @ 26' (Artesian Effect)	US		15/43			
26								
28		@ 28.0 +/- ft. : Heavy Groundwater Seepage						
30			US		12/50 for 3"			
32		Practical Refusal @ 31.5 ft. (Drilling Rate Less Than 6"/ Hour)						
34								
36								
38								
40								

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

DAMMEYER RESIDENCE	
BY: DBA/MF/TSW	DATE: 9/23/99
JOB NUMBER: 9911152.1	PLATE NO.: 8

LOG OF TEST BORING NUMBER B-3

Date Excavated: 9/15/99
Equipment: PORTABLE BUCKET RIG
Surface Elevation (ft): 70.5 +/-

Logged by: MF
Project Manager: DBA
Depth to Water (ft): 31.0 +/-
140 lb. Hammer/ 30 inch Drop

DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES UNDISTURBED	SAMPLES BULK	PENETRATION (blows/ 6" of drive)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
0–2	ML/ CL	FILL (Qaf) / DISTURBED TOPSOIL - Brown, Wet, Soft, SANDY CLAYEY SILT						
2–5	SM/ SC	WEATHERED BAYPOINT FORMATION (Qbp) - Brown to Orange-Brown, Very Moist, Loose to Medium Dense, Slightly CLAYEY SILTY SAND						
5–7	SM/ SC	BAYPOINT FORMATION (Qbp) -Orange Brown to Brown to Gray, Moist, Medium Dense, Slightly CLAYEY SILTY SAND (fine),	US		3/11			
7–20	CL/ ML	Contact Subhorizontal, Irregular ARDATH SHALE (Ta) - Gray to Grayish Brown to Rust, Moist, Hard, CLAYEY SILT/ SILTY CLAY (Mudstone), Highly Fractured, Numerous Random, Discontinuous Shears, Waxy Parting Surfaces	US		7/20			
		@ 8'-10' : Joint Dips 80 degrees/ 080 degrees						
		@ 14.5' : 6" Concretionary Lens	US		13/35			
		@ 18' & 19.5' : Thin Rust-Stained Beds, Appear to Dip 10-15 degrees/ S-SW (Faint)						

Depth scale: 2, 4, 6, 8, 10, 12, 14, 16, 18, 20

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

DAMMEYER RESIDENCE	
BY: DBA/MF/TSW	DATE: 9/23/99
JOB NUMBER: 9911152.1	PLATE NO.: 9

LOG OF TEST BORING NUMBER B-3

Date Excavated: 9/15/99
Equipment: Portable Bucket Rig
Surface Elevation (ft): 70.5 +/-

Logged by: MF
Project Manager: DBA
Depth to Water (ft): 31.0
140 lb Hammer / 30" Drop

DEPTH (ft)	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: UNDISTURBED	SAMPLES: BULK	PENETRATION (blows/ 6" of drive)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
		ARDATH SHALE (Ta) - No Change	US		19/34			
22	CL/ ML	@ 21'-23' : Joint/Shear Plane with up to 3" Remolded Clay, Dips 75 degrees / 310 degrees						
24		@ 23' : Bedding Plane Shear, 1" +/- Thick Zone with Multiple Remolded Clay/ Sheared Silt Seams, Dips 25 degrees/ 250-270 degrees	US		11/50 for 5"			
26		@ 23'-25' : Dark Gray, Relatively Unfractured Mudstone Bed						
		@ 26' : 3" Rust Silt Bed, Dips 35 degrees / 260 degrees						
28		@ 27' - 29' : Joint/ Shear Plane Dips 45 degrees/ 290 degrees; 1" Remolded Clay *over* 1" Fractured Concretionary Layer *over* 1/2" Remolded Clay, Caliche & Gypsum	US		7/26			
30–32		@ 31'-35' : Ardath Shale Becomes Well Cemented, Moderately Fractured, Concretions More Common						
34		@31' : Light Point- Source Seepage						
		@ 34.5' : Moderate Groundwater Seepage	US		15/50 for 5"			
36–40		Practical Refusal @ 31.5 ft.						
		Note: Water Level @ 31.0' +/- @ 8:00 AM 09/16/99 (Artesian Effect)						

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

DAMMEYER RESIDENCE	
BY: DBA/MF/TSW	DATE: 9/23/99
JOB NUMBER: 9911152.1	PLATE NO.: 10

LOG OF TEST BORING NUMBER B-4

Date Excavated: 9/24/99 Logged by: RD
Equipment: BEAVER RIG Project Manager: DBA
Surface Elevation (ft): 83.0' +/- Depth to Water (ft): N/A

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: UNDISTURBED	SAMPLES: BULK	PENETRATION (blows/ ft. of drive)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
0–8	SC/ SM	FILL - Dark Brown, Moist to Wet, Medium Dense, SILTY CLAYEY SAND			23			
8–9.5	SM/ SC	FILL AND/OR TOPSOIL - Dark Brown, Moist, Medium Dense, CLAYEY SILTY SAND			73+			
9.5–11	ML/ CL	POINT LOMA FORMATION - Olive to Tan, Moist, Very Hard, CLAYEY SILT / SILTY CLAY			78			
		Boring Ended at 11'						

Depth scale: 2, 4, 6, 8, 10, 12, 14, 16, 18, 20

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

DANMEYER RESIDENCE	
BY: DBA/MF/KMS	DATE: 11/15/99
JOB NUMBER: 9911152.2	PLATE NO.: 3

LOG OF TEST BORING NUMBER B-5

Date Excavated: 9/24/99
Equipment: BEAVER RIG
Surface Elevation (ft): 83.0' +/-

Logged by: RD
Project Manager: DBA
Depth to Water (ft): N/A

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: UNDISTURBED	SAMPLES: BULK	PENETRATION (blows/ ft. of drive)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
0	SM/ SC	FILL - Dark Brown, Moist to Wet, Medium Dense, SILTY CLAYEY SAND						
5					22			
9					68			
12					75/5"			
13.5		Refusal at 13.5' on Apparent Cobbles						

SOUTHERN CALIFORNIA SOIL & TESTING, INC.	DANMEYER RESIDENCE	
	BY: DBA/MF/KMS	DATE: 11/15/99
	JOB NUMBER: 9911152.2	PLATE NO.: 4

LOG OF TEST BORING NUMBER B-6

Date Excavated: 9/24/99
Equipment: BEAVER RIG
Surface Elevation (ft): 70.0' +/-

Logged by: RD
Project Manager: DBA
Depth to Water (ft): N/A

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: UNDISTURBED	SAMPLES: BULK	PENETRATION (blows/ ft. of drive)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
0–10.5	SM/ SC	FILL - Dark Brown, Very Moist to Wet, Medium Dense, CLAYEY SILTY SAND						
~5					10			
~10					50/2"			
10.5		Bottom at 10.5' on Point Loma Formation as Seen in Sampler Tip						

Depth scale: 2, 4, 6, 8, 10, 12, 14, 16, 18, 20

SOUTHERN CALIFORNIA SOIL & TESTING, INC.

DANMEYER RESIDENCE	
BY: DBA/MF/KMS	DATE: 11/15/99
JOB NUMBER: 9911152.2	PLATE NO.: 5

LOG OF TEST BORING NUMBER B-7

Date Excavated: 9/24/99
Equipment: BEAVER RIG
Surface Elevation (ft): 83.0' +/-

Logged by: RD
Project Manager: DBA
Depth to Water (ft): N/A

DEPTH (ft)	USCS	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLES: UNDISTURBED	SAMPLES: BULK	PENETRATION (blows/ ft. of drive)	MOISTURE (%)	DRY UNIT WT. (pcf)	LABORATORY TESTS
0	SC/ SM	FILL - Dark Brown, Very Moist to Wet, Medium Dense, SILTY CLAYEY SAND						
5					29			
10	SM/ SC	BAY POINT FORMATION - Tan to Brown, Very Moist, Medium Dense to Dense, CLAYEY SILTY SAND			89+			
15					60/6			
17.5		Refusal at 17.5' on Cobbles						

SOUTHERN CALIFORNIA SOIL & TESTING, INC.	DANMEYER RESIDENCE		
	BY: DBA/MF/KMS	DATE: 11/15/99	
	JOB NUMBER: 9911152.2	PLATE NO.: 6	

APPENDIX B
UNIFIED SOIL CLASSIFICATION SYSTEM (U.S.C.S.)
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	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	SM	Silty sands, poorly graded sand and silty mixtures.
	SC	Clayey sands, poorly graded sand and clay mixtures.

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

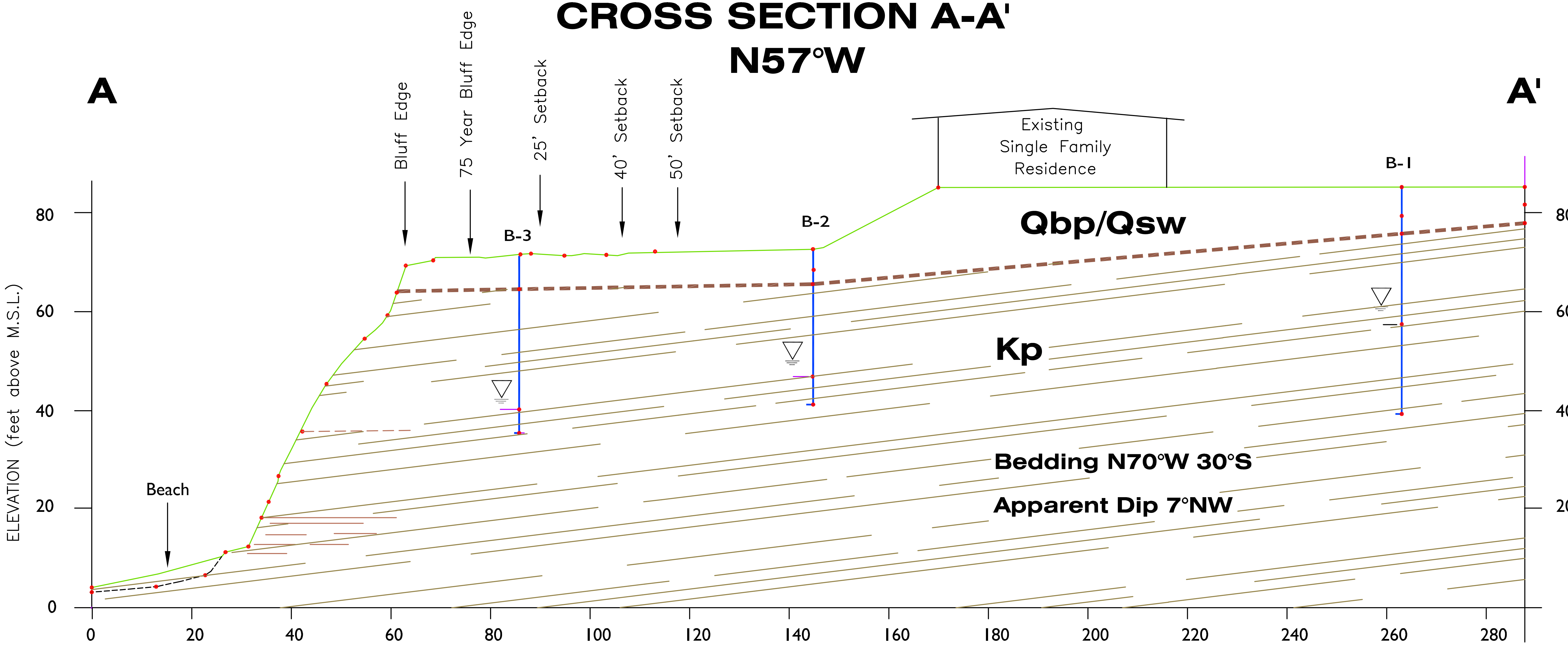
SILTS AND CLAYS

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

APPENDIX C

Cross Sections A-A' to E-E' from GEI reports dated 2/9/2000 and 10/3/2001

CROSS SECTION A-A'
N57°W



LEGEND

Qbp/Qsw Quaternary Baypoint Formation/Quaternary Slopewash

Kp Cretaceous Point Loma Formation (Kp1 highly fractured massively bedded zone; Kp2 highly fractured, more apparent bedding)

B-3 Boring location by Southern California Soil and Testing

Approximate Geologic contact

Approximate Apparent Dip of Bedding

Groundwater

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.

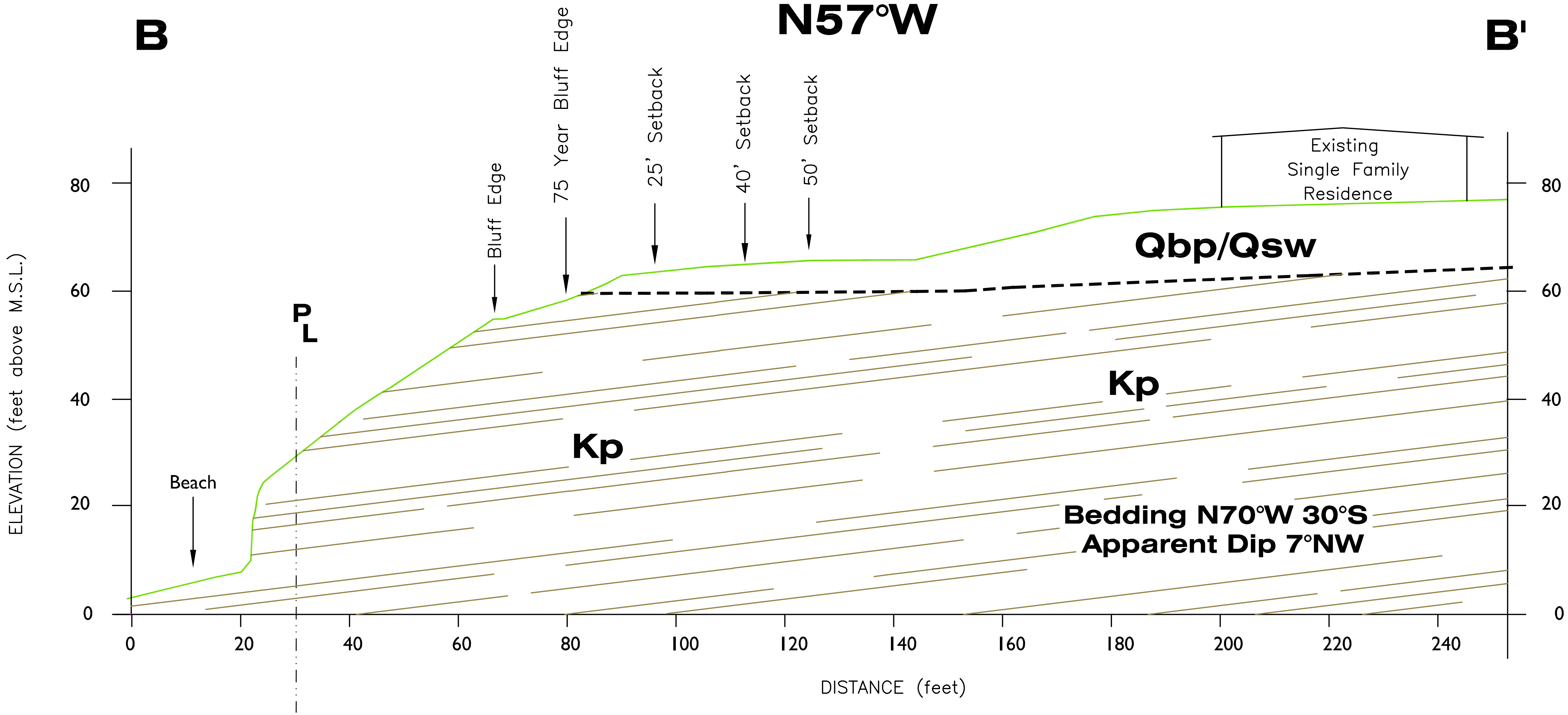
Lowenthal Residential Project
1720 Torrey Pines Road
La Jolla, CA.

Geotechnical Exploration, Inc.

July 2024

01-8018-AA

CROSS SECTION B-B'
N57°W



LEGEND

Qbp/Qsw Quaternary Baypoint Formation/Quaternary Slopewash

Kp Cretaceous Point Loma Formation (Kp1 highly fractured massively bedded zone; Kp2 highly fractured, more apparent bedding)

B-3 Boring location by Southern California Soil and Testing

Approximate Geologic contact

Approximate Apparent Dip of Bedding

Groundwater

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.

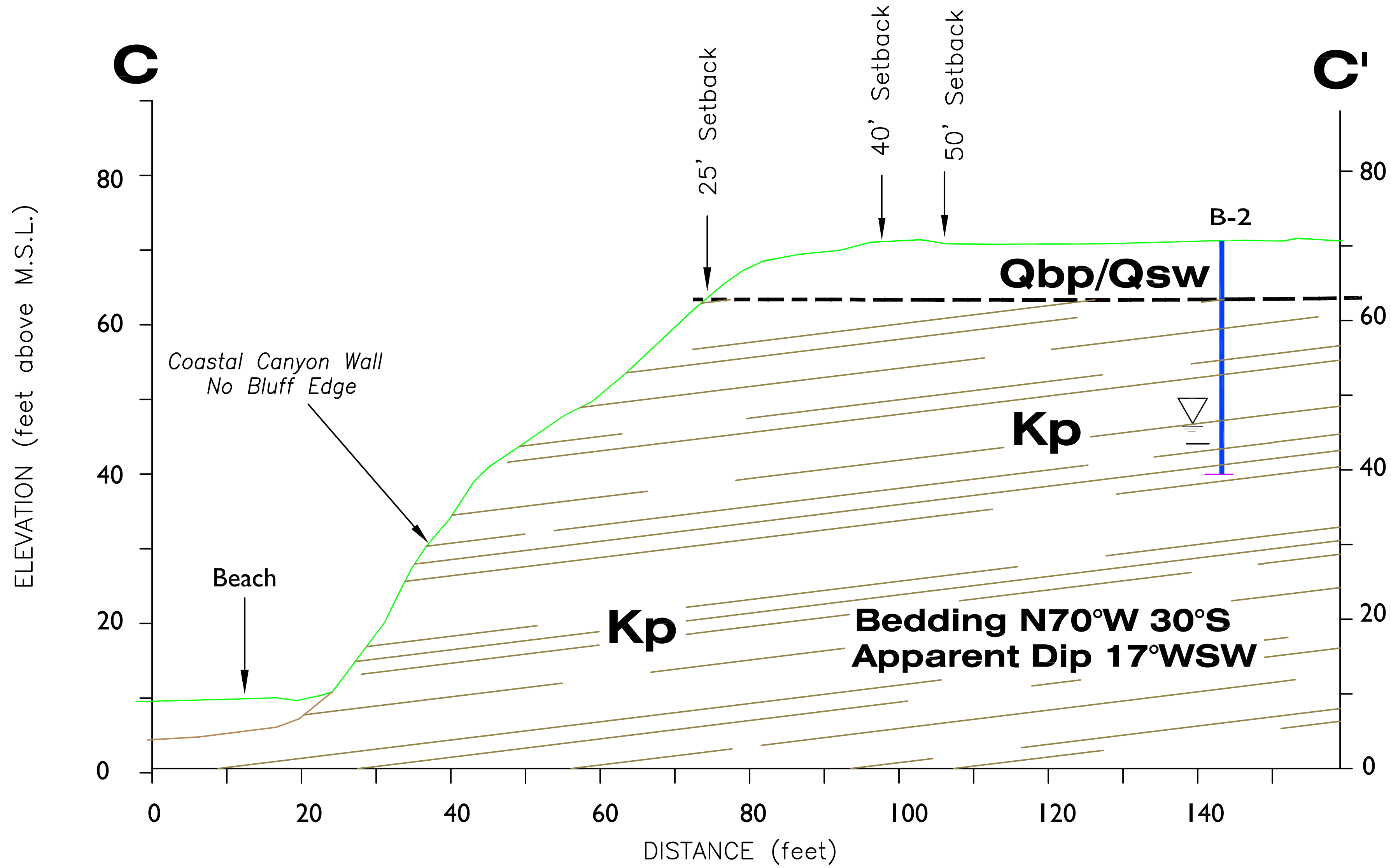
Lowenthal Residential Project
1720 Torrey Pines Road
La Jolla, CA.

Geotechnical Exploration, Inc.

July 2024

01-8018-BB

CROSS SECTION C-C'
N78°E



LEGEND

Qbp/Qsw Quaternary Baypoint Formation/Quaternary Slopewash

Kp Cretaceous Point Loma Formation (Kp1 highly fractured massively bedded zone; Kp2 highly fractured, more apparent bedding)

B-2 Boring location by Southern California Soil and Testing

Approximate Geologic contact

Approximate Apparent Dip of Bedding

Groundwater

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.

01-8018-CC

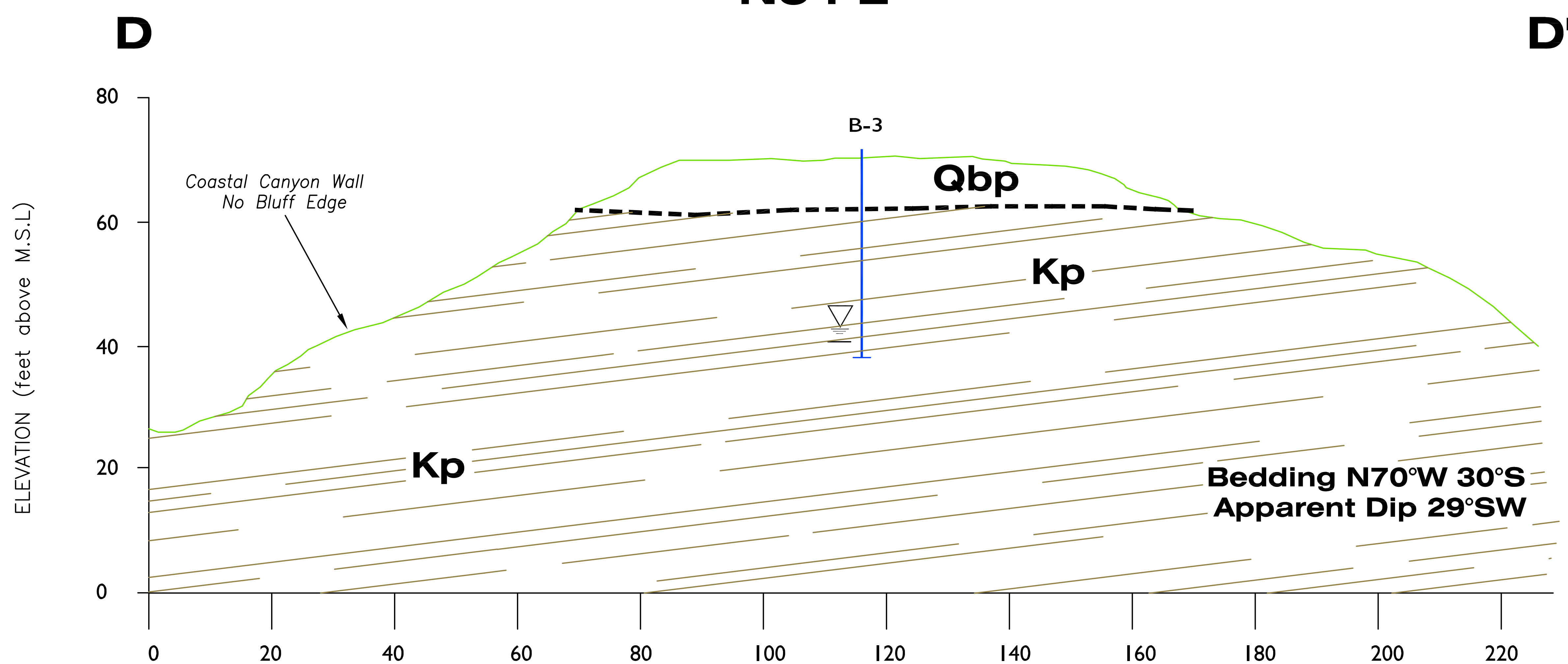
Lowenthal Residential Project
1720 Torrey Pines Road
La Jolla, CA.

Geotechnical Exploration, Inc.

July 2024

CROSS SECTION D-D'

N34°E



LEGEND

Qbp/Qsw Quaternary Baypoint Formation/Quaternary Slopewash

Kp Cretaceous Point Loma Formation (Kp1 highly fractured massively bedded zone; Kp2 highly fractured, more apparent bedding)

B-3 Boring location by Southern California Soil and Testing

Approximate Geologic contact

Approximate Apparent Dip of Bedding

Groundwater

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.

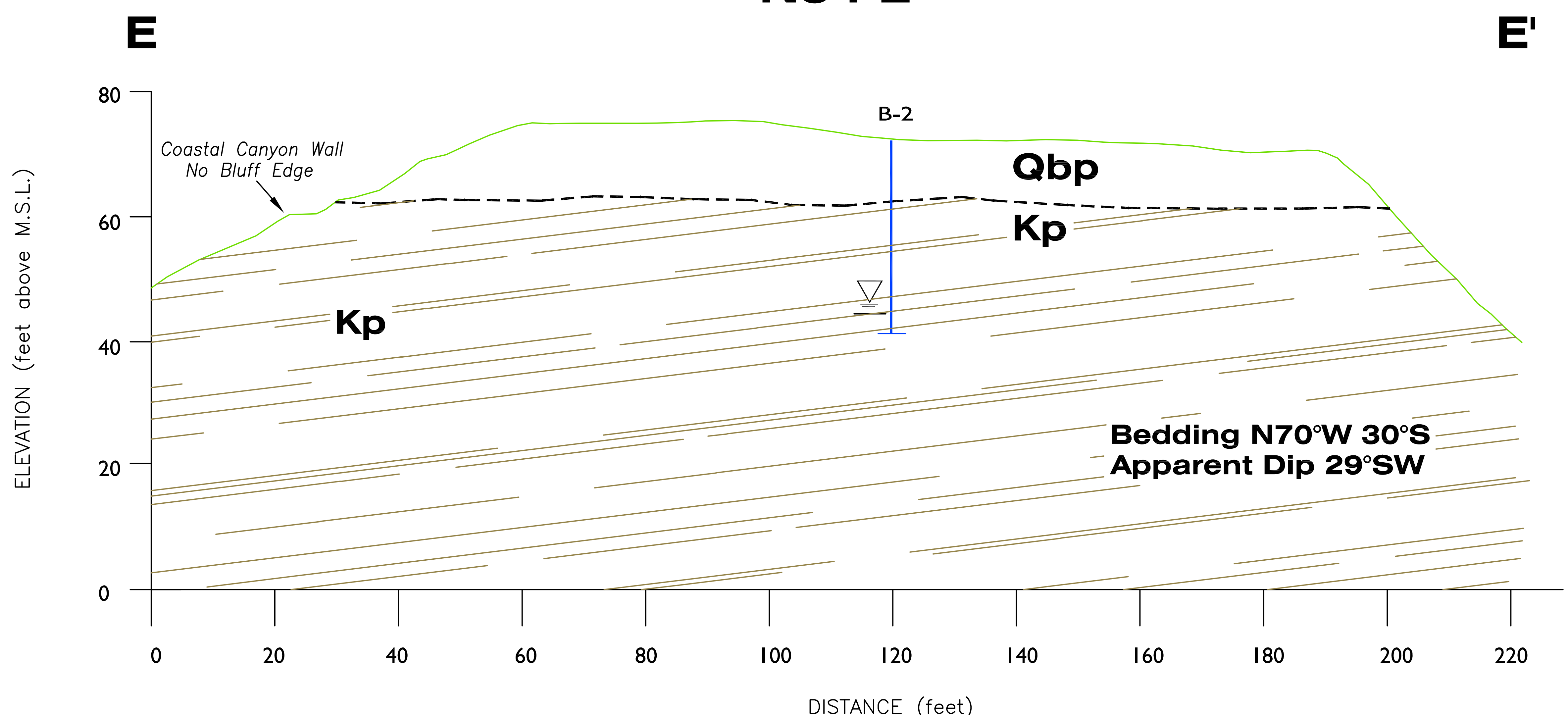
01-8018-DD

Lowenthal Residential Project
1720 Torrey Pines Road
La Jolla, CA.

Geotechnical Exploration, Inc.

July 2024

CROSS SECTION E-E'
N34°E



LEGEND

Qbp/Qsw	Quaternary Baypoint Formation/Quaternary Slopewash
Kp	Cretaceous Point Loma Formation (Kp1 highly fractured massively bedded zone; Kp2 highly fractured, more apparent bedding)
B-3	Boring location by Southern California Soil and Testing
	Approximate Geologic contact
	Approximate Apparent Dip of Bedding
	Groundwater

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.

01-8018-EE

Lowenthal Residential Project
1720 Torrey Pines Road
La Jolla, CA.

Geotechnical Exploration, Inc.

July 2024

APPENDIX D

REGIONAL GEOLOGIC DESCRIPTION

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 Nación and Rose Canyon Fault zones has broken up this sequence into a number of distinct fault blocks in the southwestern part of the county. Northwestern portions of the county are relatively undeformed by faulting (Demere, 1997).

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

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

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

In California, major earthquakes can generally be correlated with movement on active faults. As defined by the California Division of Mines and Geology, now the California Geological Survey (CGS), an "active" fault, described by CGS (2018) as a Holocene-Active fault, is one that has had (ground) surface displacement within Holocene time, the last 11,700 (CGS, 2018). In addition, "potentially active fault" has been amended to Pre-Holocene fault: a fault whose recency of past movement is older than 11,700 years, and thus does not meet the criteria of Holocene-Active fault as defined in the State Mining and Geology Board regulations.

For the City of San Diego, the lead agency for this project, a three-tier fault classification is used as follows:

- *Active Faults*: Faults that have demonstrable surface displacement during Holocene time.
- *Potentially Active Faults*: Faults with Quaternary displacement but Holocene surface displacement is indeterminate.
- *Inactive Faults*: Pre-Quaternary faults.

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. The youngest paleoearthquake that cuts the early historical living surface is likely the 1862 San Diego earthquake that had an estimated magnitude of M6 (Singleton et al., 2019). Paleoseismic trenches at the Presidio Hills Golf Course on the main trace of the Rose Canyon Fault contained evidence for historical ground rupturing earthquakes as recently as 1862 and the mid-1700s. Results of the study also suggest the Rose Canyon Fault has a ~700-800-year recurrence interval (Singleton et al., 2019).

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). 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. 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.

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.

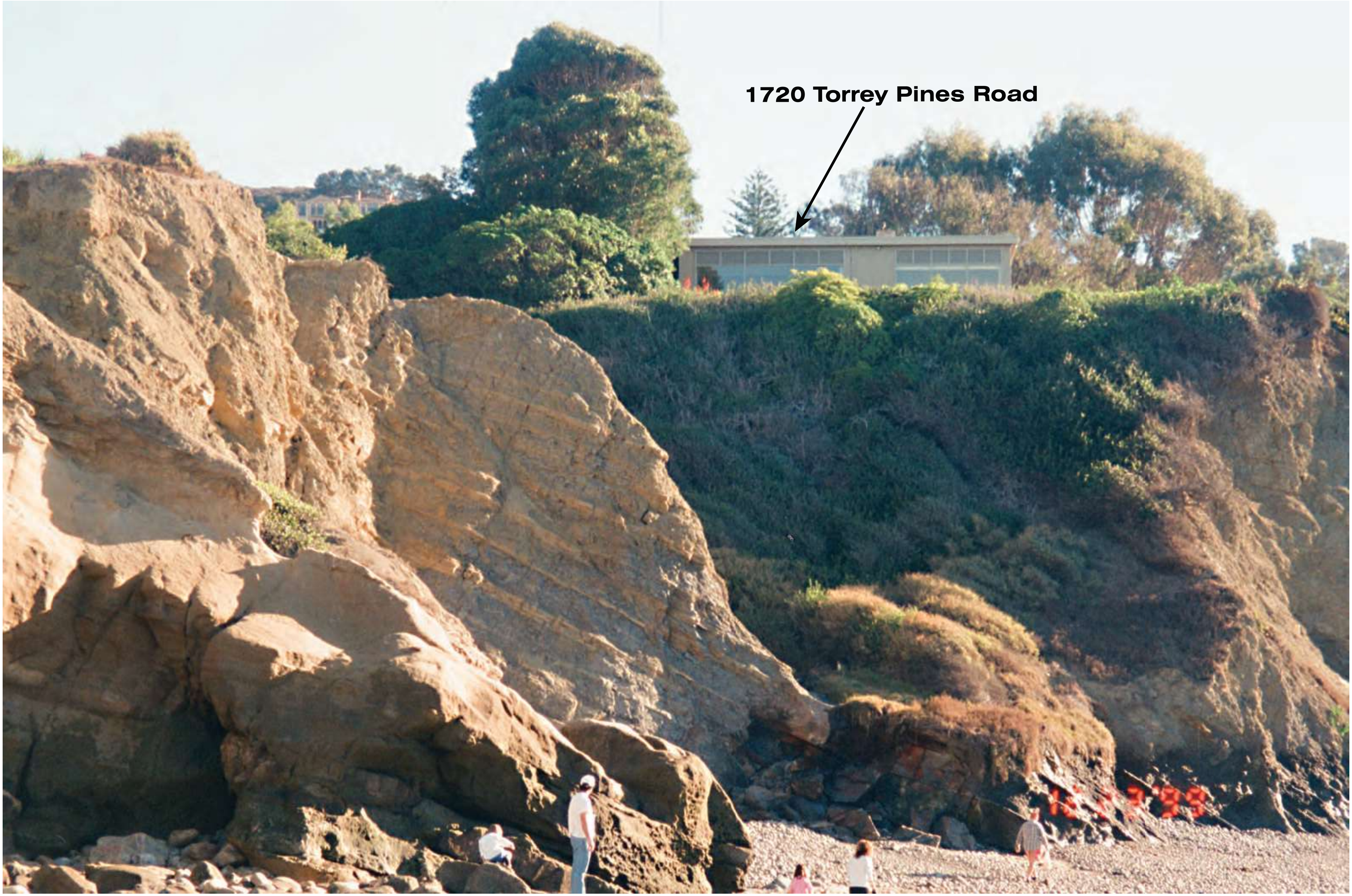
This event's aftershock zone extends significantly to the northwest, overlapping with the portion of the fault system that is thought to have ruptured in 1892. Ground motions for the April 4, 2010, main event, recorded at stations in San Diego and reported by the California Strong Motion Instrumentation 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.

APPENDIX E

Bluff Photographs
Photo Nos. 1-4 (1999)
Photo Nos. 5-6 (2024)
Photo Nos. 7-13 (1928-1966)



1

(1999)

01-8018-PIC-1.AI

2 (1999)

01-8018-PIC-1.AI

3

(1999)

01-8018-PIC-3.AI

4

(1999)

01-8018-PIC-4.AI

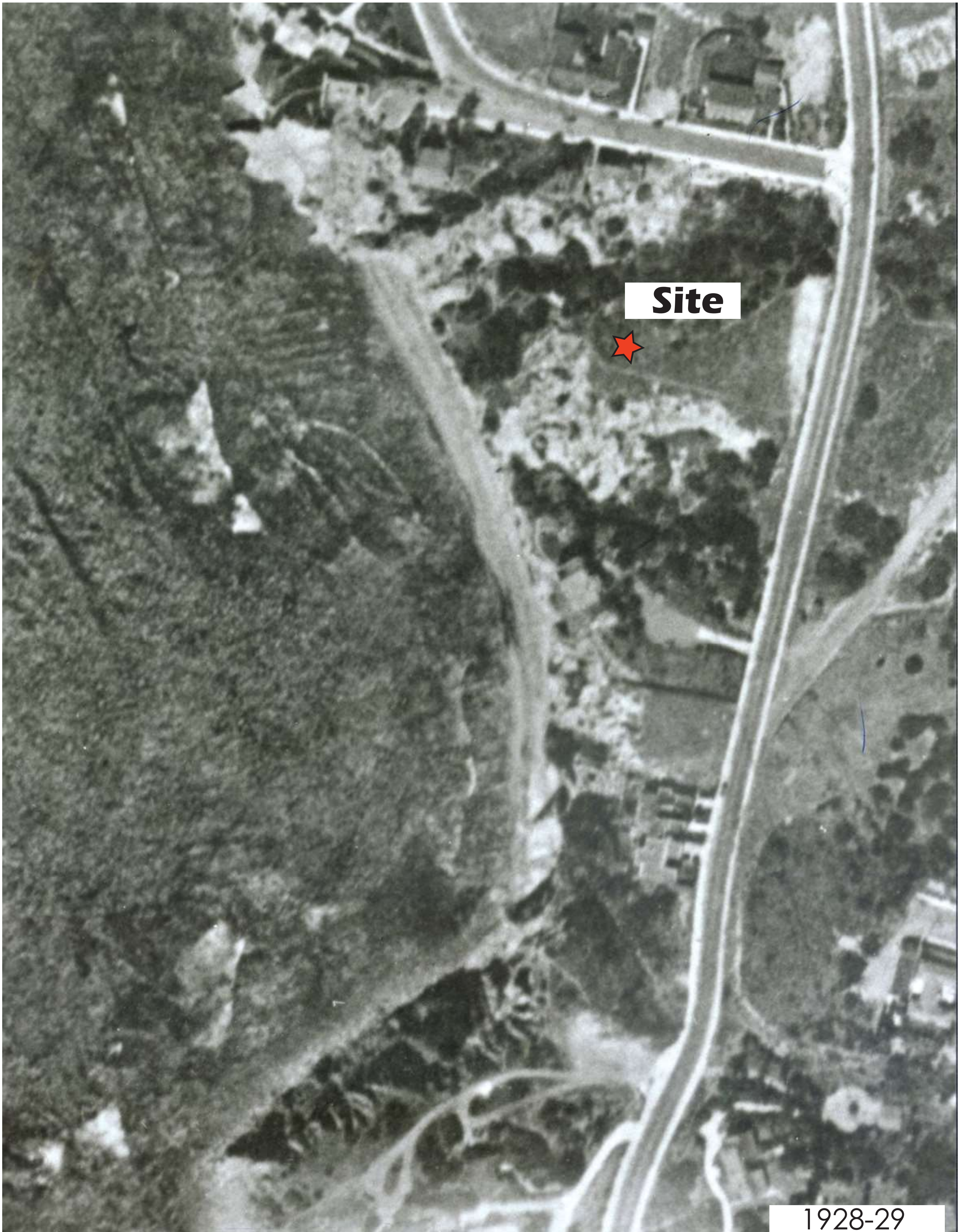
5

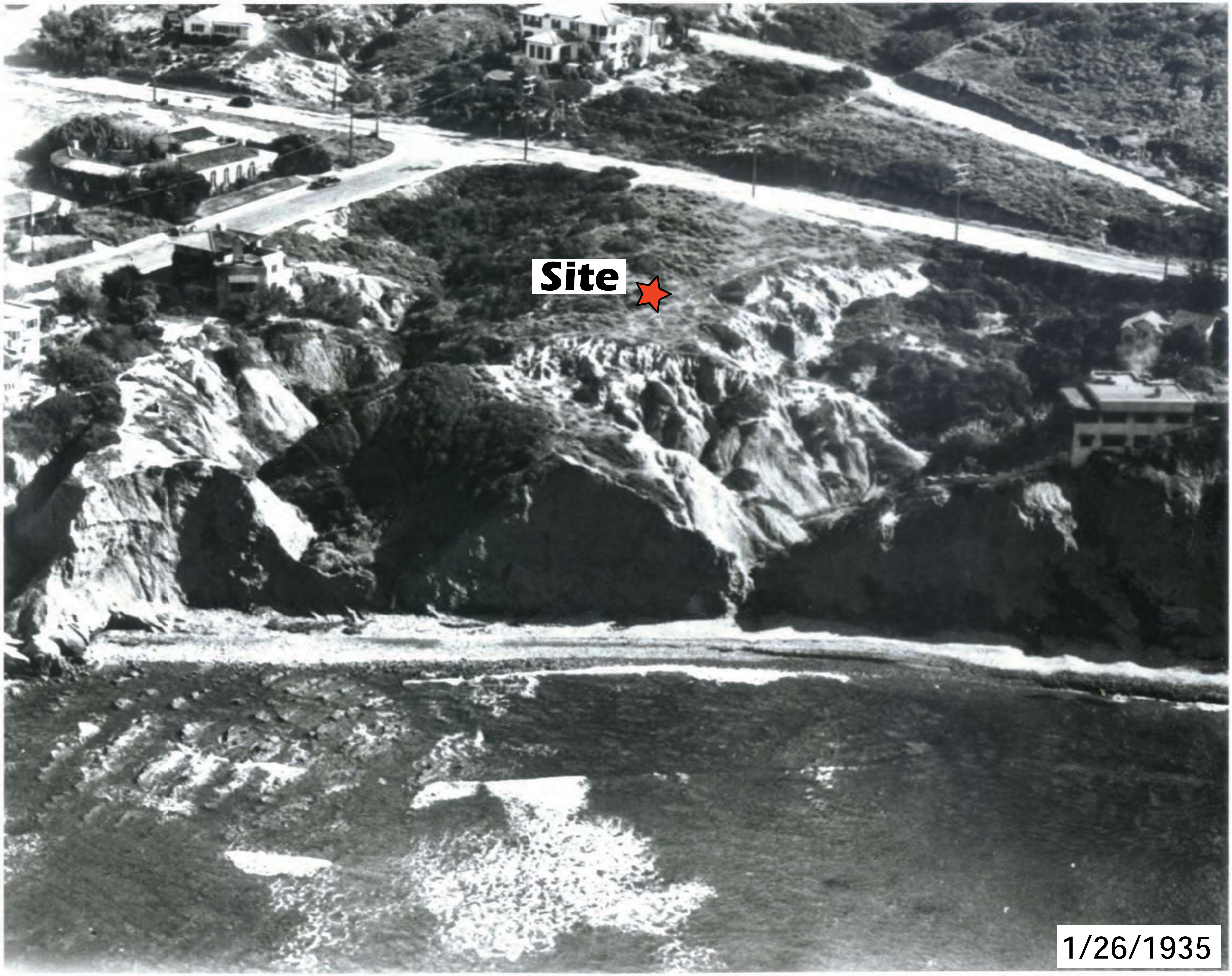
(2024)

01-8018-PIC-5.AI

6 (2024)

01-8018-PIC-6.AI


Site
1928-29


Site
1/26/1935


Site
Nov. 1936
ERICHSON-S.F.
36-277-1


Site
2-16-1949


Site
4-11-1953

12

12-9-1966

APPENDIX F

SLOPE STABILITY CALCULATIONS WITH SLIDE 6 COMPUTER PROGRAM

Proposed Lowenthal Residence Additions

Job No. 01-8018

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

The program calculates the factor of safety against shear failure for potential slide surfaces over a selected range. We chose the range of slide surfaces where failures are most likely to occur. When analyzing the circular surfaces, the printout shows a block with contours of different colors and shades that correspond to the different factors of safety calculated that can be obtained for the analyzed range of slide surfaces for Section F-F′ and G-G′, which include the most unfavorable slope conditions at the site (see attached printouts). For the block analysis, the printout shows block surfaces from the centers of rotation. The green circular and block surface displayed in the printout is the lowest possible factor of safety located within the specified search range of each cross-section analysis. Soil strength values, geometry, and water conditions (seepage was not encountered) used in the program were based on geological information from the site, obtained from a past geotechnical report from another geotechnical consultant (refer to the report). Direct shear test results from the on-site soils were performed and were used for the gross slope stability analysis. Shear strength values were conservatively adjusted.

The factors of safety for static global stability of circular and block slide failures were calculated and yielded a factor of safety value greater than the acceptable value of 1.5. In the analysis, we included surcharge loading due to the structure foundation and the concrete slab. A lateral triangular earth pressure was used to simulate the basement retaining wall earth support.

Apparent dips were calculated for each cross section by the project geologist using the obtained strikes and dips of the geologic structure at project the site.

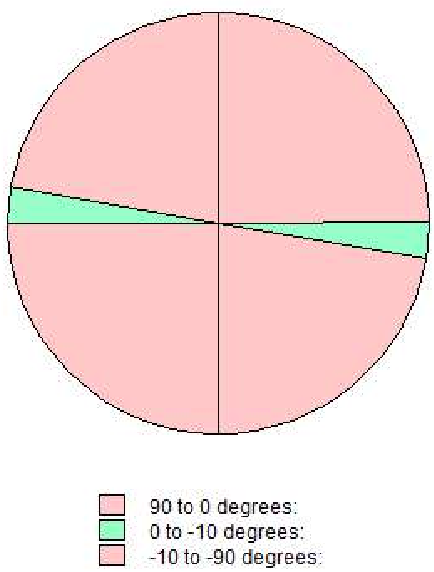
The slope stability program incorporates the bedding using the following layer in the program:

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
POINT LOMA (Kp) with Bedding		125	Generalized Anisotropic			None	0	User Defined 1

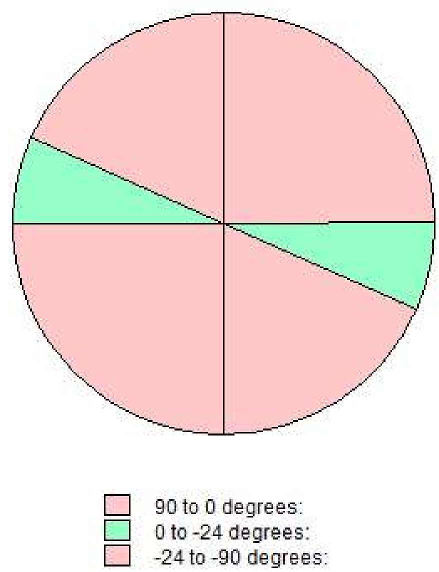
The layer shown above is a function of two separate independent layers assigned individual strength parameters.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
POINT LOMA		125	Mohr-Coulomb	1100	25	None	0	
BEDDING		125	Mohr-Coulomb	375	24	None	0	

The following dip angles were assigned to the Point Loma (Kp) with bedding:



Section F-F'

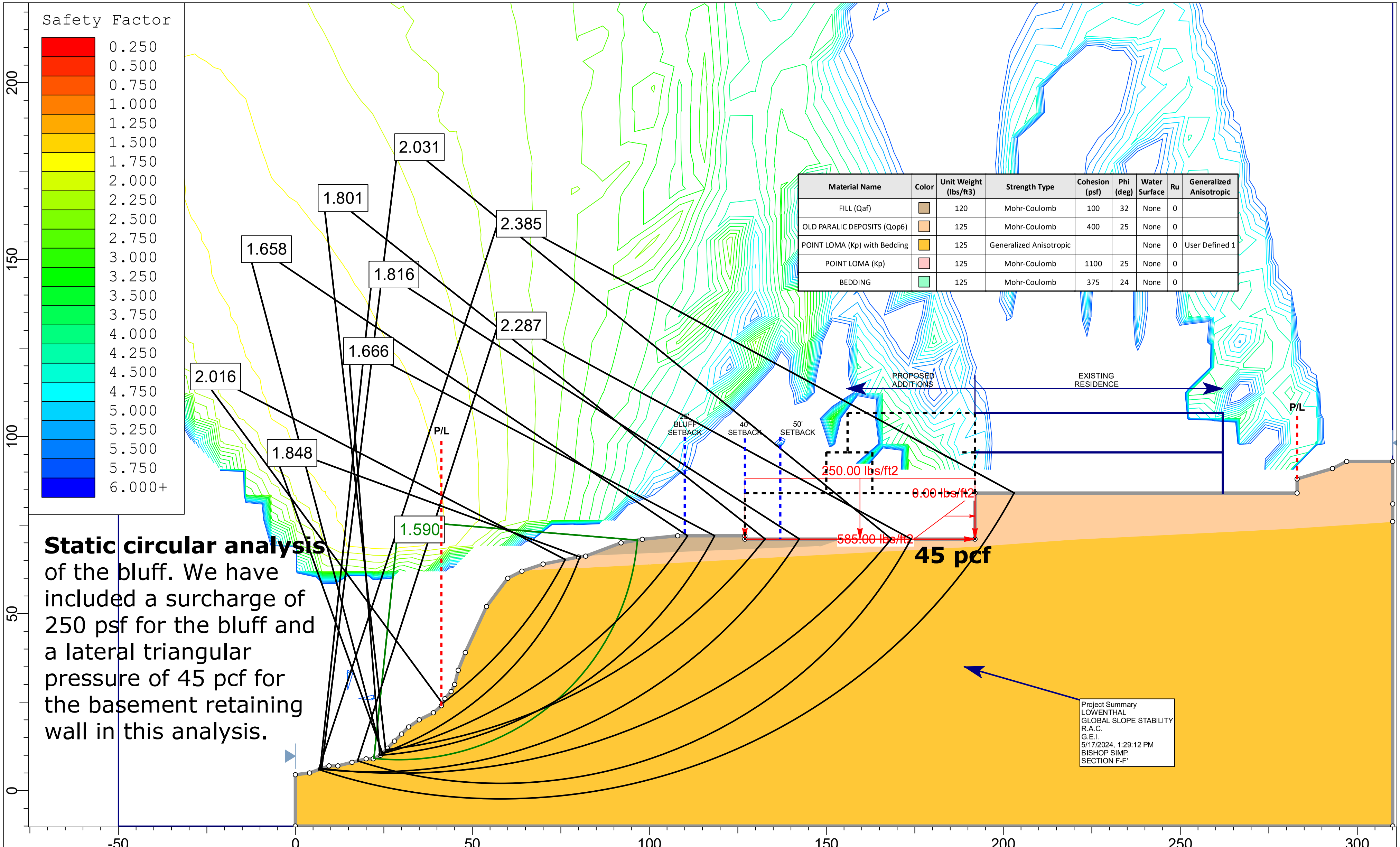


Section G-G'

Once the static gross stability was determined, a seismic analysis was performed for the same analyzed sections with the same conditions. The seismic analysis yielded a factor of safety value above the acceptable value of 1.15 as required by the City of San Diego and the State of California.

The surficial slope stability calculations were performed on the bluff face for Section F-F' only as G-G' goes beyond the property line. Using a geotechnical accepted equation for infinite slopes, the calculations were performed by assuming that the upper 1 meter (3.28 feet) of the soils were saturated. It is our professional opinion that the surficial failures are likely to occur in the upper 1 meter and per Special Publication 117A (2008, page 27): for infinite slope analysis, the minimum assumed depth of soil saturation is the smaller of either a depth of one meter or depth to firm bedrock. The analyzed slope segment was assumed to have an infinite length in the calculations. Based on the current existing slope, the calculations yielded the factor of safety against shear failure above the acceptable value of 1.5 for a sliding block 1-meter high against the soil shear strength frictional and cohesion strength opposing the driving force.

It is our professional opinion that the construction will not destabilize the slopes, adjacent structures, or City Right-of-Way, following our geotechnical report recommendations.

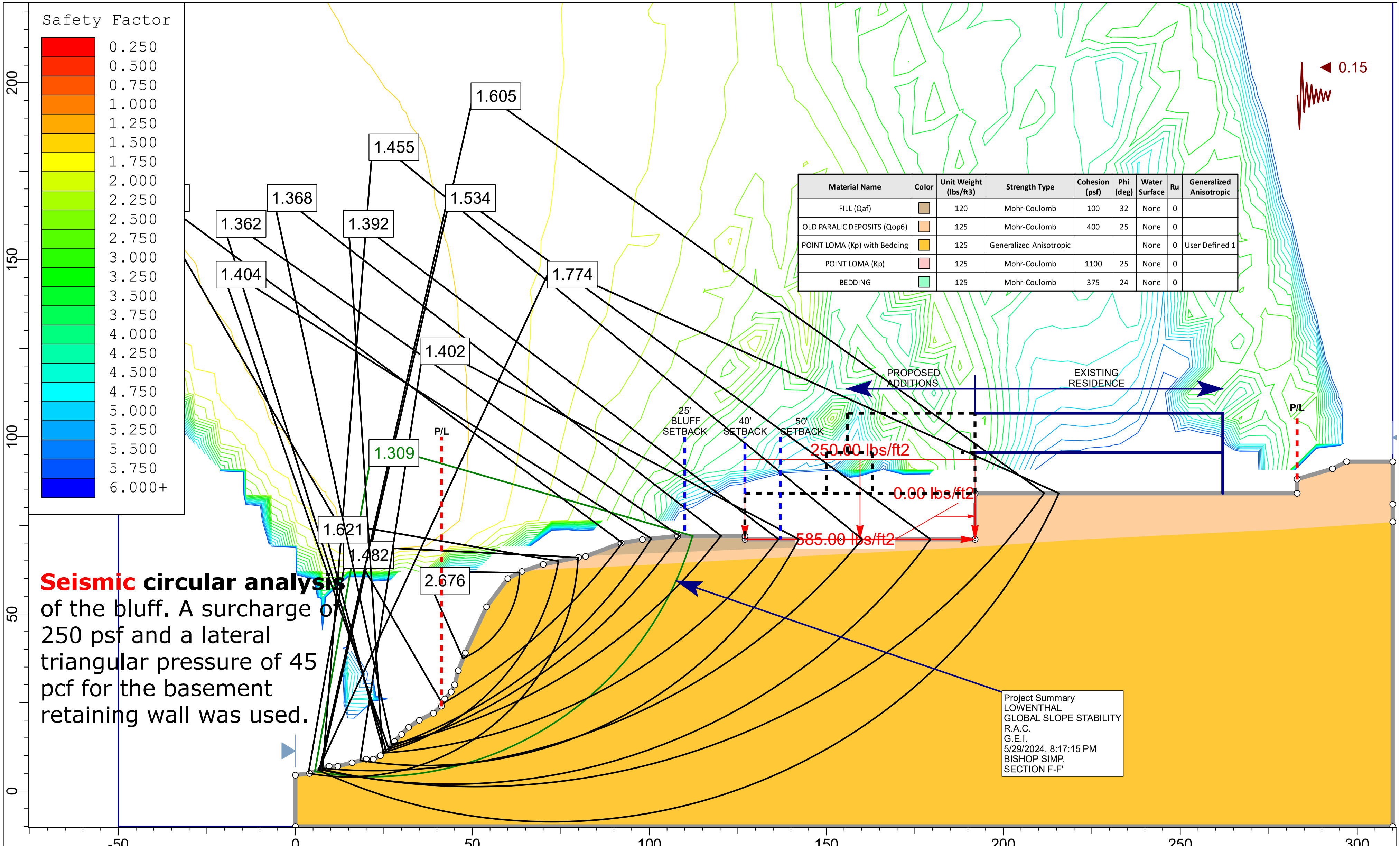


Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
FILL (Qaf)		120	Mohr-Coulomb	100	32	None	0	
OLD PARALIC DEPOSITS (Qop6)		125	Mohr-Coulomb	400	25	None	0	
POINT LOMA (Kp) with Bedding		125	Generalized Anisotropic			None	0	User Defined 1
POINT LOMA (Kp)		125	Mohr-Coulomb	1100	25	None	0	
BEDDING		125	Mohr-Coulomb	375	24	None	0	

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	LOWENTHAL		**SECTION F-F'**
Analysis Description	GLOBAL SLOPE STABILITY		
Drawn By R.A.C.	*Scale* 1:450	*Company*	G.E.I.
Date	5/17/2024, 1:29:12 PM	*File Name*	JOB NO. 01-8018_S(F)_01.slim

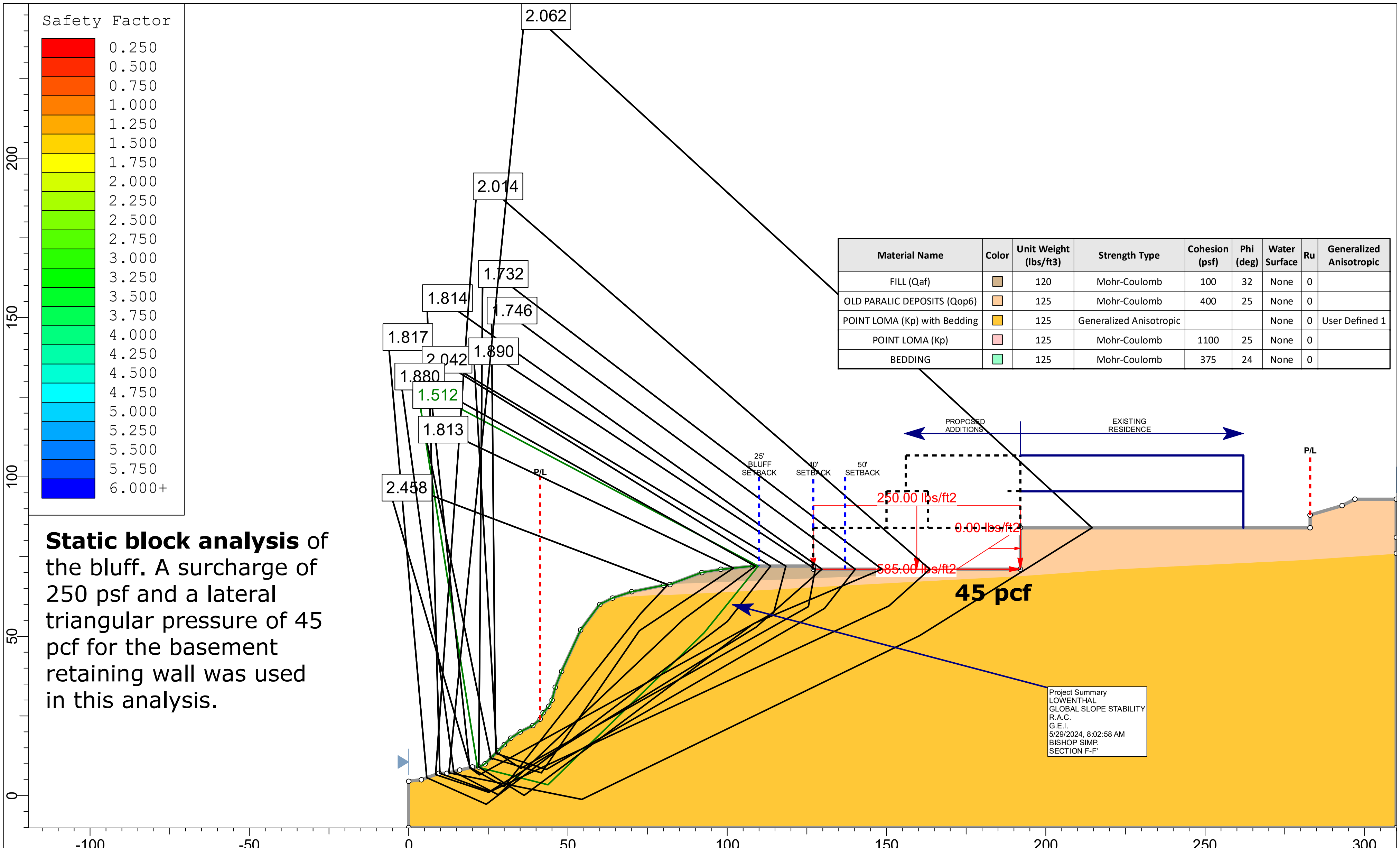


Seismic circular analysis of the bluff. A surcharge of 250 psf and a lateral triangular pressure of 45 pcf for the basement retaining wall was used.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	LOWENTHAL		**SECTION F-F'**
Analysis Description	GLOBAL SLOPE STABILITY		
Drawn By R.A.C.	*Scale* 1:450	*Company* G.E.I.	
Date 5/29/2024, 8:17:15 PM		*File Name* JOB NO. 01-8018_S(F)_01gSHAKE.slim	



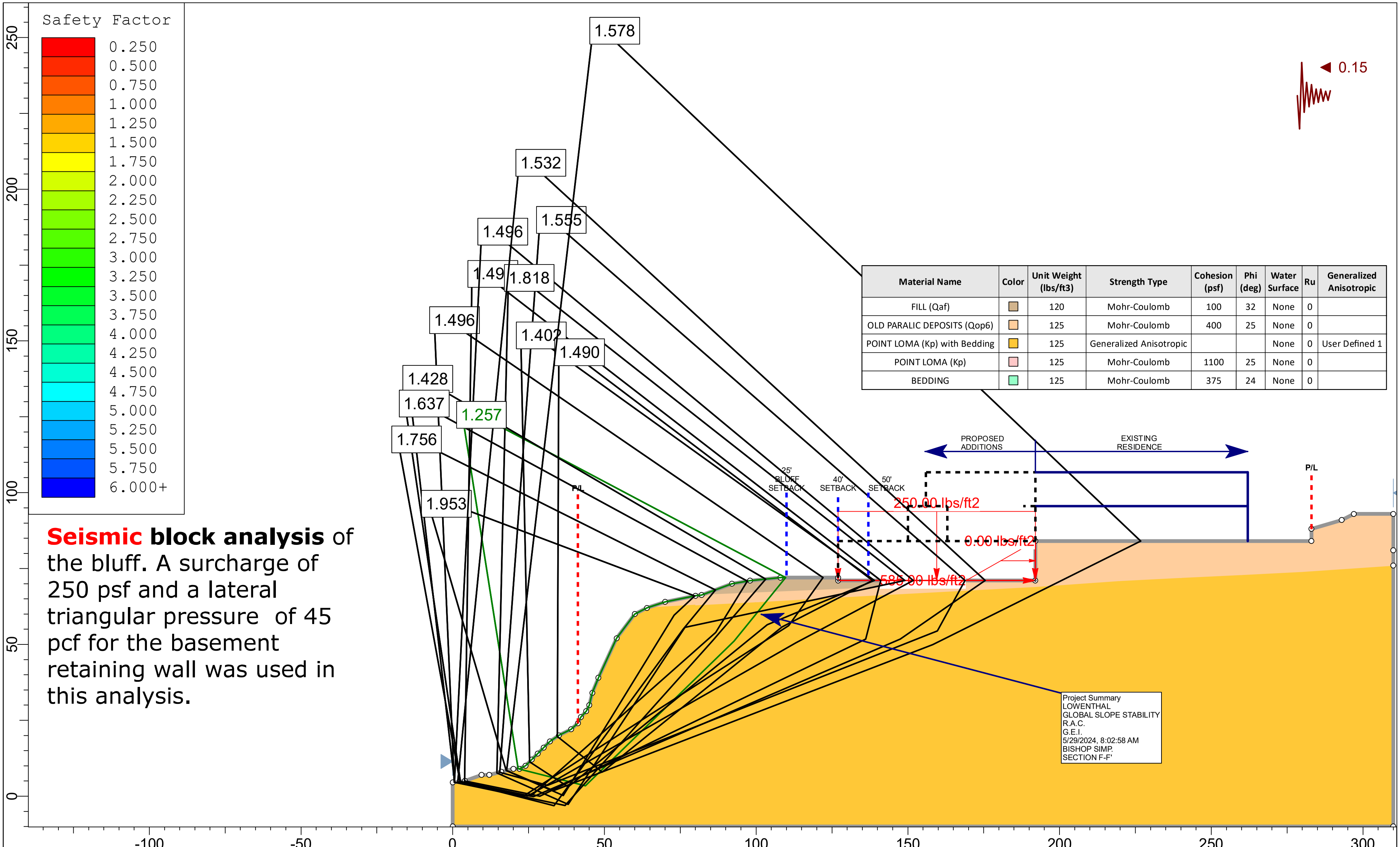
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
FILL (Qaf)		120	Mohr-Coulomb	100	32	None	0	
OLD PARALIC DEPOSITS (Qop6)		125	Mohr-Coulomb	400	25	None	0	
POINT LOMA (Kp) with Bedding		125	Generalized Anisotropic			None	0	User Defined 1
POINT LOMA (Kp)		125	Mohr-Coulomb	1100	25	None	0	
BEDDING		125	Mohr-Coulomb	375	24	None	0	

Static block analysis of the bluff. A surcharge of 250 psf and a lateral triangular pressure of 45 pcf for the basement retaining wall was used in this analysis.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	LOWENTHAL		SECTION F-F'
Analysis Description	GLOBAL SLOPE STABILITY		
Drawn By	R.A.C.	Scale	1:500
Company	G.E.I.		
Date	5/29/2024, 8:02:58 AM	File Name	JOB NO. 01-8018_S(F)_02.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
FILL (Qaf)		120	Mohr-Coulomb	100	32	None	0	
OLD PARALIC DEPOSITS (Qop6)		125	Mohr-Coulomb	400	25	None	0	
POINT LOMA (Kp) with Bedding		125	Generalized Anisotropic			None	0	User Defined 1
POINT LOMA (Kp)		125	Mohr-Coulomb	1100	25	None	0	
BEDDING		125	Mohr-Coulomb	375	24	None	0	

Seismic block analysis of the bluff. A surcharge of 250 psf and a lateral triangular pressure of 45 pcf for the basement retaining wall was used in this analysis.

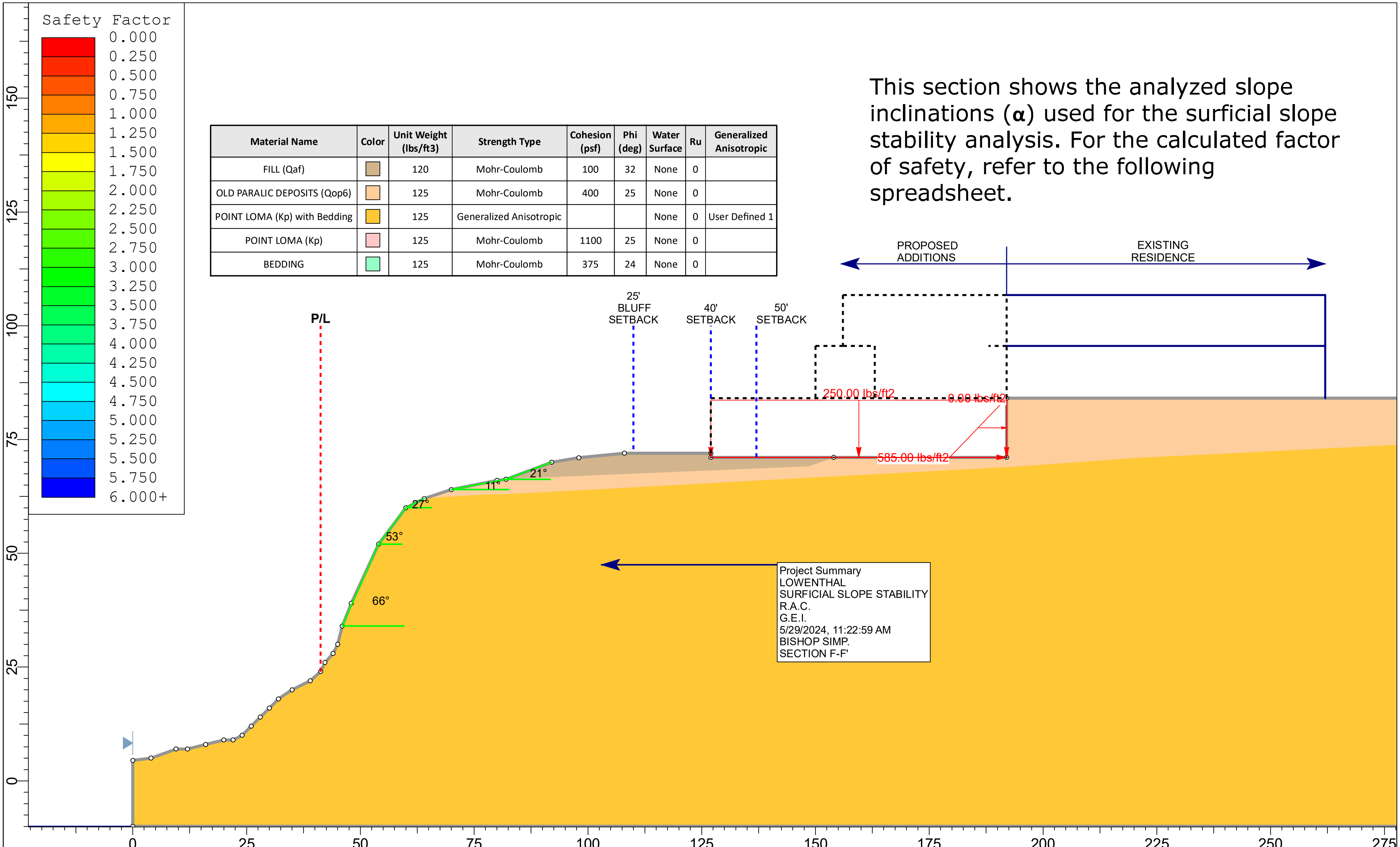
Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	LOWENTHAL		SECTION F-F'
Analysis Description	GLOBAL SLOPE STABILITY		
Drawn By	R.A.C.	Scale 1:525	Company G.E.I.
Date	5/29/2024, 8:02:58 AM	File Name	JOB NO. 01-8018_S(F)_02w_0.15gSHAKE.slim

This section shows the analyzed slope inclinations (α) used for the surficial slope stability analysis. For the calculated factor of safety, refer to the following spreadsheet.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
FILL (Qaf)		120	Mohr-Coulomb	100	32	None	0	
OLD PARALIC DEPOSITS (Qop6)		125	Mohr-Coulomb	400	25	None	0	
POINT LOMA (Kp) with Bedding		125	Generalized Anisotropic			None	0	User Defined 1
POINT LOMA (Kp)		125	Mohr-Coulomb	1100	25	None	0	
BEDDING		125	Mohr-Coulomb	375	24	None	0	



Geotechnical Exploration, Inc.

Project	LOWENTHAL		SECTION F-F'
Analysis Description	SURFICIAL SLOPE STABILITY		
Drawn By	R.A.C.	Scale 1:350	Company G.E.I.
Date	5/29/2024, 11:22:59 AM		File Name JOB NO. 01-8018_S(F)_03.slim

SLIDEINTERPRET 6.039

SURFICIAL FAILURE

EQUATION 1

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

γ_t	γ_w	γ'	z_w
pcf	pcf	pcf	ft
125	62.4	62.6	3.28

SURFICIAL SLOPE STABILITY ANALYSIS IS BASED ON EQUATION (1) FOR THE CALCULATED VALUES. Reference: Abramson L.W., Lee T.S., Sharma S., Boyce G.M., 2002, Slope Stability and Stabilziation Methods, 2nd Edition, John Wiley and Sons, Inc.,

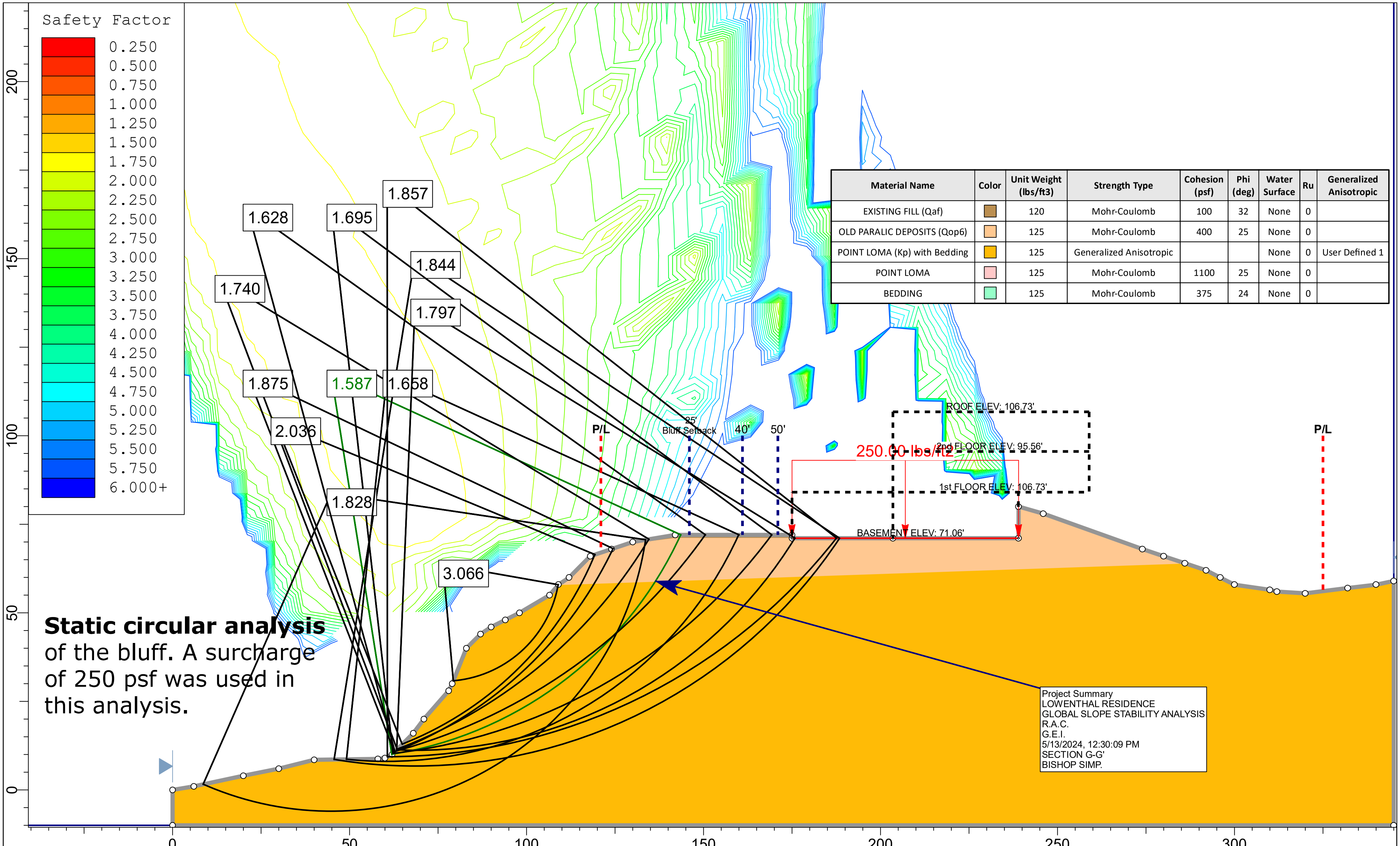
SECTION F-F'				
SOIL TYPE	**c (psf)**	**ϕ'(°)**	**α(°)**	**F.O.S.**
FILL (Q_{af})	100	32	21	1.544
OLD PARALIC DEPOSIT (Q_{op6})	400	25	11	6.410
POINT LOMA (K_p)	1100	25	27	7.091
POINT LOMA (K_p)	1100	25	53	5.758
POINT LOMA (K_p)	1100	25	66	7.324

1 meter = 3.28 feet

Special Publication 117A (2008, page 27): for infinite slope analysis, the minimum assumed depth of soil saturation is the smaller of either a depth of one meter or depth to firm bedrock.

α	The slope angle; (inclination angle) with respect to the horizontal plane
ϕ'	The effective friction angle of the soil
c'	The effective cohesion of the soil
γ_t	The total unit weight (Soil with moisture)
γ_w	The unit weight of the water
γ'	Submerged unit weight of the soil (Saturated unit weight - unit weight of water)
z_w	Vertical depth of the saturated soil
F.O.S.	Factor of Safety

The Factor of Safety values are **ABOVE** 1.50 and are adequate.



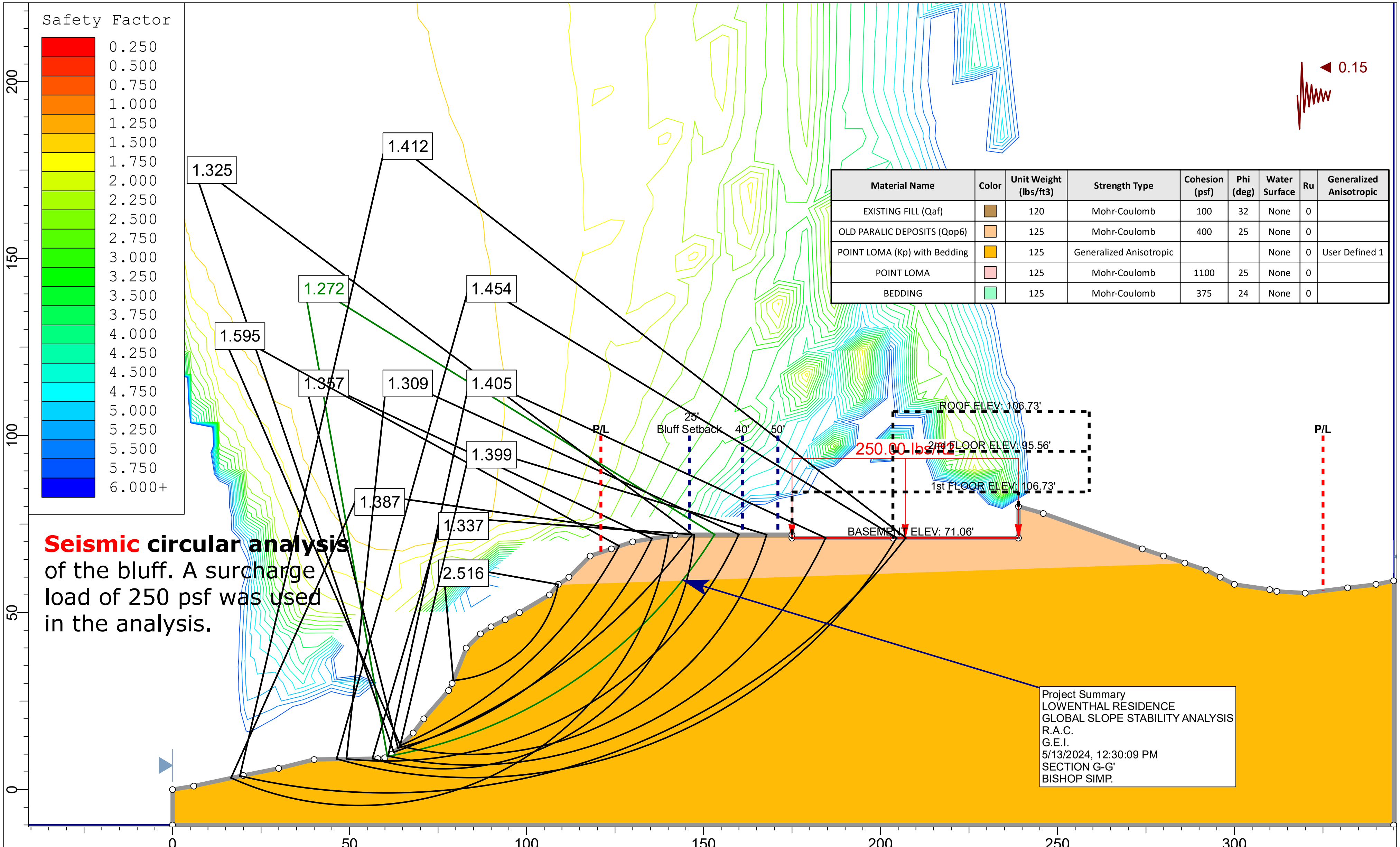
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)		120	Mohr-Coulomb	100	32	None	0	
OLD PARALIC DEPOSITS (Qop6)		125	Mohr-Coulomb	400	25	None	0	
POINT LOMA (Kp) with Bedding		125	Generalized Anisotropic			None	0	User Defined 1
POINT LOMA		125	Mohr-Coulomb	1100	25	None	0	
BEDDING		125	Mohr-Coulomb	375	24	None	0	

Static circular analysis of the bluff. A surcharge of 250 psf was used in this analysis.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	LOWENTHAL RESIDENCE — **SECTION G-G'**
Analysis Description	GLOBAL SLOPE STABILITY ANALYSIS
Drawn By	R.A.C.
Scale	1:450
Company	G.E.I.
Date	5/13/2024, 12:30:09 PM
File Name	JOB NO. 01-8018_S(G)_01.slim



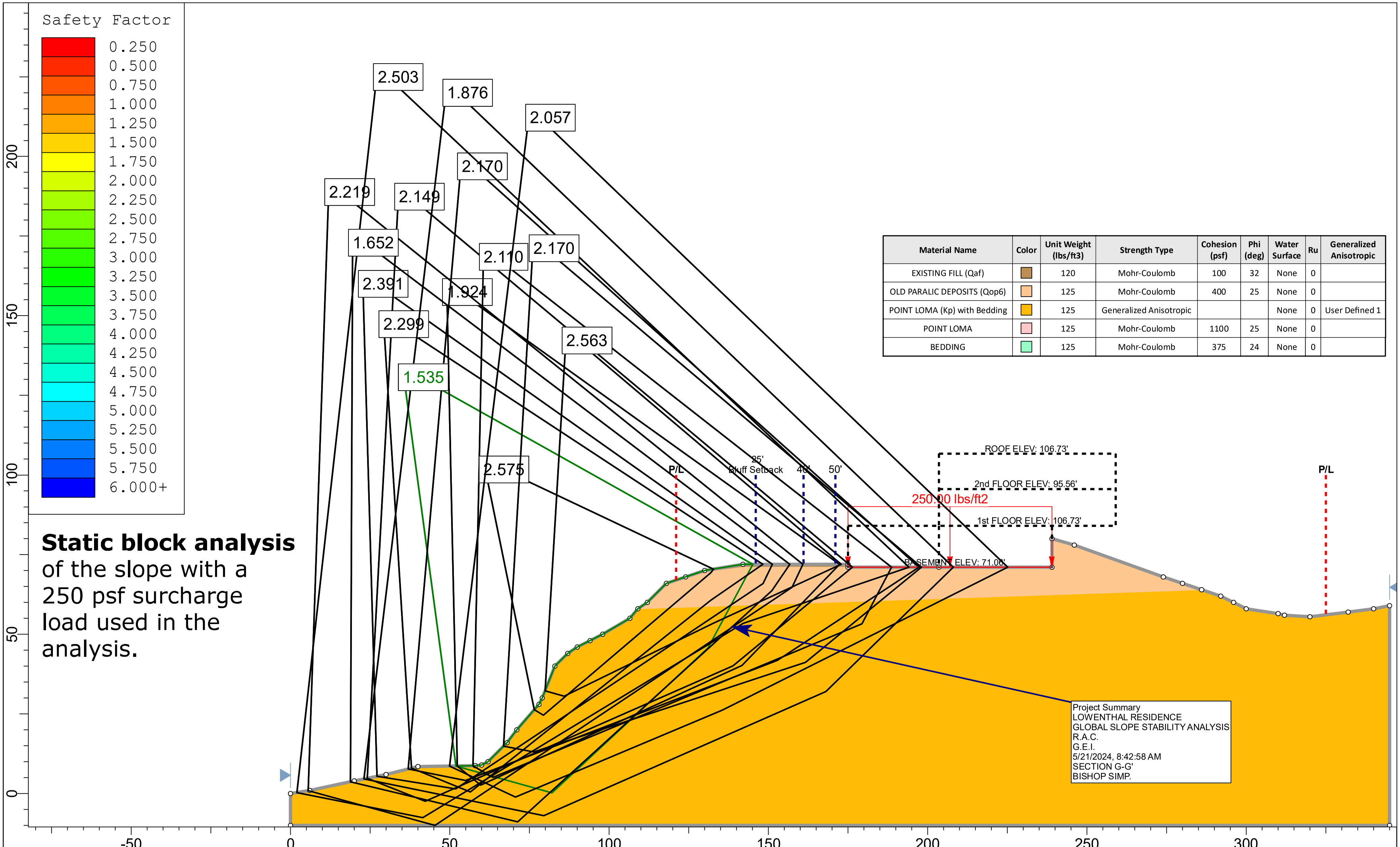
Seismic circular analysis of the bluff. A surcharge load of 250 psf was used in the analysis.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)		120	Mohr-Coulomb	100	32	None	0	
OLD PARALIC DEPOSITS (Qop6)		125	Mohr-Coulomb	400	25	None	0	
POINT LOMA (Kp) with Bedding		125	Generalized Anisotropic			None	0	User Defined 1
POINT LOMA		125	Mohr-Coulomb	1100	25	None	0	
BEDDING		125	Mohr-Coulomb	375	24	None	0	

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	LOWENTHAL RESIDENCE	**SECTION G-G'**	
Analysis Description	GLOBAL SLOPE STABILITY ANALYSIS		
Drawn By R.A.C.	*Scale* 1:450	*Company* G.E.I.	
Date 5/13/2024, 12:30:09 PM	*File Name* JOB NO. 01-8018_S(G)_01 w_0.15gSHAKE.slim		



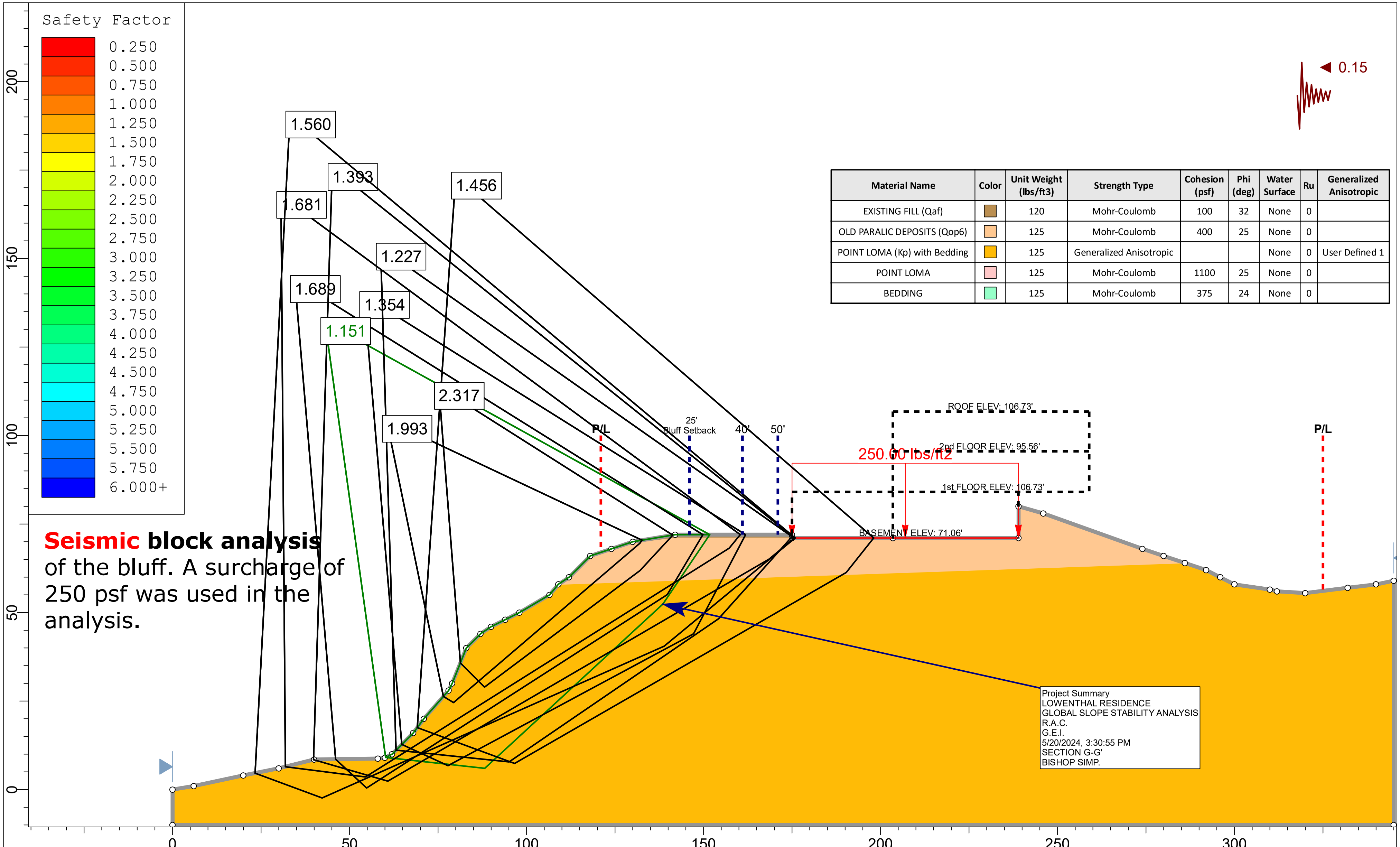
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)		120	Mohr-Coulomb	100	32	None	0	
OLD PARALIC DEPOSITS (Qop6)		125	Mohr-Coulomb	400	25	None	0	
POINT LOMA (Kp) with Bedding		125	Generalized Anisotropic			None	0	User Defined 1
POINT LOMA		125	Mohr-Coulomb	1100	25	None	0	
BEDDING		125	Mohr-Coulomb	375	24	None	0	

Static block analysis of the slope with a 250 psf surcharge load used in the analysis.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	LOWENTHAL RESIDENCE		SECTION G-G'
Analysis Description	GLOBAL SLOPE STABILITY ANALYSIS		
Drawn By	R.A.C.	Scale 1:500	Company G.E.I.
Date	5/21/2024, 8:42:58 AM	File Name	JOB NO. 01-8018_S(G)_02.slim



Seismic block analysis of the bluff. A surcharge of 250 psf was used in the analysis.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)		120	Mohr-Coulomb	100	32	None	0	
OLD PARALIC DEPOSITS (Qop6)		125	Mohr-Coulomb	400	25	None	0	
POINT LOMA (Kp) with Bedding		125	Generalized Anisotropic			None	0	User Defined 1
POINT LOMA		125	Mohr-Coulomb	1100	25	None	0	
BEDDING		125	Mohr-Coulomb	375	24	None	0	

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

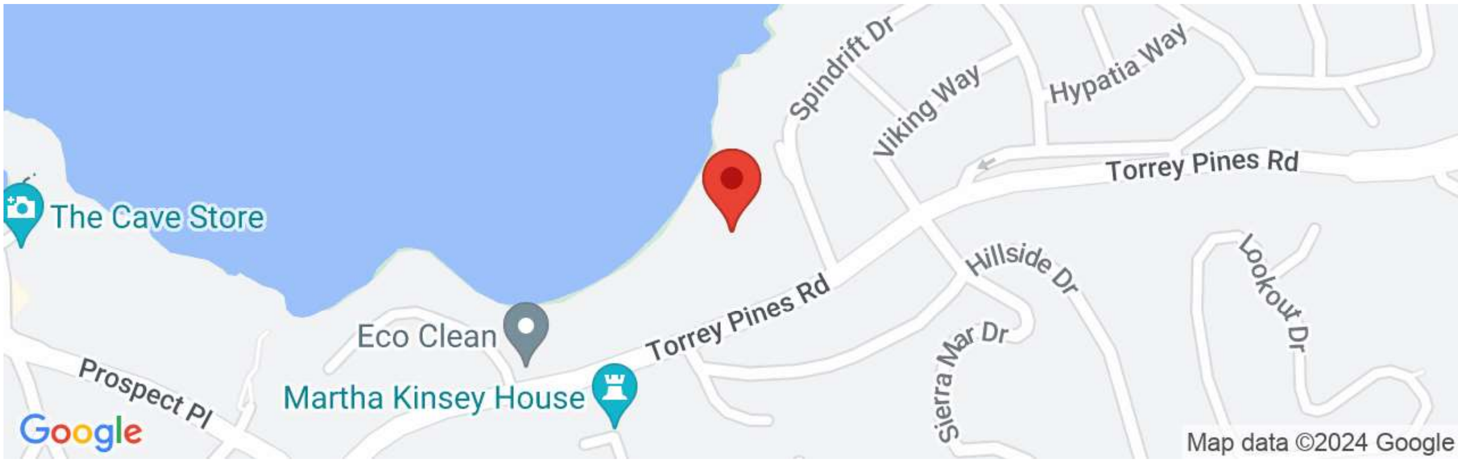
Project	LOWENTHAL RESIDENCE		**SECTION G-G'**
Analysis Description	GLOBAL SLOPE STABILITY ANALYSIS		
Drawn By R.A.C.	*Scale* 1:450	*Company* G.E.I.	
Date 5/20/2024, 3:30:55 PM		*File Name* JOB NO. 01-8018_S(G)_02 w_0.15gSHAKE.slim	

APPENDIX G

Lowenthal Residence
Job No. 01-8018

1720 Torrey Pines Road, La Jolla, CA 92037

Latitude, Longitude: 32.8493, -117.2635



Date	4/24/2024, 2:22:35 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value		Description
S_S	1.399		MCE_R ground motion. (for 0.2 second period)
S_1	0.49		MCE_R ground motion. (for 1.0s period)
S_{MS}	1.399		Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	0.887	Site-modified spectral acceleration value
S_{DS}	0.932		Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	0.591	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	1.809	Site amplification factor at 1.0 second
PGA	0.639		MCE_G peak ground acceleration
F_{PGA}	1.1		Site amplification factor at PGA
PGA_M	0.702		Site modified peak ground acceleration
T_L	8		Long-period transition period in seconds
SsRT	1.399		Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.614		Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.259		Factored deterministic acceleration value. (0.2 second)
S1RT	0.49		Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.552		Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.795		Factored deterministic acceleration value. (1.0 second)
PGAd	0.937		Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.639		Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.867		Mapped value of the risk coefficient at short periods
C_{R1}	0.886		Mapped value of the risk coefficient at a period of 1 s
C_V	1.38		Vertical coefficient

APPENDIX H

SLAB MOISTURE INFORMATION AND MOISTURE BARRIER MEMBRANES

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 E1745-09 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 302.2R-06 Guide to Concrete Floor and Slab Construction.

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 E1745-09 Class A requirements. Installation of vapor barriers should be in accordance with ASTM E1643-18a. The basis of design is 15-mil StegoWrap vapor barrier placed per the manufacturer's guidelines. Reef Industries Vapor Guard membrane has also been shown to achieve a permeance of less than 0.01 perms. We recommend that the slab be poured directly on the vapor barrier, which is placed directly on the prepared properly compacted smooth subgrade soil surface.

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.

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.

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.

APPENDIX I

COASTAL BLUFFS AND BEACHES GUIDELINES

The City of San Diego Municipal Code "*Coastal Bluffs and Beaches Guidelines*" (adopted September 28, 1999, amended June 6, 2000, per City Council Resolution R-293254-2) provides the following definitions, guidelines and diagrams to be utilized in the identification and evaluation of coastal bluffs. For the purposes of discussion, we provide the following excerpts as published in the 2001 San Diego Association of Geologist document "*Coastal Processes and Engineering Geology of San Diego County.*"

<u>Section I, Explanation of Definitions</u>

For each of the following terms, the definition is repeated (in italics) from Chapter 11, Article 3, Division 1, Land Development Terms, followed by additional information intended to clarify the definitions. The additional information provided is not part of the definition.

<u>A. Coastal Bluff</u>

Coastal bluff means an escarpment or steep face of rock, decomposed rock, sediment, or soil resulting from erosion, faulting, or folding of the land mass that has a vertical relief of 10 feet or more and is located in the coastal zone.

A coastal bluff is a naturally formed precipitous landform that generally has a gradient of at least 200 percent (1:2 slope) with a vertical elevation of at least 10 feet. See Diagram I-1 (not included). The gradient of a coastal bluff could be less than 200 percent but the vertical elevation must always be at least 10 feet. A coastal bluff is a form of environmentally sensitive lands that is included in the definition of steep hillsides. The coastal bluff includes the bluff face, which is all the area between the toe of the bluff and the bluff edge. Steep landforms meeting the criteria of coastal bluffs occur both inside and outside the Coastal Zone. These landforms and all other steep hillsides, both inside and outside the Coastal zone, are regulated by the steep hillside regulations of the Environmentally Sensitive Lands Regulations (Section 143.0142) and are subject to the Steep Hillside Guidelines…

<u>D. Coastal Bluff Edge</u>

Coastal Bluff Edge means the termination of the top of a sensitive coastal bluff where the downward gradient of the land surface begins to increase more or less continuously until it reaches the general gradient of the coastal bluff face.

The coastal bluff edge is the upper termination of a coastal bluff face where the downward gradient of the top of bluff face increases more or less continuously until it reaches the general gradient of the bluff face. When the top edge of the coastal bluff is rounded away from the bluff face as a result of erosional processes related to the presence of the bluff face, the coastal bluff edge shall be defined as that point at the top of bluff nearest the bluff face beyond which the downward gradient of the land surface increases more or less continuously until it reaches the general gradient of the bluff face. If evidence shows that the rounding is a result of geologic processes other than processes related to the presence of the bluff face, the location of the coastal bluff edge shall be determined through consideration of the available geologic data.

In a case where there is a step like feature at the top of the coastal bluff, the landward edge of the topmost riser shall be considered the coastal bluff edge.

The coastal bluff edge is a continuous line across the entire length of the coastal bluff on the premises from which all bluff setbacks shall be measured.

See Section III, Part A for details on determining the location of the coastal bluff edge for sensitive coastal bluffs.

E. Coastal Bluff Face

Coastal Bluff Face means that portion of a sensitive coastal bluff lying between the toe of the existing bluff and the coastal bluff edge.

The coastal bluff face is vertical or contains a relatively steep consistent gradient and may be rounded at the top, adjacent to the coastal bluff edge. When the bluff is rounded at the top as a result of erosional processes due to the presence of the bluff face, the bluff face shall include the rounded portion. The coastal bluff face of a sensitive coastal bluff (at least at the toe of the bluff) is typically subject to marine erosion. See Diagram I-4 (not included).

Generally, no development is permitted on the face of a sensitive coastal bluff, except as permitted in Section 143.0143(h) and (i) of the Environmentally Sensitive Lands Regulations.

Section II, Description of Regulations

The regulations for development proposed on a sensitive coastal bluff are located in Section 143.0143. The regulations for development proposed on a site containing a coastal beach are located in Section 143.0144. The following guidelines are intended to aid in the interpretation and implementation of pertinent development regulations in these sections. The numbers referenced for each development regulation refer to the Code section numbers of the Environmentally Sensitive Lands Regulations. The text provided for each regulation does not repeat the Code language but rather restates the regulation with more details and explanations…

C. 143.0143(f) Distance from Coastal Bluff Edge of Sensitive Coastal Bluffs

Development proposed on a sensitive coastal bluff, including primary and accessory structures, and grading, shall be located at least 40 feet landward from the coastal bluff edge, except as follows:

1. A distance of more than 40 feet from the coastal bluff edge may be required based on current geologic conditions.

2. Development may be located less than 40 feet but not less than 25 feet from the coastal bluff edge if there is evidence in a geology report that the site is stable enough to support the development at the proposed distance and if the development will neither be subject to nor contribute to significant geologic instability or require a shoreline or bluff erosion control device. In determining the stability of

the sensitive coastal bluff, consideration shall be given to the rate of bluff retreat to determine whether the proposed development will be impacted within a reasonable economic life-span, taken to be 75 years. If a development is approved with a less-than-40-foot distance to the coastal bluff edge, future erosion control measures are precluded. Air-placed concrete, retaining walls and seawalls will only be permitted when the principal structure, or public improvements not capable of being relocated, are in eminent danger. Less environmentally damaging alternatives that reduce risk and avoid the need to significantly alter the natural landforms of the beach and/or bluff shall be considered as feasible.

[NOTE: If a seawall (or other stabilization/erosion control measure) has been installed due to excessive erosion on a premises, that premises shall not qualify for a reduction of the required 40-foot distance to the coastal bluff edge. Since the instability of the coastal bluff would not be considered stable enough to support development within the 40-foot bluff edge setback.]

3. A distance of five feet from the coastal bluff edge may be granted for landscape features and accessory structures that are located at grade so that they are not elevated at the base or constructed with a raised floor and are capable of being relocated. Permitted features and structures include landscaping, paved walkways, at-grade decks, unenclosed patios, open shade structures, lighting standards, fences and walls, seating benches, and signs. A distance of five feet from the coastal bluff edge may not be granted for buildings, garages, carports, pools, spas, and raised decks with load bearing support structures.

4. Open fences may be permitted closer than 5 feet to the coastal bluff edge only if necessary to provide for public safety and to protect resource areas accessible from public right-of-way or on public parkland…

<u>Section III, Bluff Measurement Guidelines</u>

The following guidelines provide details on determining the location of the bluff edge for sensitive coastal bluffs and measuring the required bluff edge setback.

<u>A. Determination of Coastal Bluff Edge for Sensitive Coastal Bluffs</u>

1. Simple Bluff

The coastal bluff edge is a line across the sensitive coastal bluff at the seaward edge of the top of bluff. The line of the coastal bluff edge is formed by measuring the uppermost point of change in gradient at any location on the subject premises. See Diagram III-1 (not included).

7. Coastal Canyons

Where a site is bounded on at least one side by a coastal canyon (a large, established regional drainage course that traditionally accepts runoff from off-site), the coastal bluff edge is defined as the portion of the site which drains directly into the ocean. That portion of the site which drains first to the canyon (landward of the drainage divide) is not considered to be a sensitive coastal bluff. See Diagram III-7 (not included).