



REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION

**PROPOSED SINGLE-FAMILY RESIDENCE
AMALFI STREET
LA JOLLA, CALIFORNIA 92037**

PREPARED FOR

SONRIEN LLC

PREPARED BY

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CHRISTIAN WHEELER
ENGINEERING

April 9, 2026

Sonrien LLC

CWE 2260068.01

**Subject: Report of Preliminary Geotechnical Investigation
Proposed Single-Family Residence
Amalfi Street, La Jolla, California**

Ladies and Gentlemen:

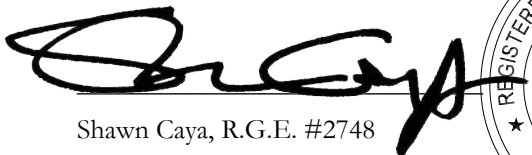
In accordance with your request, we have completed a preliminary geotechnical investigation for the subject project. We are presenting herein our findings and recommendations.

In general, no geotechnical conditions were encountered which would preclude the construction of the proposed residence provided the foundation and remedial grading recommendations discussed herein are followed. The main geotechnical and geologic conditions that will impact the proposed construction are the presence of potentially compressible near-surface soils. Based on these conditions, it is our opinion that the existing surficial soils should be overexcavated and replaced as properly compacted fill in order to support new fill and/or settlement-sensitive improvements. Additionally, the on-site soils are potentially expansive and may require select grading at the discretion of the owner. Specific remedial grading and foundation design criteria based on these conditions are provided in the attached report.

If you have any questions after reviewing this report, please do not hesitate to contact our office. This opportunity to be of professional service is sincerely appreciated.

Respectfully submitted,

CHRISTIAN WHEELER ENGINEERING



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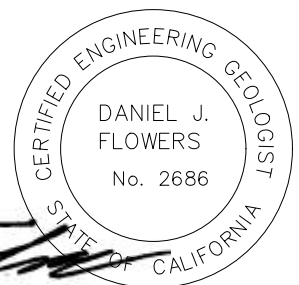


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REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION

PROPOSED SINGLE-FAMILY RESIDENCE

AMALFI STREET

LA JOLLA, CALIFORNIA

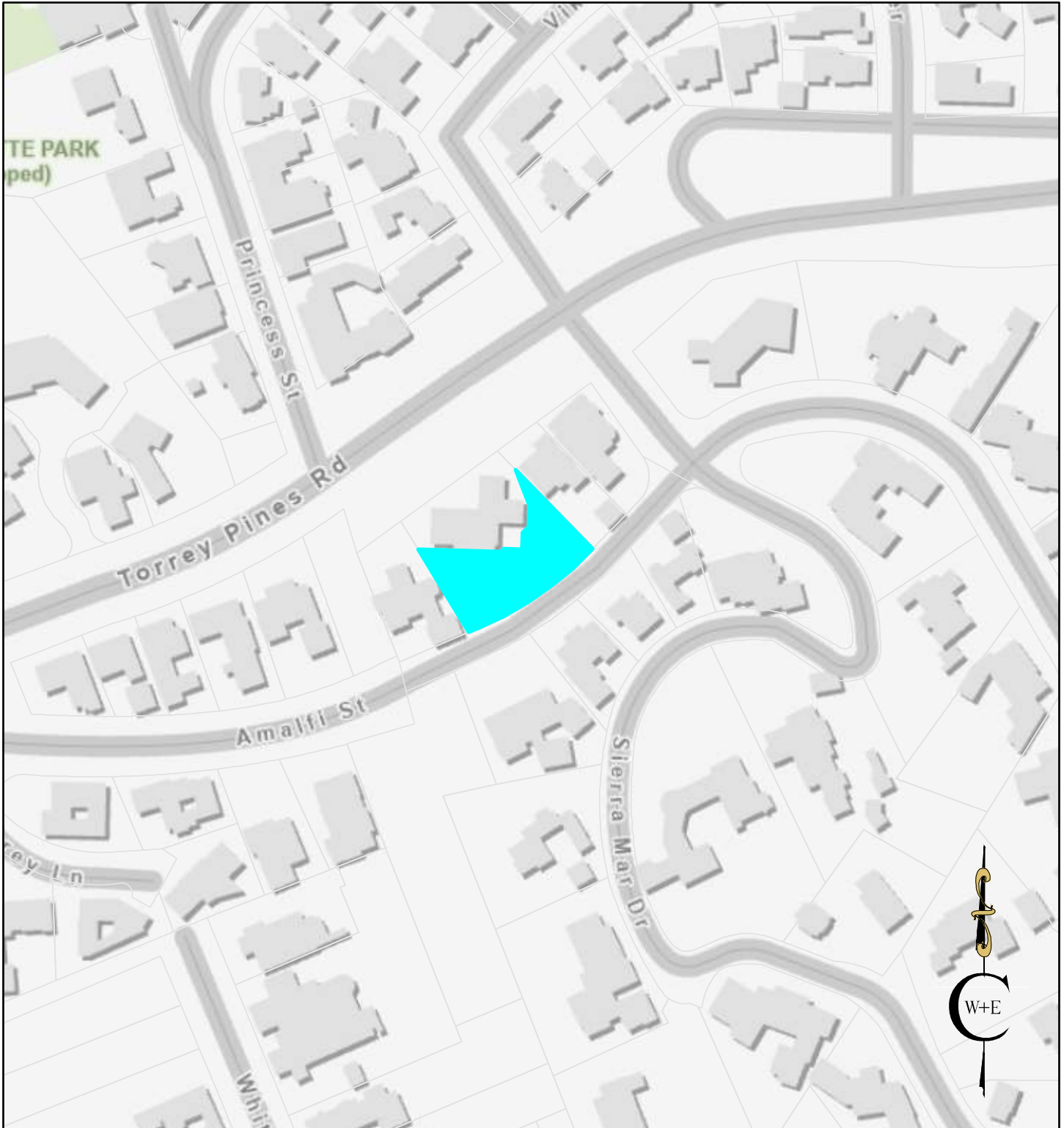
INTRODUCTION AND PROJECT DESCRIPTION

This report presents the results of a preliminary geotechnical investigation performed for a proposed single-family residence to be constructed north of Amalfi Street, in the La Jolla neighborhood within the city of San Diego, California. Figure Number 1, on the following page, presents a vicinity map showing the location of the project.

To assist in the preparation of this report, we were provided with a set of architectural plans prepared by Fragmento, dated August 2025. The proposed site plan included in the plan set was used as the base for our Geotechnical Maps, which is included herewith as Plate Number 1. We have also created a geologic cross-section A-A', included herewith as Plate Number 2.

We understand that it is proposed to develop the site with the construction of a two-story, single-family residence and associated improvements. The upper level will be constructed at or near the same elevation as the adjacent Amalfi Street. The lower level will be cut into the descending slope and daylight to a lower-level patio and pool. We anticipate that the proposed structure will be of wood-frame and masonry construction, and will be supported by standard shallow foundations with a concrete slab-on-grade floor. Grading to accommodate the proposed improvements is expected to consist of cuts and fills of up to about 5 feet and 10 feet, respectively, from the current grades. Building and site retaining walls are also expected. Stormwater management devices that incorporate infiltration are not expected.

This report has been prepared for the exclusive use of Sonrien LLC and its consultants for specific application to the project described herein. Should the project be modified, the conclusions and recommendations presented in this report should be reviewed by Christian Wheeler Engineering for conformance with our recommendations and to determine whether any additional subsurface evaluation,



NO SCALE

SITE VICINITY MAP

PROPOSED SINGLE-FAMILY RESIDENCE
AMALFI STREET - APN 350-161-1000
LA JOLLA, CALIFORNIA

DATE: APRIL 2026

REPORT NO.: 2260058.01

BY: SCC

FIGURE NO.: 1



CHRISTIAN WHEELER
ENGINEERING

laboratory testing and/or recommendations are necessary. Our professional services have been performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties, expressed or implied.

PROJECT SCOPE

Our geotechnical investigation consisted of subsurface exploration, surface reconnaissance, laboratory testing, and review of relevant geologic literature. More specifically, our intent was to provide the items listed below.

- Drill three small-diameter borings (maximum depth of 20 feet) to explore the subsurface conditions of the site and to obtain samples for laboratory testing.
- Backfill the boring holes using a grout or a grout/bentonite mix as required by the County of San Diego Department of Environmental Health.
- Evaluate, by laboratory tests and our experience with similar soil types, the engineering properties of the various soil strata that may influence the proposed construction, including bearing capacities, expansive characteristics, and settlement potential.
- Describe the general geology at the site, including possible geologic hazards that could have an effect on the proposed construction, and provide the seismic design parameters in accordance with the 2025 edition of the California Building Code based on the default soil site class.
- Discuss potential construction difficulties that may be encountered due to soil conditions, groundwater, or geologic hazards, and provide geotechnical recommendations to mitigate identified construction difficulties.
- Provide site preparation and grading recommendations for the anticipated work, as necessary.
- Provide recommendations for shored and unshored temporary cut slopes.
- Provide foundation recommendations for the type of construction anticipated and develop soil engineering design criteria for the recommended foundation designs.
- Provide earth retaining wall design recommendations.
- Prepare this report, which includes, in addition to our conclusions and recommendations, a plot plan showing the aerial extent of the geological units and the locations of our test pits and boring, and exploration logs.

FINDINGS

SITE DESCRIPTION

The subject site is an undeveloped residential lot located on Amalfi Street in the La Jolla neighborhood of the city of San Diego, California. The irregular-shaped property, which is identified as Assessor's Parcel Number 350-161-11, is located north of Amali Street, just west of the intersection with Sierra Mar Drive. The lot is bordered by Amalfi Street to the south and developed residential properties to the remaining sides. The site currently supports a wooden deck, brick walkways, and landscape vegetation. Topographically, the site slopes moderately downward towards the north-northwest, with elevations ranging from approximately 130 feet along Amalfi Street to 105 feet near the neighboring lot to the north.

GENERAL GEOLOGY AND SUBSURFACE CONDITIONS

GEOLOGIC SETTING AND SOIL DESCRIPTION: The subject site is located in the Coastal Plains Physiographic Province of San Diego County. Based upon the review of geologic literature and our subsurface explorations, it was determined that the project area is underlain by a layer of artificial fill and/or slopewash overlying Pleistocene-age old paralic deposits and Cretaceous-age sedimentary deposits known as the Point Loma Formation. These materials are described below in order of increasing age:

ARTIFICIAL FILL (Qaf): An approximately 4-foot-thick layer of artificial fill was encountered in our boring B-3. The fill soils appear to be associated with the construction of the Amalfi Street south of the property and may be thicker in areas of the site not explored. The encountered fill soil generally consisted of grayish-brown, moist, soft to stiff, sandy lean clay (CL) with roots. The fill soils were judged to have a medium to high expansive potential ($50 < EI \leq 130$) and a moderate potential for consolidation in their present condition.

SLOPEWASH (Qsw): An approximately 2- to 5-foot-thick layer of slopewash was encountered below the fill layer in our boring B-3 and at the ground surface in our borings B-1 and B-2. The slopewash may be thicker in areas of the site not explored. The slopewash generally consists of grayish-brown, moist, stiff, porous, lean clay (CL). The slopewash soil was judged to have a medium to high expansive potential ($50 < EI \leq 130$) and a moderate potential for consolidation in its present condition.

OLD PARALIC DEPOSITS (Qop6): Pleistocene-age old paralic deposits were found to underlie the artificial fill and slopewash within each of our borings and are expected underlie the entire site. As

encountered in our borings, the old paralic deposits were encountered at depths ranging from 2 to 7 feet in our explorations. The contact with the old paralic deposit may be deeper in areas of the site not investigated. The old paralic deposits were observed to generally consist of olive-brown, moist, very stiff to hard, sandy clay (CL). At a depth of about 18 feet within boring B-1 a bed of poorly graded sand with silt (SP-SW) and gravels was encountered. This bed was interpreted as a possible basal deposition bed overlying the Point Loma Formation below. The old paralic deposits were judged to have a medium to high expansive potential ($50 < EI \leq 130$) and a low potential for consolidation in its present condition.

POINT LOMA FORMATION (Kp): Cretaceous-age sedimentary deposits of the Point Loma Formation were found to underlie the old paralic deposits within our boring B-2 and are expected underlie the entire old paralic deposits at depth. The Point Loma Formation was encountered at a depth of approximately 12 feet below grade in boring B-2, but was not encountered in borings B-1 and B-3, which were drilled to approximately 20 and 14 feet, respectively. A claystone bed of the Point Loma Formation was encountered in our boring which generally consist of olive-gray, moist, hard, sandy clay (CL). Form our experience the Point Loma Formation typically consists of a yellowish-brown sandstone with olive-gray claystone beds. The formational deposits were judged to have a medium to high expansive potential ($50 < EI \leq 130$) and a very low potential for consolidation in their present condition.

GEOLOGIC STRUCTURE: Based on our review of the referenced geologic map of the area (Kennedy and Tan, 2008), the bedding of the Cretaceous-age sedimentary deposits (Point Loma Formation) that underlie the site dip approximately 20° to 30° to the southwest within the vicinity of the site. Such bedding is considered favorable with regards to the stability of the northerly facing slopes at the site. Above the Point Loma Formation, the bedding of the old paralic deposits is anticipated to be generally massive.

GROUNDWATER: Groundwater was not encountered in our subsurface explorations. It should be noted that variations in subsurface water (including perched water zones and seepage) may result from fluctuations in the ground surface topography, subsurface stratification, precipitation, irrigation, and other factors that may not have been evident at the time of the evaluation. It should also be recognized that minor groundwater seepage problems might occur after development of a site even where none were present before development. These are usually minor phenomena and are often the result of an alteration in drainage patterns and/or an increase in irrigation water. It is further our opinion that these problems can be most effectively corrected on an individual basis if and when they occur.

TECTONIC SETTING: Much of Southern California, including the San Diego County area, is characterized by a series of Quaternary-age fault zones that consist of several individual, en echelon faults that generally strike in a northerly and northwesterly direction. According to the 2018 revision of '*Special Publication 42, Earthquake Fault Zones*' by the California Geologic Survey (CGS), a three-tier fault classification system is used:

- '*Holocene-active faults*'; Faults with displacement within the Holocene epoch (the last 11,700 years)
- '*Pre-Holocene faults*'; Faults with displacement with the Pleistocene epoch (between 11,700 to 2.6 million years ago), but not within the Holocene epoch. Previous terminology by the CGS referred to these faults as '*potentially active*', however this terminology is being phased out.
- '*Age-undetermined faults*'; Faults with displacement unconstrained by dating methods or limitations in stratigraphic resolution.

Faults older than Quaternary-age are not specifically defined in Special Publication 42, however, it is generally accepted that faults showing no movement during the Quaternary period (up to 2.6 million years) may be considered to be '*inactive*'.

Projects near *Holocene-active faults* are regulated by the Alquist-Priolo Earthquake Fault Zoning Act of 1972. The Act prohibits the development of structures for human occupancy across the surface trace of an active fault in California, with certain exceptions. Faults determined by the CGS as *sufficiently active* and *well-defined* are identified on Alquist-Priolo regulatory maps as *Active Fault Traces* and bounded by buffers called *Earthquake Fault Zones*. Sites within these zones are required to undergo fault hazard studies as part of the geotechnical investigation. Review of the local Alquist-Priolo regulatory maps indicates that the subject site is not underlain by a known active fault and is not within an active Earthquake Fault Zone.

A review of available geologic maps including the CGS Fault Activity Map of California (2010), and the USGS Quaternary Fault and Fold Database (2017) indicates that the nearest active fault zone is the Newport-Inglewood-Rose Canyon Fault Zone located approximately 500 feet to the northeast of the site. Other active fault zones in the region that could possibly affect the site include the Coronado Bank, San Diego Trough and San Clemente Fault Zones to the west, and the Elsinore, San Jacinto, and San Andreas Fault Zones to the east and northeast. A portion of these fault zones are summarized in Table I below.

TABLE I: PROXIMAL FAULT ZONES

Fault Zone	Distance
Newport-Inglewood-Rose Canyon	500 feet
Coronado Bank	13 mi
San Diego Trough	23 mi
Elsinore	42 mi
San Clemente	48 mi
San Jacinto	64 mi
San Andreas	87 mi

GEOLOGIC HAZARDS

CITY OF SAN DIEGO SEISMIC SAFETY STUDY: As part of our services, we have reviewed the City of San Diego Seismic Safety Study. This study is the result of a comprehensive investigation of the City that rates areas according to geological risk potential (nominal, low, moderate, and high) and identifies potential geotechnical hazards and/or describes geomorphic conditions.

According to the San Diego Seismic Safety Study Map No. 29, the site is located in Geologic Hazard Category 27. Geologic Hazard Category 27 is indicative of areas of areas that are underlain by “Slide Prone Formations” other than the Ardath Shale and Friars Formations (in the case of the subject property, the Point Loma Formation). A review of current geologic literature indicates that the local geologic structure of the bedding within the Point Loma Formation dips into slope in the vicinity of the site.

SEISMIC HAZARD: A likely geologic hazard to affect the site is ground shaking as a result of movement along one of the major active fault zones mentioned in the “Tectonic Setting” section of this report. Seismic design parameters were determined in accordance with the applicable sections of *ASCE/SEI 7-22 Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. For the subject site, the results of our subsurface explorations and experience within the area indicate that the default site conditions are appropriate to determine soil Site Class at the subject site. Table II presents the seismic design parameters using the coordinates 32.848665°, -117.261875°.

TABLE II: ASCE 7-22 – SEISMIC DESIGN PARAMETERS

Seismic Design Parameter	Recommended Value
Soil Site Class	Default
MCE_R Acceleration for Short Periods (0.2 sec), S_s	1.55 g
MCE_R Acceleration for 1.0 Sec Periods (1.0 sec), S_1	0.49 g
S_{MS} = MCE_R Spectral Response at 0.2 sec.	1.64 g
S_{M1} = MCE_R Spectral Response at 1.0 sec.	1.16 g
S_{DS} = Design Spectral Response at 0.2 sec. = $2/3(S_{MS})$	1.09 g
S_{D1} = Design Spectral Response at 1.0 sec. = $2/3(S_{M1})$	0.77 g
Seismic Design Category	D
Mapped Long-Period Transition Period, T_L	8 sec
PGA_M per Section 11.8.3 of ASCE 7	0.70 g

LANDSLIDE POTENTIAL AND SLOPE STABILITY: As part of this investigation, we reviewed the publication, “Landslide Hazards in the Southern Part of the San Diego Metropolitan Area” by Tan, 1995. This reference is a comprehensive study that classifies San Diego County into areas of relative landslide susceptibility. According to this publication, the site is located in Relative Landslide Susceptibility Area 3-1. Area 3-1 is considered to be “generally susceptible” to landsliding. Natural slopes within the 3-1 classification are considered at or near their stability limits due to their steep inclinations and can be expected to fail locally when adversely modified. Sites within this classification are located outside the boundaries of known landslides.

The subject site is underlain by massively bedded old paralic deposits overlying competent materials of the Point Loma Formation. No evidence of landsliding or deep-seated slope failures were noted during our reconnaissance of the site or during review of the referenced geotechnical literature. In addition, bedding attitudes taken in the vicinity of the site (Kennedy and Tan, 2008) indicate favorable bedding (into the slope). Therefore, the potential for deep-seated landsliding is considered a low in our opinion. It is further our opinion that the proposed construction will not affect the stability of neighboring properties or right-of-way if the recommendations presented herein are implemented. However, care should be taken to ensure the proper drainage of all surface runoff away from the slope areas and along slope faces. Saturation of the slopes caused by excessive or improperly channeled runoff could detrimentally affect the surficial stability of the sloping site. Irrigation on and adjacent to the slopes should be carefully monitored to make sure that only the minimum amount necessary to sustain plant life is used.

SURFACE RUPTURE: The site is not within an Alquist-Priolo Earthquake Fault Zone, and no faults are mapped as bisecting the site. It is our professional opinion that no active or potentially faults are present at the subject site and that the probability of fault rupture at the site is negligible.

FLOODING: As delineated on the Flood Insurance Rate Map (FIRM), prepared by the Federal Emergency Management Agency, the site is in Zone X which is considered to be an “area of minimal flood hazard.” Areas of minimal flood hazards are located outside of the boundaries of both the 100-year and 500-year flood zones.

LIQUEFACTION: The near-surface soils encountered at the site are not considered susceptible to liquefaction due to such factors as depth to the groundwater table, soil density and grain-size distribution.

TSUNAMIS: Tsunamis are great sea waves produced by a submarine earthquake or volcanic eruption. According to the Tsunami Inundation Map for Emergency Planning (CGS, 2022) the site is located outside of a tsunami inundation area. Due to the site’s elevation and location, the site is not subject to risk from tsunamis.

SEICHES: Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays or reservoirs. Due to the site’s location, it should not be affected by seiches.

CONCLUSIONS

In general, no geotechnical conditions were encountered which would preclude the construction of the proposed residence provided the foundation and remedial grading recommendations discussed herein are followed. The main geotechnical and geologic conditions that will impact the proposed construction are the presence of surficial soils (artificial fill and slopewash) that are compressible and considered unsuitable in their present condition to support the proposed improvements. Based on this condition, it is our opinion that existing surficial soil should be overexcavated and replaced with compacted structural fill in areas to support fill and/or settlement-sensitive improvements.

Portions of the existing soils are known to have a medium potential for expansion. The design of building foundations will need to incorporate this condition. Routine maintenance and possible replacement can be expected for concrete flatwork such as sidewalks, patios, and driveways as well as other lightweight exterior improvements underlain by expansive soil. The potential for expansion can be largely mitigated through grading techniques and by maintaining proper drainage and limiting irrigation to only the amount necessary to sustain plant life. Additional mitigation can be achieved by placing a two-foot-thick mat of sandy soils with an expansion index of 50 or less below the improvements; however, the decision to do so is an economic decision that will need to be made by the owner. It may be more cost effective for this project to provide occasional maintenance, repair and/or replacement of light exterior improvements if necessary.

No geologic hazards of sufficient magnitude to preclude continued residential use and redevelopment of the site as we presently contemplate it are known to exist. In our professional opinion and to the best of our knowledge, the site is suitable for the existing and proposed improvements.

RECOMMENDATIONS

GRADING AND EARTHWORK

GENERAL: All grading should conform to the guidelines presented in Appendix J of the California Building Code and the minimum requirements of the City of San Diego, except where specifically superseded in the text of this report. Prior to grading, a representative of Christian Wheeler Engineering should be present at the pre-construction meeting to provide additional grading guidelines, if necessary, and to review the earthwork schedule.

OBSERVATION OF GRADING: Continuous observation by the Geotechnical Consultant is essential during the grading operations to confirm conditions anticipated by our investigation, to allow adjustments in design criteria to reflect actual field conditions exposed, and to determine that the grading proceeds in general accordance with the recommendations contained herein.

CLEARING AND GRUBBING: Site grading should begin with the removal of all existing structures, asphalt and concrete pavement, trees, light poles and underground utilities in the portions of site that will be graded and/or will receive new improvements. The resulting materials should be disposed of off-site. Any abandoned underground pipes found during the grading operation should be removed and the resulting depressions backfilled with uniformly compacted fill material.

EXCAVATION CHARACTERISTICS: We expect that planned excavations and excavations for the removal of unsuitable soils can be made using normal grading equipment. Deeper excavations extending into the old paralic deposits or Point Loma Formation should be expected to encounter gravel, cobble, concretions, and/or strongly cemented zones.

REMEDIAL GRADING: In the areas to support the residence, site retaining walls, and swimming pool, we recommend that the existing fill and slopewash be overexcavated to the contact with competent old paralic deposits or to a depth of 5 feet below the bottom of foundations, whichever depth is greater. The removals should include all areas to receive new fill and extend at least 5 feet outside the planned foundation locations, or to the property line, whichever distance is less. In areas to support surface improvements such as pavements or

exterior flatwork, the existing soil should be overexcavated to a depth of at least 2 feet below the existing grade. Such removals should generally extend at least 2 feet outside the limits of the surface improvement.

Once exposed, the bottom of the overexcavation should be observed by a representative of Christian Wheeler Engineering. Depending on the conditions encountered, deeper removals may be recommended by our field personnel. Prior to placing fills, removal areas should be processed as described below in the “Processing of Removal Bottom” section of this report. After the bottom is processed, the previously removed material can be replaced as structural fill in accordance with the “Fill Soil and Method of Compaction” section below. See also the option for “Select Grading” in the following section.

Excavations for remedial grading near the existing property line may need to be performed in slots. We expect that the width of the slots can be 8 feet or less parallel to the property line; however, this could be revised based upon the field conditions exposed during excavation operations. Slots for remedial grading must be entirely backfilled before excavating adjacent slots. We recommend that our firm be contacted to have an engineering geologist observe the temporary slot excavations during grading to ascertain that no unforeseen adverse geotechnical conditions exist such as slope instability, adverse bedding, or seepage. If adverse conditions are identified, additional recommendations will be provided, as necessary. No surcharge loads such as soil or equipment stockpiles, foundations, etc. should be allowed within a distance from the top of temporary slot excavations equal to half the slope height. If remedial grading cannot encompass property line improvements the foundations for the improvements will need to be deepened into formation soils. This will be determined once the as-graded site conditions are completed.

SELECT GRADING: We expect that clayey soil with a medium expansion index will be encountered during the proposed grading. If it is desired to reduce the potential for heave damage to exterior improvements, the areas to support said improvements should be capped with an at least two-foot-thick layer of non-detrimentally expansive ($E.I. \leq 50$) material. This cap should extend at least two feet outside the improvement area. The client should determine if this select grading operation will be performed prior the start of the final mass grading operations.

PROCESSING OF REMOVAL BOTTOM: Prior to placing any new fill soils in removal areas that have been observed by our field personnel, the exposed soils should be scarified to a depth of at least 6 inches, moisture-conditioned, and compacted in accordance with the following sections. Where the existing ground has a slope of 5:1 (horizontal to vertical) or steeper, it should be benched into as the fill extends upward. The benching should remove all surficial soils, as noted in the previous section, and should create level areas on which to place the fill material.

STABILIZATION: If soft, pumping, or otherwise unsuitable soils are encountered that cannot be properly compacted, it will be necessary to remove the unstable soil to a competent stratum and replace it with soil that is suitable for compaction. Alternatively, wet soil can be allowed to dry back to a moisture content that allows proper compaction. Other methods of stabilization such as geosynthetic reinforcement, rock blankets, or chemical admixture can be discussed during construction upon request.

FILL SOIL AND METHOD OF COMPACTION: Fill and backfill soil should be thoroughly mixed and placed at a moisture content at least 2 percent above optimum moisture content, in loose lifts 6 to 8 inches thick, with each lift compacted by mechanical means. Fills should consist of approved earth material, free of trash or debris, roots, vegetation, or other materials determined to be unsuitable by our field personnel. Fill material should be free of rocks or lumps of soil in excess of 3 inches in maximum dimension.

SUBGRADE PREPARATION: Subgrade is considered to be the upper 12 inches of soil in areas to support surface improvements such as vehicular pavements other roadway structures, flatwork, curbs and gutters, driveways, or sidewalks. Preparation of subgrade should be performed just prior to the placement of subbase, aggregate base, or the surface improvement, and should not be considered to be completed as part of the mass grading requirements or operation. The preparation of subgrade should result in a uniform soil having a moisture content that is minus 1 percent of optimum or wetter just prior to compaction. Achieving this condition will likely require the contractor to scarify, overexcavate, or otherwise loosen the subgrade soil and perform moisture-conditioning by adding water or allowing the existing material to dry. The moisture-conditioned material should be thoroughly mixed and compacted. Proof rolling with a fully loaded water truck may be requested in order to verify that a uniform, stable subgrade has been achieved. Areas that exhibit rutting, pumping, yielding, and/or low compaction should be stabilized as discussed above.

COMPACTION REQUIREMENTS: All structural fill placed at the site should be compacted to a relative compaction of at least 90 percent of its maximum dry density as determined by ASTM Laboratory Test D1557. In areas to support vehicular pavements, the upper 12 inches of subgrade should be compacted to at least 95 percent of the material's maximum dry density.

IMPORTED FILL MATERIAL: Soils to be imported to the site should be evaluated and approved by the Geotechnical Consultant prior to being imported. At least five working days-notice of a potential import source should be given to the Geotechnical Consultant so that appropriate testing can be accomplished. The type of material considered most desirable for import is granular material containing some silt or clay binder, which has an Expansion Index of less than 50. Less than 25 percent of the material should be larger than the

Standard #4 sieve, and less than 25 percent finer than the Standard # 200 sieve. Soils not meeting these criteria should not be used for structural fill or backfill.

TEMPORARY SLOPES: The contractor is solely responsible for designing and constructing stable, temporary excavations and will need to shore, slope, or bench the sides of trench excavations as required to maintain the stability of the excavation sides. The contractor's "competent person", as defined in the OSHA Construction Standards for Excavations, 29 CFR, Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety process. We anticipate that the existing on-site soils will consist of Type C material. Our firm should be contacted to observe all temporary cut slopes during grading to ascertain that no unforeseen adverse conditions exist. No surcharge loads such as foundation loads, or soil or equipment stockpiles, vehicles, etc. should be allowed within a distance from the top of temporary slopes equal to half the slope height.

SURFACE DRAINAGE: The ground around the proposed structures should be graded so that surface water flows rapidly away from the structure without ponding. In general, we recommend that the ground adjacent to structure slope away at a gradient of at least 5 percent for a minimum distance of 10 feet. If the minimum distance of 10 feet cannot be achieved, an alternative method of drainage runoff away from the building at the termination of the 5 percent slope will need to be used. Swales and impervious surfaces that are located within 10 feet of the building should have a minimum slope of 2 percent. Rain gutters with downspouts that discharge runoff away from the structure into controlled drainage devices are also recommended.

GRADING PLAN REVIEW: The final grading plans should be submitted to this office for review in order to ascertain that the geotechnical recommendations remain applicable to the final plan and that no additional recommendations are needed due to changes in the anticipated development. Our firm should be notified of changes to the proposed project that could necessitate revisions of or additions to the information contained herein.

TEMPORARY SHORING

GENERAL: Where it is not possible to construct temporary cut slopes in accordance with the previously recommended criteria, it will be necessary to use temporary shoring to support the proposed excavations. For shoring systems, we considered the use of cantilevered soldier pile walls and soldier pile walls using tieback anchors or internal bracing (rakers). We are including herein recommendations for cantilevered walls. We recommend that a specialty contractor with experience in shoring and bracing provide the shoring recommendations and plans. It is recommended that a "survey" be made of adjacent properties and

structures prior to the start of grading and excavation in order to establish the existing condition of existing neighboring structures and to reduce the possibility of potential damage claims as a result of site grading.

DRILLING CHARACTERISTICS: It is anticipated that the proposed soldier piles may be drilled using properly sized, conventional heavy-duty drilling equipment in good working condition; however, the Point Loma Formation is anticipated to contain well-cemented concretions which will likely require the use of a core barrel.

SHORING DESIGN AND LATERAL PRESSURES: For design of cantilevered shoring, a triangular distribution of lateral earth pressure may be used. It may be assumed that retained soils having a level surface behind the cantilevered shoring will exert a lateral pressure equal to that developed by a fluid with a density of 35 pounds per cubic foot (pcf). For 2:1 (H:V) sloping backfill the equivalent fluid pressure should be increased to 55 pcf.

DESIGN OF SOLDIER PILES: Soldier piles should be spaced no closer than two diameters on center. The ultimate lateral bearing value (passive value) of the soils below the level of excavation may be assumed to be 400 pounds per square foot per foot of depth from the excavated surface, up to a maximum of 6,000 pounds per square foot. The lateral bearing can be applied over a horizontal distance equal to twice the pile diameter. To develop the full lateral value, provisions should be made to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavations should be of sufficient strength to adequately transfer the imposed loads to the surrounding soils.

LAGGING: Continuous lagging will be required between the soldier piles. The soldier piles and anchors should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will likely be somewhat less due to arching in the soils. We recommend that the lagging be designed for a semi-circular distribution of earth pressure where the maximum pressure is 400 pounds per square foot at the mid-point between soldier piles and zero at the soldier piles.

Lagging should keep pace with excavation. The excavation should not be advanced deeper than 3 feet below the bottom of lagging at any time. These unlagged gaps of up to 3 feet should only be allowed to stand for short periods of time in order to decrease the probability of soil instability and should never be unsupported overnight. Backfilling should be conducted, when necessary, between the back of lagging and excavation sidewalls to reduce sloughing in this zone and all voids should be filled by the end of each day.

DEFLECTIONS: We recommend from a geotechnical standpoint that the deflection at the top of the shoring not exceed about one inch. If greater deflection occurs during construction, additional bracing may be necessary. If desired to reduce the deflection of the shoring, a greater lateral earth pressure could be used in the shoring design.

MONITORING: Some means of monitoring the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of the soldier piles approximately every 50 lineal feet. We will be pleased to discuss this further with the design consultants and the contractor when the design of the shoring system has been finalized.

SHALLOW FOUNDATIONS

GENERAL: It is our opinion that the proposed home may be supported by conventional continuous and isolated spread footings. The following recommendations are considered the minimum based on the anticipated soil conditions anticipated after the recommendations contained in this report are implemented and are not intended to be in lieu of structural considerations. All foundations should be designed by a qualified structural engineer.

SETTLEMENT CRITERIA: Provided the recommendations presented in this report are followed, the anticipated total and differential foundation settlement is expected to be less than about 1 inch and 1 inch over 40 feet, respectively. It should be recognized that minor cracks normally occur in concrete slabs and foundations due to shrinkage during curing or redistribution of stresses, therefore some cracks should be anticipated. Such cracks are not necessarily an indication of excessive vertical movements.

MINIMUM DIMENSIONS: New spread footings supporting the planned structure should be embedded at least 24 inches below the lowest adjacent grade. Continuous and isolated footings should have minimum widths of 12 and 24 inches, respectively.

ALLOWABLE BEARING PRESSURE: Footings with the minimum dimensions noted above may be designed based on the values presented in Table III. Foundations should be designed to support imposed vertical and lateral loads without exceeding the strength of the soil and to support sustained loads without exceeding the settlement criteria noted above. Vertical loads include combinations of dead (D) and live (L) loads while lateral loads are those combinations that also include transient wind (W) or seismic forces (E). Sustained loads include the dead load and the portion of live load that is considered to be permanent or of

long-term duration such that it contributes to settlement. For typical buildings, the sustained load is commonly taken as the dead load plus 50 percent of the live load.

TABLE III: ALLOWABLE BEARING PRESSURE

Controlling Soil Condition	ASD Load Combinations	Allowable Bearing Pressure	Minimum Factor of Safety
Strength – vertical loads	D + L	5,000 psf ¹	3
Strength – vertical plus lateral loads	D + L + (E or W)	4,000 psf ²	3
Settlement – sustained load	D + 0.5L	2,500 psf	2

1. Can be increased by 500 psf and 300 psf for each additional foot of depth and width, respectively, to a maximum of 10,000 psf.
2. Can be increased by 300 psf and 150 psf for each additional foot of depth and width, respectively, to a maximum of 10,000 psf.

FOOTING REINFORCING: Reinforcement requirements for foundations should be provided by a structural engineer. However, based on the anticipated soil conditions, we recommend that the minimum reinforcing for continuous footings consist of at least two No. 5 bars positioned near the bottom of the footing and at least two No. 5 bars positioned near the top of the footing.

LATERAL LOAD RESISTANCE: Lateral loads against foundations may be resisted by friction between the bottom of the footing and the supporting soil, and by the passive pressure against the footing. The coefficient of friction between concrete and soil may be considered to be 0.30. The passive resistance may be considered to be equal to an equivalent fluid weight of 350 pounds per cubic foot. This assumes the footings are poured tight against undisturbed soil. If a combination of the passive pressure and friction is used, the friction value should be reduced by one-third.

EXPANSIVE CHARACTERISTICS: The anticipated foundation soils are expected to have a medium expansion potential ($50 < EI < 90$). The recommendations presented in this report reflect this condition.

CORROSIVITY

The water-soluble sulfate content was determined in accordance with California Test Method 417 for a representative soil sample from the site. The result of this test indicates that the foundation soils may be categorized as negligible (S0) per *ACI 318: Building Code Requirements for Structural Concrete*.

It should be understood Christian Wheeler Engineering does not practice corrosion engineering. If such an analysis is considered necessary, we recommend that the client retain an engineering firm that specializes in

this field to consult with them on this matter. The results of our tests should only be used as a guideline to determine if additional testing and analysis is necessary.

ON-GRADE SLABS

GENERAL: It is our understanding that the ground floor of the proposed addition will consist of a concrete slab-on-grade. The following recommendations are considered the minimum slab requirements based on the soil conditions and are not intended to be in lieu of structural considerations.

INTERIOR SLAB: From a geotechnical perspective, we recommend that the minimum floor slab thickness be 5 inches and that the floor slab be reinforced with at least No. 3 reinforcing bars placed at 18 inches on center each. Slab reinforcement should be supported on chairs such that the reinforcing bars are positioned at mid-height in the floor slab. The owner and the project structural engineer should determine if the on-grade slabs need to be designed for special loading conditions. For such cases, a subgrade modulus of 100 pounds per cubic inch can be assumed for the subgrade provided it is prepared as recommended in this report. The allowable bearing load for the slab is 1,500 pounds per square foot.

UNDER-SLAB VAPOR RETARDERS: Where floor coverings are installed, steps should be taken to minimize the transmission of moisture vapor from the subsoil through the interior slabs where it can potentially damage the interior floor coverings. We recommend that the owner/contractor follow national standards for the installation of vapor retarders below interior slabs as presented in currently published standards including ACI 302, "Guide to Concrete Floor and Slab Construction" and ASTM E1643, "Standard Practice for Installation of Water Vapor Retarder Used in Contact with Earth or Granular Fill Under Concrete Slabs."

EXTERIOR CONCRETE FLATWORK: Exterior concrete on-grade slabs not subject to vehicle loads should have a minimum thickness of 4 inches. Exterior slabs abutting perimeter foundations should be doveled into the footings. All slabs should be provided with weakened plane joints in accordance with the American Concrete Institute (ACI) guidelines. Alternative patterns consistent with ACI guidelines can also be used. A concrete mix with a 1-inch maximum aggregate size and a water/cement ratio of less than 0.6 is recommended for exterior slabs. Lower water content will decrease the potential for shrinkage cracks. Both coarse and fine aggregate should conform to the latest edition of the "Standard Specifications for Public Works Construction" ("Greenbook").

Special attention should be paid to the method of concrete curing to reduce the potential for excessive shrinkage and resultant random cracking. It should be recognized that minor cracks occur normally in

concrete slabs due to shrinkage. Some shrinkage cracks should be expected and are not necessarily an indication of excessive movement or structural distress.

EARTH RETAINING WALLS

FOUNDATIONS: Foundations for retaining walls can be designed in accordance with the foundation recommendations previously presented.

ACTIVE PRESSURES: The active soil pressure for the design of unrestrained earth retaining structures with level backfill surface may be assumed to be equivalent to the pressure of a fluid weighing 35 and 80 pounds per cubic foot for drained and undrained conditions, respectively. In the design of walls restrained from movement at the top (non-yielding walls) with a level backfill surface, the at-rest soil pressure may be assumed to be equivalent to the pressure of a fluid weighing 60 and 90 pounds per cubic foot for drained and undrained conditions, respectively.

Thirty percent of any area surcharge placed adjacent to the retaining wall may be assumed to act as a uniform horizontal pressure against the wall. Where vehicles will be allowed within ten feet of the retaining wall, a uniform horizontal pressure of 100 pounds per square foot should be added to the upper 10 feet of the retaining wall to account for the effects of adjacent traffic. Special cases such as a combination of shored and sloping temporary slopes, or other surcharge loads not described above, may require an increase in the design values recommended above. These conditions should be evaluated by the project geotechnical engineer on a case-by-case basis. If any other loads are anticipated, the Geotechnical Consultant should be contacted for the necessary increase in soil pressure.

For load combinations including earthquake, the seismic increment may be assumed to be equivalent to the pressure of a fluid weighing 17 pounds per cubic foot. For restrained basement walls, this value should be added to the active pressure (i.e. 35 pcf) as if the walls are unrestrained.

PASSIVE PRESSURES: The passive pressure for the prevailing soil conditions may be considered to be 350 pounds per square foot per foot of depth for foundations in formational soil. This pressure may be increased one-third for seismic loading. The coefficient of friction for concrete to soil may be assumed to be 0.30 for the resistance to lateral movement. When combining frictional and passive resistance, the friction should be reduced by one-third.

WATERPROOFING AND SUBDRAINS: The project architect should provide (or coordinate) waterproofing details for the retaining walls. The design values presented above are based on both drained and undrained backfill conditions. The final drainage configuration should be specified by the wall designer; however, several acceptable details are provided on Plate No. 3. Outlets points for the retaining wall subdrains should be coordinated by the project civil engineer. For subterranean walls, it may be necessary to collect the subdrain water in sumps and then pump it to an appropriate outlet.

BACKFILL: All retaining wall backfill should be compacted to at least 90 percent relative compaction. Expansive or clayey soils should not be used for backfill material (see Remedial Grading section of this report). Retaining walls should not be backfilled until the masonry/concrete has reached an adequate strength.

LIMITATIONS

REVIEW, OBSERVATION AND TESTING

The recommendations presented in this report are contingent upon our review of final plans and specifications. Such plans and specifications should be made available to the geotechnical engineer and engineering geologist so that they may review and verify their compliance with this report and with the California Building Code. It is recommended that Christian Wheeler Engineering be retained to provide continuous soil engineering services during the earthwork operations. This is to verify compliance with the design concepts, specifications or recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to start of construction.

UNIFORMITY OF CONDITIONS

The recommendations and opinions expressed in this report reflect our best estimate of the project requirements based on an evaluation of the subsurface soil conditions encountered at the subsurface exploration locations and on the assumption that the soil conditions do not deviate appreciably from those encountered. It should be recognized that the performance of the foundations and/or cut and fill slopes may be influenced by undisclosed or unforeseen variations in the soil conditions that may occur in the intermediate and unexplored areas. Any unusual conditions not covered in this report that may be encountered during site development should be brought to the attention of the geotechnical engineer so that he may make modifications if necessary.

CHANGE IN SCOPE

This office should be advised of any changes in the project scope or proposed site grading so that we may determine if the recommendations contained herein are appropriate. This should be verified in writing or modified by a written addendum.

TIME LIMITATIONS

The findings of this report are valid as of this date. Changes in the condition of a property can, however, occur with the passage of time, whether they be due to natural processes or the work of man on this or adjacent properties. In addition, changes in the Standards-of-Practice and/or Government Codes may occur. Due to such changes, the findings of this report may be invalidated wholly or in part by changes beyond our control. Therefore, this report should not be relied upon after a period of two years without a review by us verifying the suitability of the conclusions and recommendations.

PROFESSIONAL STANDARD

In the performance of our professional services, we comply with that level of care and skill ordinarily exercised by members of our profession currently practicing under similar conditions and in the same locality. The client recognizes that subsurface conditions may vary from those encountered at the locations where our test pits, surveys, and explorations are made, and that our data, interpretations, and recommendations be based solely on the information obtained by us. We will be responsible for those data, interpretations, and recommendations, but shall not be responsible for the interpretations by others of the information developed. Our services consist of professional consultation and observation only, and no warranty of any kind whatsoever, express or implied, is made or intended in connection with the work performed or to be performed by us, or by our proposal for consulting or other services, or by our furnishing of oral or written reports or findings.

CLIENT'S RESPONSIBILITY

It is the client's responsibility, or its representatives, to ensure that the information and recommendations contained herein are brought to the attention of the structural engineer and architect for the project and incorporated into the project's plans and specifications. It is further their responsibility to take the necessary measures to ensure that the contractor and his subcontractors carry out such recommendations during construction.

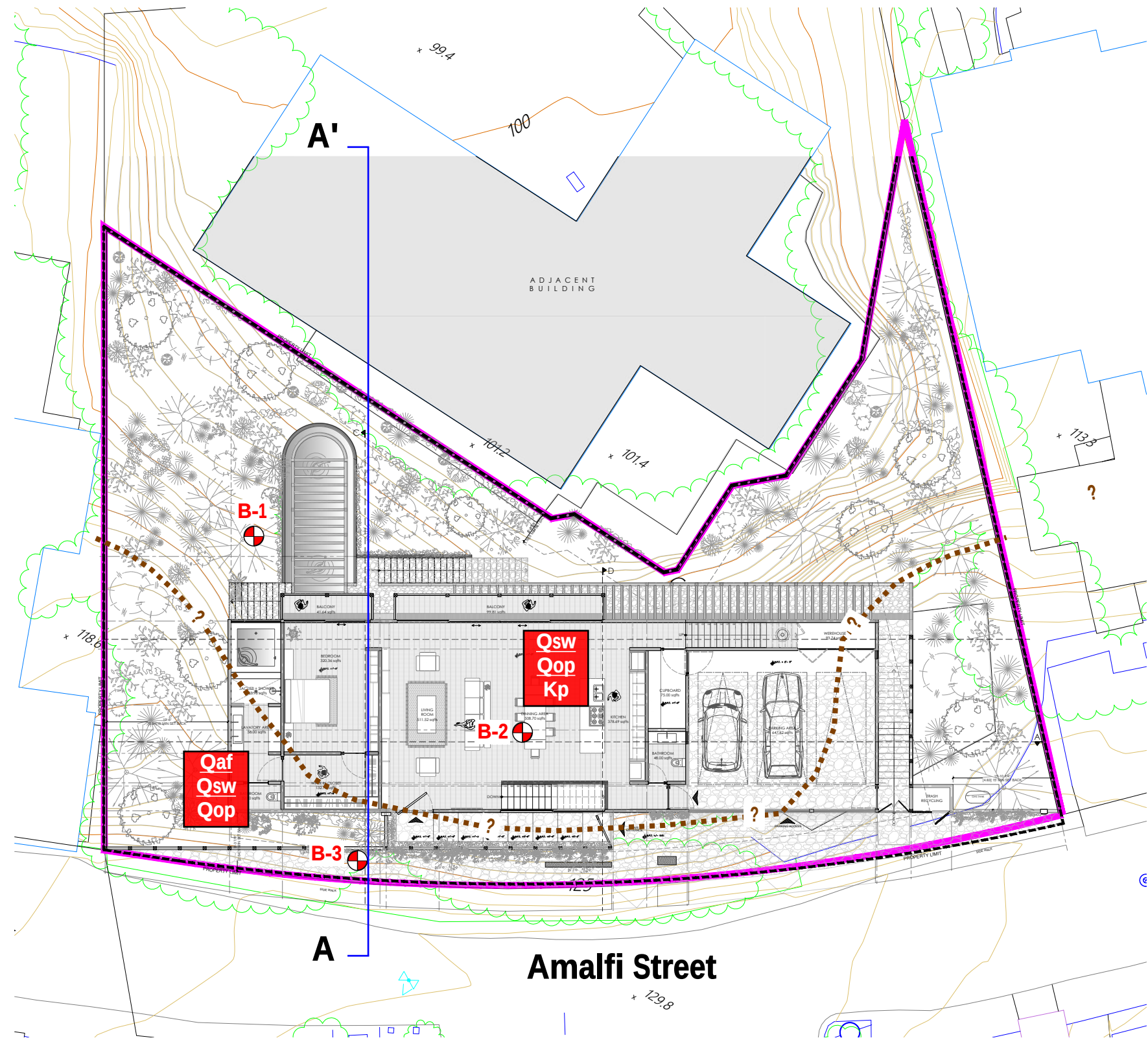
FIELD EXPLORATIONS

Three subsurface explorations were made during our investigation at the locations indicated on the Site Plan included herewith as Plate No. 1. These explorations consisted of small-diameter borings made on February 25, 2026 using limited access equipment. The fieldwork was conducted under the observation and direction of our engineering geology personnel.

The borings were carefully logged when made. The boring logs are presented in the attached Appendix A. The soils are described in accordance with the Unified Soils Classification. In addition, a verbal textural description, the wet color, the apparent moisture and the density or consistency are provided. The density of granular soils is given as either very loose, loose, medium dense, dense or very dense. The consistency of silts or clays is given as either very soft, soft, medium stiff, stiff, very stiff, or hard. Undisturbed samples of typical and representative soils were obtained and returned to the laboratory for testing. The undisturbed samples were obtained by driving a 2 3/8-inch inside diameter split-tube sampler ahead of the auger using a 140-pound weight free-falling a distance of 30 inches. The number of blows required to drive the sampler each foot was recorded and this value is presented on the attached boring logs as "Penetration Resistance." Bulk samples of disturbed soil were also collected in bags from the auger cuttings during the advancement of the borings and transported to the laboratory for testing.

LABORATORY TESTING

Laboratory tests were performed in accordance with the generally accepted American Society for Testing and Materials (ASTM) test methods or suggested procedures. A brief description of the tests performed and the subsequent results are presented in Appendix B.



CWE LEGEND

B-2

APPROXIMATE BORING LOCATION

Qaf
Qsw
Qop

ARTIFICIAL FILL over
SLOPEWASH over
OLD PARALIC DEPOSITS

Qsw
Qop
Kp

SLOPEWASH over
OLD PARALIC DEPOSITS over
POINT LOMA FORMATION

QUERRIED GEOLOGIC CONTACT

GEOLOGIC CROSS SECTION

FRAGMENTO

D: Av. Gobernador
Balarezo 9585,
Colonia Dávila 22044,
Tijuana, B.C.

T: 904 3902
M: frg.estmx@gmail.com

www.fragmento.mx

Name Project / Nombre del Proyecto

AMALFI HOUSE

Area / Superficie de construcción

8717.25 Sqfts / 809.85 m²

Type Project / Tipo de proyecto

Residential / Residencial

Dirección

1821 Torrey Pines Road,
La Jolla, CA 92037

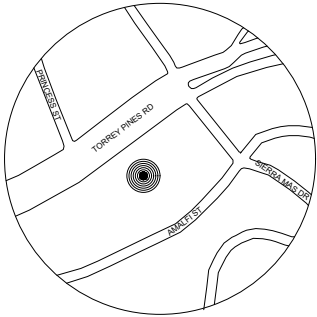
Parcel Number/Clave Catastral

350-161-1000

Owner / Propietario

David Shroedl

Site / Ubicación



Director de administración urbana

Planeación y Control urbano

Responsable de Proyecto

Responsable director de Obra

CED ESTATAL: -

CED FEDERAL: -

PERITO NO.: -

Diseño de proyecto

Arq. Marcos Gael Guerrero C.

Drawing / Dibujo

Arq. Vianey Sol.

Date /Fecha

August 2025 / Agosto 2025

Etap

ANTE PROYECTO

Scale/Escala

Acotación Pies

Plan/Indicada

Second Floor Plan

Sheet/Clave

SCALE: 1" = 20'

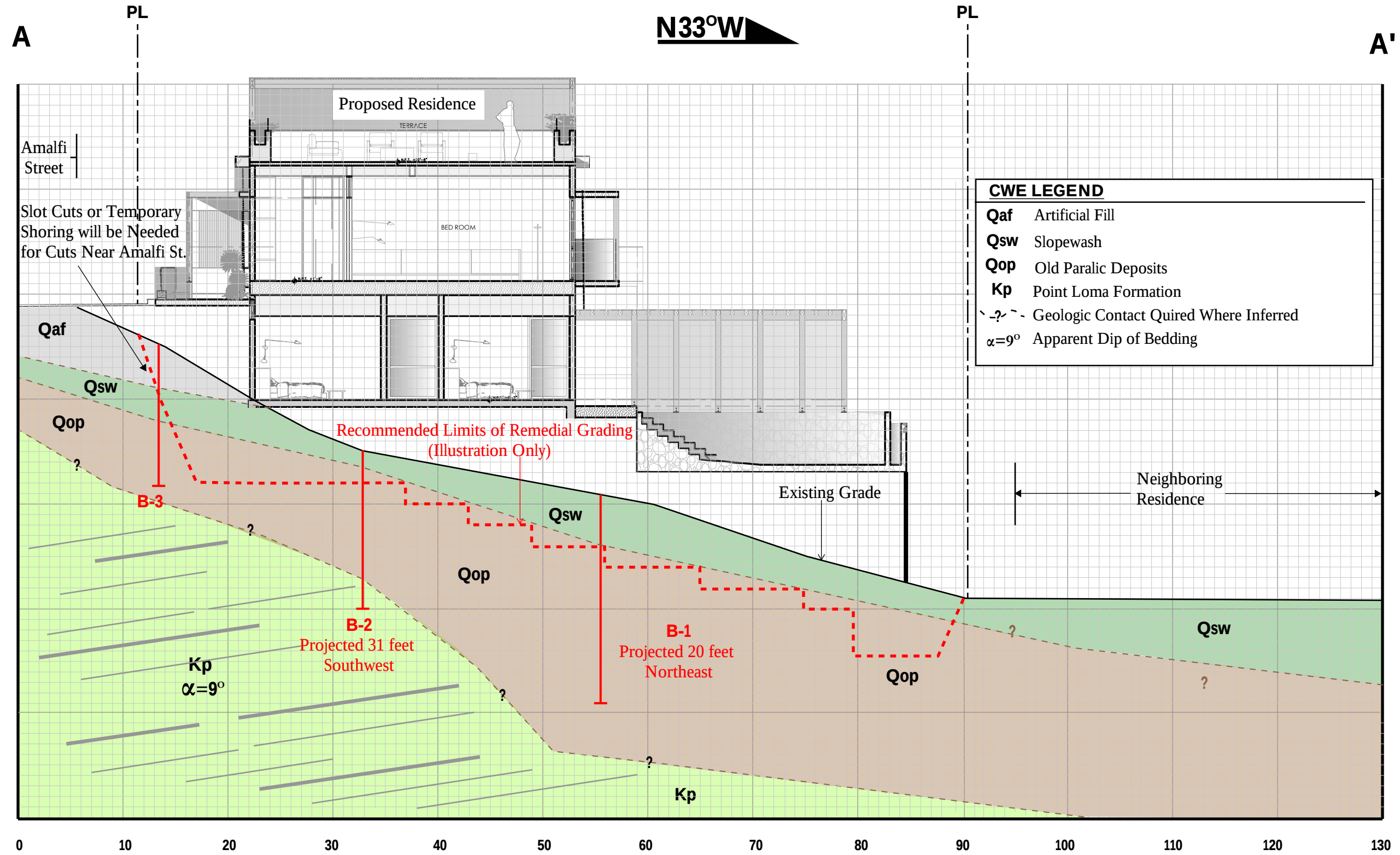
A1-02 2/35

FRAGMENTO.MX

SITE PLAN AND GEOTECHNICAL MAP

PROPOSED SINGLE-FAMILY RESIDENCE AMALFI STREET - APN 350-161-1000 LA JOLLA, CALIFORNIA	
DATE: APRIL 2026	REPORT NO.: 2260068.01
BY: SCC/SSD	PLATE NO.: 1

CHRISTIAN WHEELER
ENGINEERING



GEOLOGIC CROSS-SECTION A-A'

PROPOSED SINGLE-FAMILY RESIDENCE
 AMALFI STREET - APN 350-161-1000
 LA JOLLA, CALIFORNIA

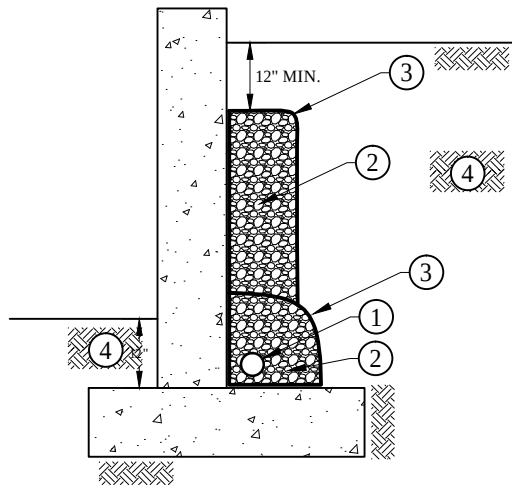
DATE: APRIL 2026

REPORT NO.: 2260068.01

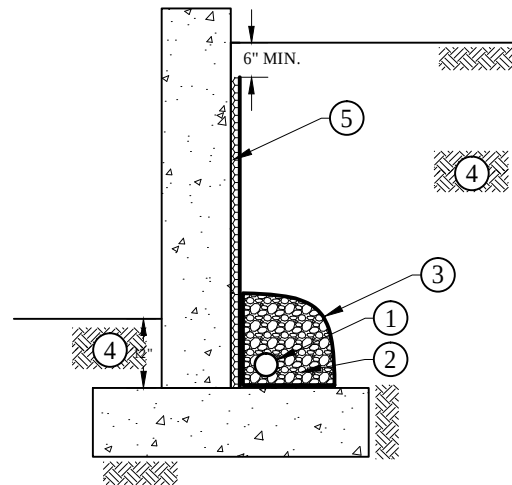
BY: SCC

PLATE NO.: 2

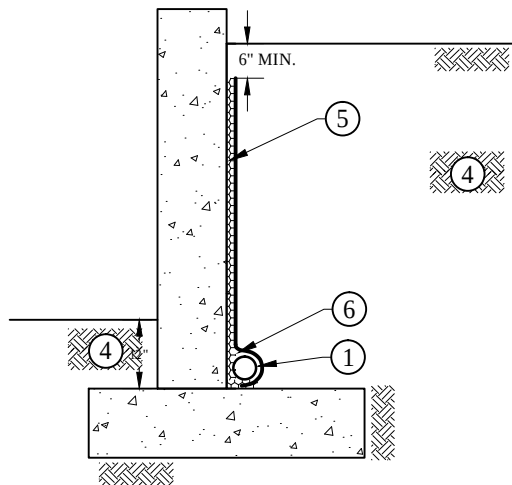




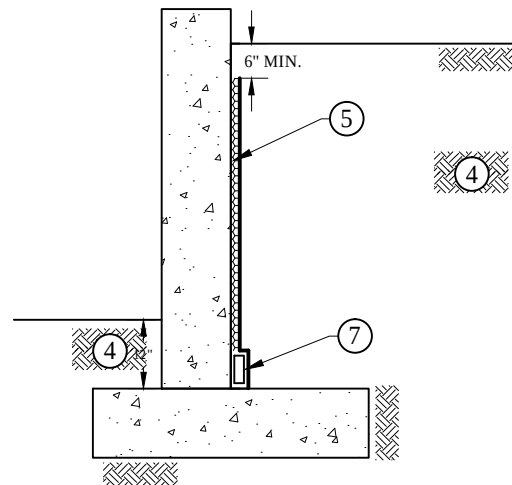
1 DETAIL



2 DETAIL



3 DETAIL



4 DETAIL

NOTES AND DETAILS

GENERAL NOTES:

- 1) THE NEED FOR WATERPROOFING SHOULD BE EVALUATED BY OTHERS.
- 2) WATERPROOFING TO BE DESIGNED BY OTHERS (CWE CAN PROVIDE A DESIGN IF REQUESTED).
- 3) EXTEND DRAIN TO SUITABLE DISCHARGE POINT PER CIVIL ENGINEER.
- 4) DO NOT CONNECT SURFACE DRAINS TO SUBDRAIN SYSTEM.

DETAILS:

- | | |
|---|--|
| <p>① 4-INCH PERFORATED PVC PIPE ON TOP OF FOOTING, HOLES POSITIONED DOWNWARD (SDR 35, SCHEDULE 40, OR EQUIVALENT).</p> <p>② ¾ INCH OPEN-GRADED CRUSHED AGGREGATE.</p> <p>③ GEOFABRIC WRAPPED COMPLETELY AROUND ROCK.</p> <p>④ PROPERLY COMPACTED BACKFILL SOIL.</p> <p>⑤ WALL DRAINAGE PANELS (MIRADRAIN OR EQUIVALENT) PLACED PER MANUFACTURER'S RECS.</p> | <p>⑥ UNDERLAY SUBDRAIN WITH AND CUT FABRIC BACK FROM DRAINAGE PANELS AND WRAP FABRIC AROUND PIPE.</p> <p>⑦ COLLECTION DRAIN (TOTAL DRAIN OR EQUIVALENT) LOCATED AT BASE OF WALL DRAINAGE PANEL PER MANUFACTURER'S RECOMMENDATIONS.</p> |
|---|--|

CANTILEVER RETAINING WALL DRAINAGE SYSTEMS

PROPOSED SINGLE-FAMILY RESIDENCE
AMALFI STREET - APN 350-161-1000
LA JOLLA, CALIFORNIA

DATE: APRIL 2026

REPORT NO.: 2260068.01

BY: SD

PLATE NO.: 3



CHRISTIAN WHEELER
ENGINEERING

Appendix C

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Appendix B

Laboratory Test Results

Laboratory tests were performed in accordance with the generally accepted American Society for Testing and Materials (ASTM) test methods or suggested procedures. Brief descriptions of the tests performed are presented below:

- a) **CLASSIFICATION:** Field classifications were verified in the laboratory by visual examination. The final soil classifications are in accordance with the Unified Soil Classification System and are presented on the exploration logs in Appendix A.
- b) **MOISTURE-DENSITY:** In-place moisture contents and dry densities were determined for representative soil samples. This information was an aid to classification and permitted recognition of variations in material consistency with depth. The dry unit weight is determined in pounds per cubic foot, and the in-place moisture content is determined as a percentage of the soil's dry weight. The results of these tests are summarized in the exploration logs presented in Appendix A.
- c) **DIRECT SHEAR:** Direct shear tests were performed to determine the failure envelope of selected soils based on yield shear strength. The shear box was designed to accommodate a sample having a diameter of 2.375 inches or 2.50 inches and a height of 1.0 inch. Samples were tested at different vertical loads and a saturated moisture content. The shear stress was applied at a constant rate of strain of approximately 0.05 inch per minute.
- d) **CONSOLIDATION TESTS:** One-dimensional consolidation testing was performed in accordance with ASTM D2435. The specimen was placed in a consolidometer with porous stones at the top and bottom and loads were applied in a geometric progression. After vertical movement ceased with each load interval, the resulting deformation was recorded. The percent consolidation is reported as the ratio of vertical compression to the original sample height. The test sample was inundated at some point in the test cycle to determine its behavior under the anticipated loads as soil moisture increases.
- e) **EXPANSION INDEX TEST:** The expansion index of a selected soil was determined in accordance with ASTM D4829. A 1-inch-thick by 4-inch-diameter specimen was prepared by compacting the soil with a specified energy at approximately 50 percent saturation. The specimen was placed in a consolidometer with porous stones at the top and bottom and a total normal pressure of 144.7 psf was applied. The specimen was allowed to consolidate for a period of 10 minutes and then saturated. The change in vertical movement was recorded until the rate of expansion became nominal.
- f) **SOLUBLE SULFATES:** The soluble sulfate content was determined for samples of soil likely to be present at the foundation level. The soluble sulfate content was determined in accordance with California Test Method 417.

LABORATORY TEST RESULTS

PROPOSED SINGLE-FAMILY RESIDENCE
AMALFI STREET - APN 350-161-1000
LA JOLLA, CALIFORNIA

DATE: APRIL 2026

PROJECT NO.: 2260068

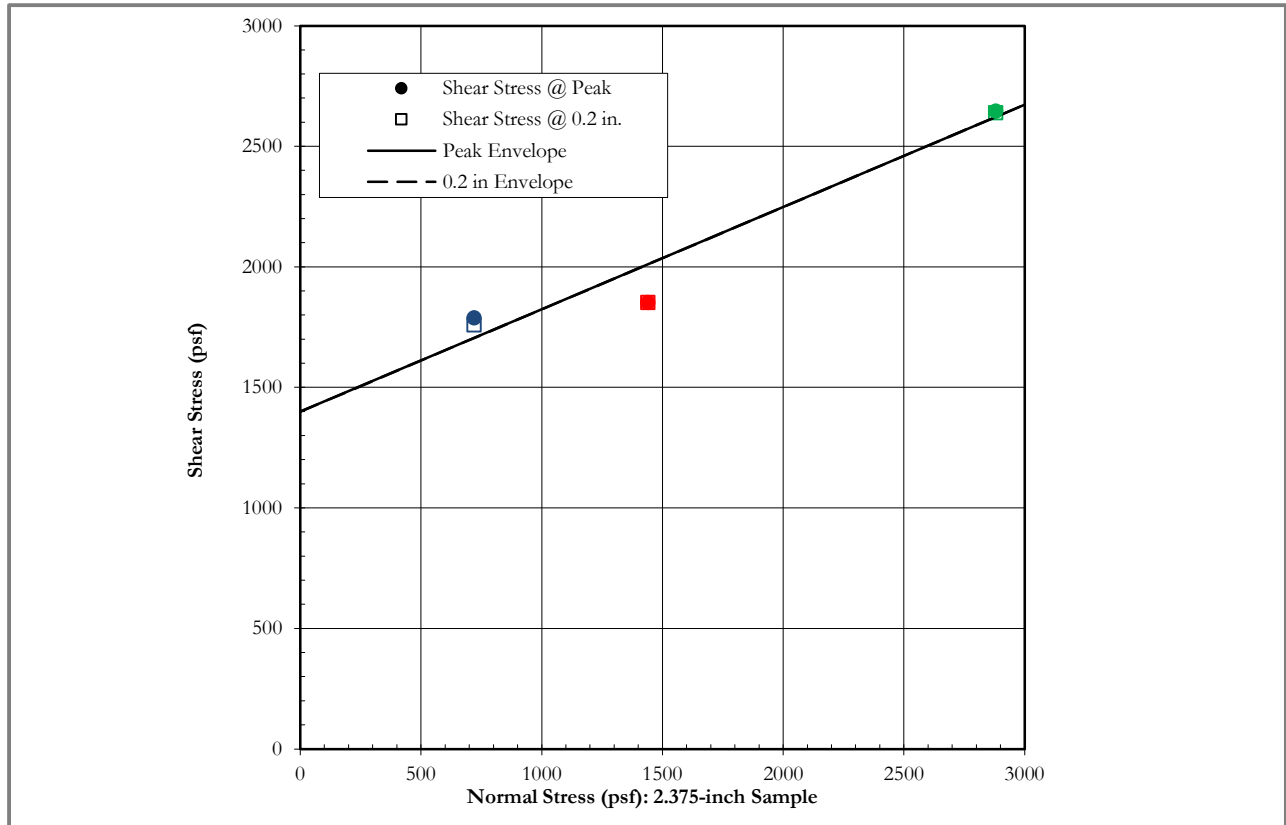
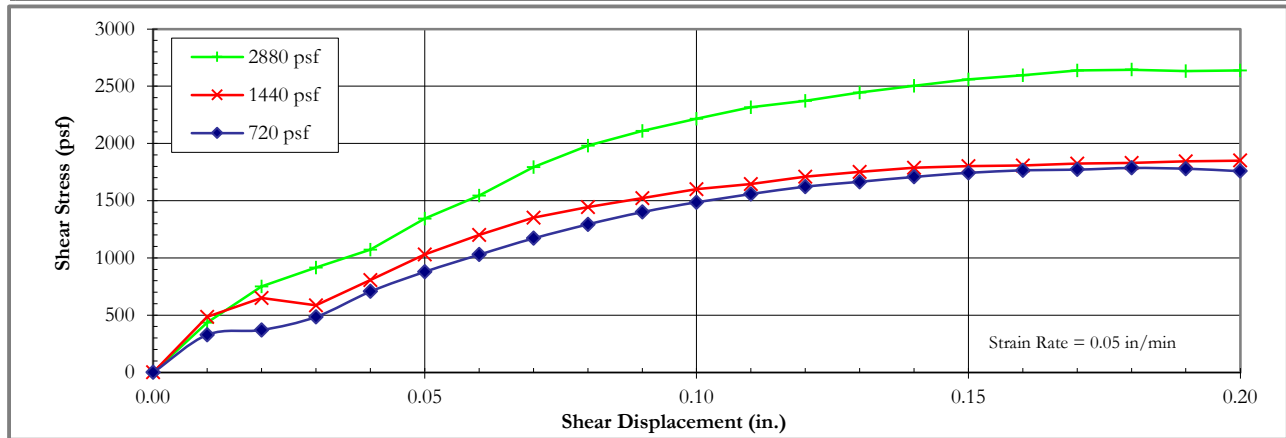
BY: SCC

FIGURE NO.: B-1



CHRISTIAN WHEELER
ENGINEERING

DIRECT SHEAR TEST (ASTM D3080)



Sample No. **B-2 @ 2½'**

Sample Type: Undisturbed (Ring)

Normal Stress (psf)	720	1440	2880
Peak Shear Stress (psf)	1788	1852	2646
Shear Stress at 0.2 in (psf)	1759	1852	2638
Initial Dry Density (pcf)	124.2	120.7	120.2
Initial Moisture Content (%)	9.4	9.2	12.2

	Peak	at 0.2 in Displacement
Friction Angle, ϕ (deg):	23	23
Cohesion Intercept, c (psf):	1400	1400

**LABORATORY
TEST RESULTS**

PROPOSED SINGLE-FAMILY RESIDENCE
AMALFI STREET - APN 350-161-1000
LA JOLLA, CALIFORNIA

DATE: APRIL 2026

PROJECT NO.: 2260068

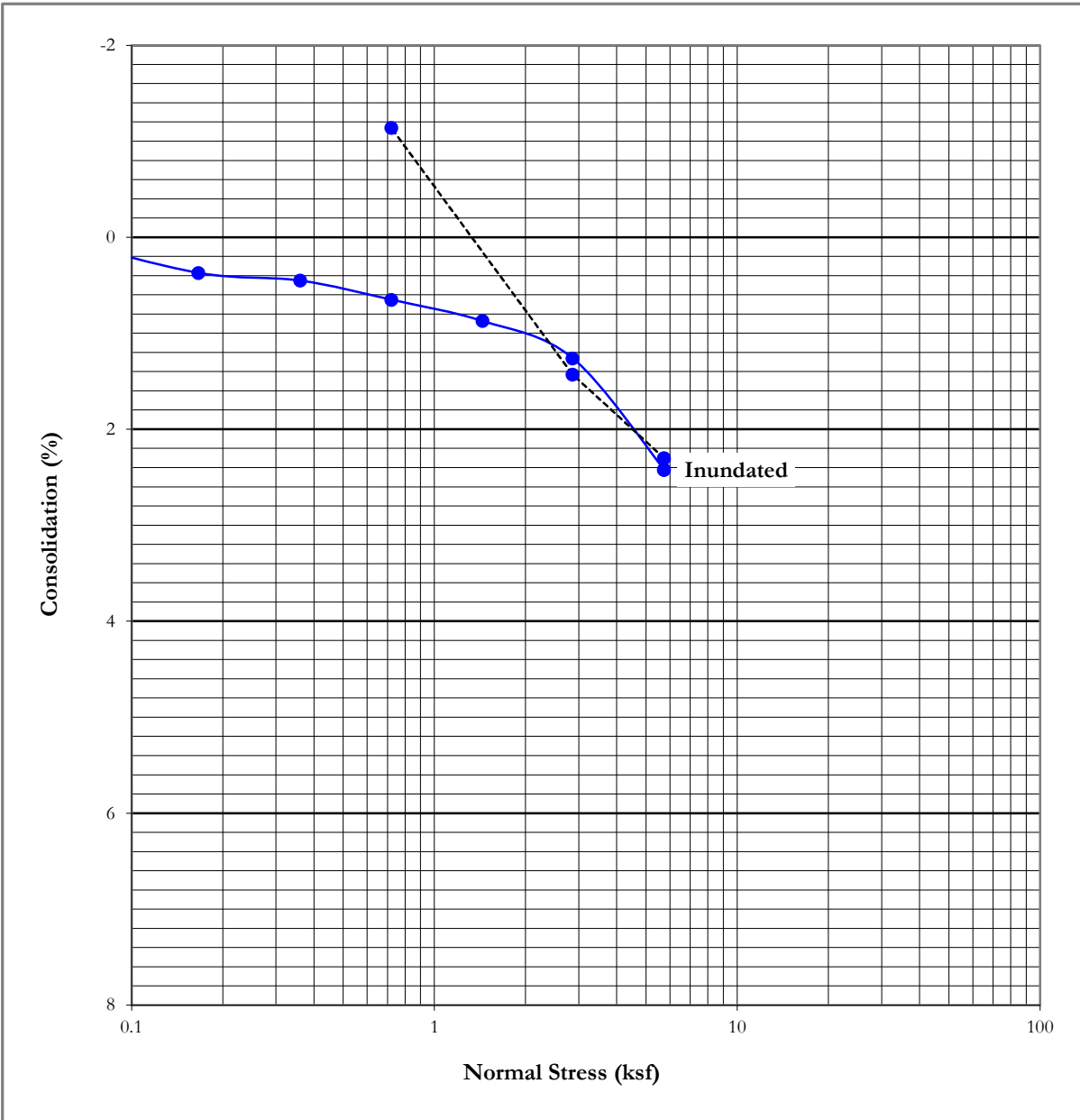
BY: SCC

FIGURE NO.: B-2



CHRISTIAN WHEELER
ENGINEERING

CONSOLIDATION (ASTM D2435)



Sample No	Initial Moisture Content	Initial Dry Density	Final Moisture Content
B-1 @ 9'	11.4%	120.5	16.1%

LABORATORY TEST RESULTS

PROPOSED SINGLE-FAMILY RESIDENCE
 AMALFI STREET - APN 350-161-1000
 LA JOLLA, CALIFORNIA

DATE: APRIL 2026

PROJECT NO.: 2260068

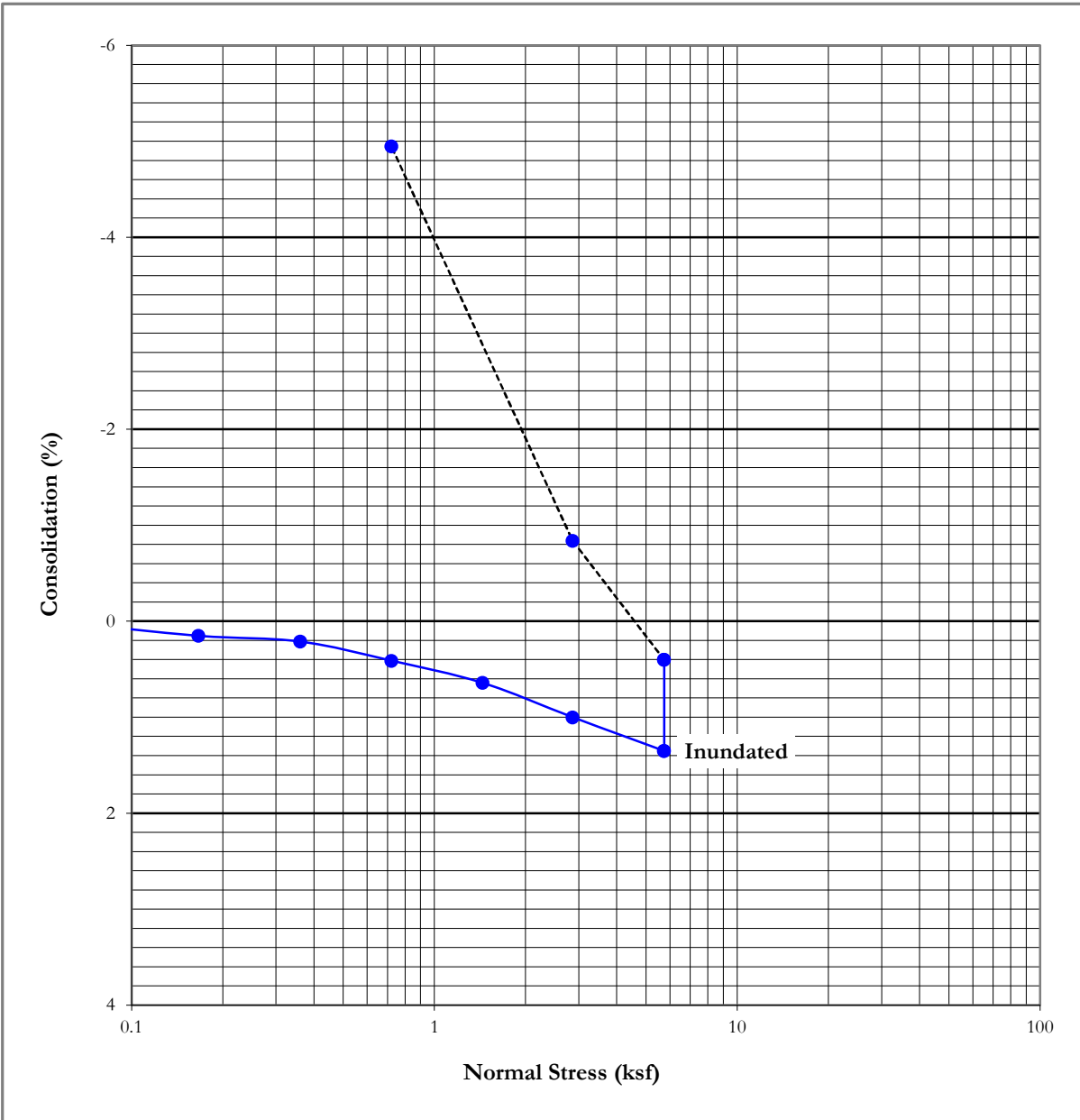
BY: SCC

FIGURE NO.: B-3



CHRISTIAN WHEELER
 ENGINEERING

CONSOLIDATION (ASTM D2435)



Sample No	Initial Moisture Content	Initial Dry Density	Final Moisture Content
B-2 @ 9'	13.5%	118.1	19.7%

LABORATORY TEST RESULTS

PROPOSED SINGLE-FAMILY RESIDENCE
 AMALFI STREET - APN 350-161-1000
 LA JOLLA, CALIFORNIA

DATE: APRIL 2026

PROJECT NO.: 2260068

BY: SCC

FIGURE NO.: B-4



CHRISTIAN WHEELER
 ENGINEERING

EXPANSION INDEX (ASTM D2849)

Sample No.	Initial Moisture (%)	Initial Dry Density (pcf)	Final Moisture (%)	Expansion Index	Expansion Potential
B-2 @ 1½'-6'	10.5	105.6	24.5	72	Medium

CLASSIFICATION OF EXPANSION POTENTIAL

Expansion Index	Expansion Potential
1-20	Very Low
21-50	Low
51-90	Medium
91-130	High
> 130	Very High

LABORATORY TEST RESULTS

PROPOSED SINGLE-FAMILY RESIDENCE
AMALFI STREET - APN 350-161-1000
LA JOLLA, CALIFORNIA

DATE: APRIL 2026

PROJECT NO.: 2260068

BY: SCC

FIGURE NO.: B-5



CHRISTIAN WHEELER
ENGINEERING

CORROSIVITY TESTS

Sample No.	CALTEST 417	CALTEST 643		CALTEST 422
	Sulfate Content (% SO ₄)	pH	Resistivity (ohm-cm)	Chloride Content (ppm)
B-2 @ 1½'-6'	0.012	--	--	--

LABORATORY TEST RESULTS

PROPOSED SINGLE-FAMILY RESIDENCE
AMALFI STREET - APN 350-161-1000
LA JOLLA, CALIFORNIA

DATE: APRIL 2026

PROJECT NO.: 2260068

BY: SCC

FIGURE NO.: B-6



CHRISTIAN WHEELER
ENGINEERING

Appendix A

Exploration Logs

LOG OF TEST BORING B-1

Sample Type and Laboratory Test Legend

Cal	Modified California Sampler	CK	Chunk
SPT	Standard Penetration Test	DR	Drive Ring
ST	Shelby Tube		
MD	Max Density	DS	Direct Shear
SO ₄	Soluble Sulfates	Con	Consolidation
SA	Sieve Analysis	EI	Expansion Index
HA	Hydrometer	R-Val	Resistance Value
SE	Sand Equivalent	Chl	Soluble Chlorides
PI	Plasticity Index	Res	pH & Resistivity
CP	Collapse Potential	SD	Sample Density

Date Logged: 02/25/26 Equipment: Tripod
 Logged By: DJF Auger Type: 6" solid flight
 Existing Elevation: 110' Drive Type: 140lbs @ 30" drop
 Finish Elevation: ±120' Depth to Water: N/A

DEPTH (ft)	ELEVATION (ft)	GRAPHIC LOG	USCS SYMBOL	SUMMARY OF SUBSURFACE CONDITIONS (based on Unified Soil Classification System)	PENETRATION (blows per foot)	SAMPLE TYPE	BULK	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	RELATIVE COMPACTION (%)	LABORATORY TESTS
2	108		CL	Slopewash (Qsw): Grayish-brown, moist, stiff, LEAN CLAY, porous.	16	Cal					
4	106										
6	104		CL	Old Paralic Deposits (Qop): Olive brown, moist, very stiff, sandy LEAN CLAY with trace gravels.	38	Cal					
8	102			Hard.							
10	100				50/5"	Cal		11.4	120.5		Con
12	98			Mottled with reddish iron staining.	50/6"	Cal					
14	96										
16	94				50/5"	Cal					
18	92										
20	90		SP-SW	Light brown, moist, medium dense, fine- to coarse-grained, Poorly Graded SAND with SILT, friable with gravels. Possible basal depositional bed.	23	SPT					
22				Terminated at 20 feet. No groundwater or seepage encountered.							
24											

Notes:

Symbol Legend

- Groundwater Level During Drilling
- Groundwater Level After Drilling
- Apparent Seepage
- * No Sample Recovery
- ** Non-representative Blow Count (rocks present)

PROPOSED SINGLE-FAMILY RESIDENCE

AMALFI STREET - APN 350-161-11
 LA JOLLA, CALIFORNIA

DATE: APRIL 2026 JOB NO.: 2260068.01
 BY: SD APPENDIX: A-1



CHRISTIAN WHEELER
 ENGINEERING

LOG OF TEST BORING B-2

Sample Type and Laboratory Test Legend

Cal	Modified California Sampler	CK	Chunk
SPT	Standard Penetration Test	DR	Drive Ring
ST	Shelby Tube		
MD	Max Density	DS	Direct Shear
SO ₄	Soluble Sulfates	Con	Consolidation
SA	Sieve Analysis	EI	Expansion Index
HA	Hydrometer	R-Val	Resistance Value
SE	Sand Equivalent	Chl	Soluble Chlorides
PI	Plasticity Index	Res	pH & Resistivity
CP	Collapse Potential	SD	Sample Density

Date Logged: 02/25/26 Equipment: Tripod
 Logged By: DJF Auger Type: 6" solid flight
 Existing Elevation: 115' Drive Type: 140lbs @ 30" drop
 Finish Elevation: ±120' Depth to Water: N/A

DEPTH (ft)	ELEVATION (ft)	GRAPHIC LOG	USCS SYMBOL	SUMMARY OF SUBSURFACE CONDITIONS (based on Unified Soil Classification System)	PENETRATION (blows per foot)	SAMPLE TYPE	BULK	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	RELATIVE COMPACTION (%)	LABORATORY TESTS
			CL	Slopewash (Qsw): Grayish-brown, moist, stiff, LEAN CLAY, porous.							
2	113		CL	Old Paralic Deposits (Qop): Olive brown, moist, very stiff, sandy LEAN CLAY, mottled with reddish iron staining. Expansion Index = 72 (Medium)	45	Cal		10.3	121.7		SA EI SO ₄ DS
4	111										
6	109				27	SPT					
8	107			Hard.	50/6"	Cal		13.5	118.1		Con
10	105										
12	103										
14	101		CL	Point Loma Formation (Kp): Claystone, olive gray, moist, hard, LEAN CLAY with orangish iron staining.	50/5"	Cal					
					46	SPT					
16	99			Terminated at 14.5 feet. Groundwater encountered at 8 feet.							
18											
20											
22											
24											

Notes:

Symbol Legend

-  Groundwater Level During Drilling
-  Groundwater Level After Drilling
-  Apparent Seepage
- * No Sample Recovery
- ** Non-representative Blow Count (rocks present)

PROPOSED SINGLE-FAMILY RESIDENCE AMALFI STREET - APN 350-161-11 LA JOLLA, CALIFORNIA

DATE: APRIL 2026 JOB NO.: 2260068.01
 BY: SD APPENDIX: A-2



CHRISTIAN WHEELER
ENGINEERING

LOG OF TEST BORING B-3

Sample Type and Laboratory Test Legend

Cal	Modified California Sampler	CK	Chunk
SPT	Standard Penetration Test	DR	Drive Ring
ST	Shelby Tube		
MD	Max Density	DS	Direct Shear
SO ₄	Soluble Sulfates	Con	Consolidation
SA	Sieve Analysis	EI	Expansion Index
HA	Hydrometer	R-Val	Resistance Value
SE	Sand Equivalent	Chl	Soluble Chlorides
PI	Plasticity Index	Res	pH & Resistivity
CP	Collapse Potential	SD	Sample Density

Date Logged: 02/25/26 Equipment: Tripod
 Logged By: DJF Auger Type: 6" solid flight
 Existing Elevation: 125' Drive Type: 140lbs @ 30" drop
 Finish Elevation: ±129' Depth to Water: N/A

DEPTH (ft)	ELEVATION (ft)	GRAPHIC LOG	USCS SYMBOL	SUMMARY OF SUBSURFACE CONDITIONS (based on Unified Soil Classification System)	PENETRATION (blows per foot)	SAMPLE TYPE	BULK	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	RELATIVE COMPACTION (%)	LABORATORY TESTS
2	123		CL	Artificial Fill (Qaf): Grayish-brown, moist, stiff, sandy LEAN CLAY with moderate roots to 18".							
					Stiff.	24	Cal				
4	121										
6	119		CL	Slopewash (Qsw): Grayish-brown, moist, very stiff, LEAN CLAY, porous.	49	Cal					
8	117										
10	115					80	Cal				
12	113		CL	Old Paralic Deposits (Qop): Olive brown, moist, hard, sandy LEAN CLAY with reddish iron staining.	50/6"	Cal					
14	111				Terminated at 13 feet. Groundwater encountered at 8 feet.						
16											
18											
20											
22											
24											

Notes:

Symbol Legend

-  Groundwater Level During Drilling
-  Groundwater Level After Drilling
-  Apparent Seepage
- * No Sample Recovery
- ** Non-representative Blow Count (rocks present)

PROPOSED SINGLE-FAMILY RESIDENCE

AMALFI STREET - APN 350-161-11
 LA JOLLA, CALIFORNIA

DATE: APRIL 2026 JOB NO.: 2260068.01
 BY: SD APPENDIX: A-3



CHRISTIAN WHEELER
 ENGINEERING