GEOTECHNICAL EVALUATION
PROPOSED RESIDENTIAL DEVELOPMENT (“COVE HOUSE”)
LOTS 2 AND 17 OF BLOCK 46
LA JOLLA, SAN DIEGO COUNTY, CALIFORNIA 92037
ASSESSOR’S PARCEL NUMBERS (APNS) 350-131-02-00 AND -29-00

FOR

HERITAGE BRIDGE, LLC, FALCON COVE, LLC
481 E. SUN SPRING PLACE
ORO VALLEY, ARIZONA 85755

W.O. 8358-A-SC REVISED AUGUST 23, 2022

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

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Revised August 23, 2022

W.O. 8358-A-SC

Heritage Bridge, LLC, Falcon Cove, LLC
481 E. Sun Spring Place
Oro Valley, Arizona 85755

Subject: Geotechnical Evaluation, Proposed Residential Development ("Cove House"), Lots 2 and 17 of Block 46, La Jolla, San Diego County, California 92037, Assessor's Parcel Numbers (APNs) 350-131-02-00 and -29-00

Dear Sir or Madame:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) has performed a geotechnical evaluation of the subject parcels. The purpose of our study was to evaluate the onsite geologic, geomorphic, and geotechnical conditions relative to the proposed residential development thereon, in order to give a geotechnical opinion regarding the feasibility of the project, and to provide preliminary geotechnical recommendations for earthwork and the design, and construction of foundations, concrete slab-on-grade floors, retaining walls, a swimming pool and spa, pedestrian and vehicular pavements, and other earth-supported improvements.

EXECUTIVE SUMMARY

Based on our review of the available data (see Appendix A), field exploration, laboratory testing, and geologic and engineering analysis, the proposed residential development at the subject parcels appears to be feasible from a geotechnical perspective, provided the recommendations presented in the text of this report are properly incorporated into the planning, design, and construction of the project. The most significant elements of this study are summarized below:

- The results of our quantitative slope stability analyses and coastal bluff retreat evaluation indicate that a geologic setback of at least 24¼ feet from the coastal bluff edge is sufficient mitigation against coastal bluff failure and retreat over the 75-year design life of the proposed development within APN 350-131-02-00 (the parcel in closest proximity to the coastal bluff). However, the minimum setback distance for primary and accessory structures, and grading from the edges of coastal bluffs in the City of San Diego is 25 feet. Thus, the proposed development and grading should be sited at least 25 feet from the coastal bluff edge. Based on our understanding of the proposed development, planned improvements and grading will occur further than 25 feet landward of the coastal bluff edge. Shoreline protection measures would not be required over the aforementioned design life provided that the geologic setback for the proposed development conforms to the City of San Diego minimum standard.

- The Quaternary-age residual soil that occurs within the upper approximately 1 foot to 2 feet of the existing grades, throughout the subject parcels, is considered unsuitable for the support of the proposed settlement-sensitive improvements and new planned fills. Any undocumented artificial fill encountered during earthwork construction would also be considered inappropriate bearing materials. If not extracted by the planned excavations, the residual soil and any undocumented fill should be removed to expose suitable old paralic deposits, cleaned of any organic matter and deleterious debris, and reused as compacted fill per the recommendations in this report.

- Our laboratory testing and past work experience with nearby sites indicates that the onsite earth materials meet the criteria of expansive soils. Thus, structural or earthwork mitigation is recommended to reduce the adverse effects of shrink/swell deformations on the proposed improvements and to comply with the currently adopted building code.

- Owing to proximity of the Pacific Ocean, structural concrete used in the proposed development may come into contact with sea spray. This should be considered in the mix design of the structural concrete, if warranted by the project architect.

- Infiltration into the onsite soils for permanent post-construction storm water best management practices (BMPs) is not recommended, as it would have negative impacts on the stability of the nearby coastal bluff and the proposed onsite and existing offsite improvements.

- Adverse geologic features that would preclude project feasibility were not encountered.

The opportunity to be of service is sincerely appreciated. If you should have any questions, please do not hesitate to contact our office.

Respectfully submitted,

GeoSoils, Inc.

John P. Franklin
Engineering Geologist, CEG 1340

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Geotechnical Engineer, GE 2057

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RBB/JPF/SJC/sh

Distribution: (3) Addressee (2 wet signed and 1 PDF)

TABLE OF CONTENTS

GEOTECHNICAL EVALUATION
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SCOPE OF SERVICES

The scope of our services has included the following:

1. Reviews of the project plans, in-house geologic literature, regional geologic maps, and aerial photographs(see Appendix A).

2. Geologic site reconnaissance, mapping, and subsurface exploration with two (2) exploratory borings to evaluate the near-surface soil and geologic conditions, and to sample the onsite earth materials (see Appendix B).

3. An evaluation of storm water infiltration feasibility (Appendix C).

4. General areal geologic hazard and seismicity evaluations (Appendix D).

5. Appropriate laboratory testing of representative soil samples (Appendix E).

6. Engineering analyses of the data collected, including slope stability (Appendix F).

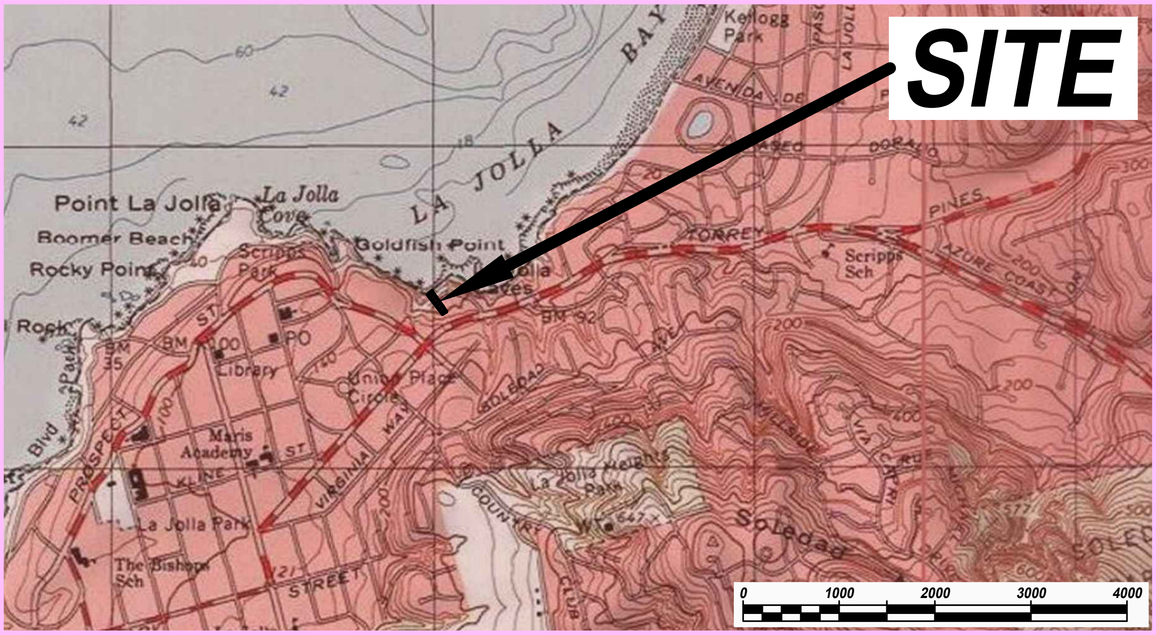
7. Preparation of this summary report and accompaniments.

EXISTING SITE CONDITIONS AND PROPOSED DEVELOPMENT

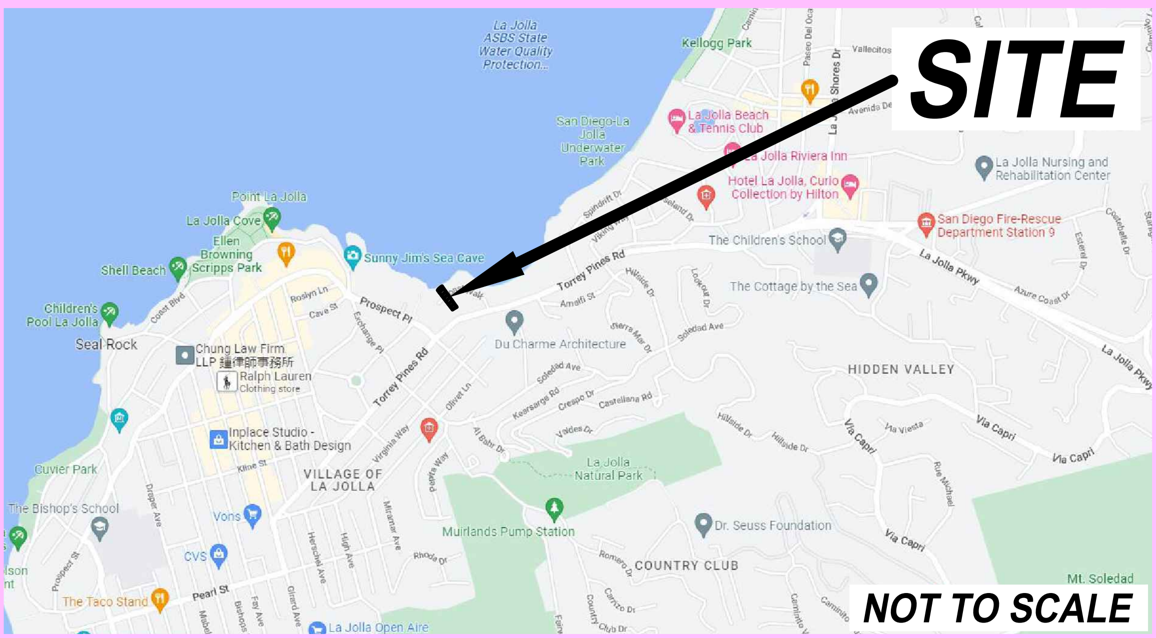
Existing Site Conditions

The subject site consists of two (2) irregularly-shaped land parcels, comprising about ½-acre southeast of Coast Walk, in La Jolla, San Diego County, California 92037 (see Figure 1, Site Location Map). The geographic coordinates of the approximate centroid of the site are 32.848272° North and -117.266764° West. The site is bounded by Coast Walk to the northwest, by Torrey Pines Road to the southeast, and by developed residential properties to the remaining quadrants.

Topographically, the site is situated upon a gently to moderately sloping coastal terrace that overlooks a steep coastal bluff, descending to Pacific Ocean shoreline. According to the site plans prepared by Island Architects ([IA], 2022a, 2022b), site elevations range between approximately 77 and 106 feet above mean sea level (MSL), for an overall relief on the order of 29 feet. In general, the site slopes toward the north and northwest at gradients on the order of 9:1 (horizontal:vertical [h:v]) or flatter, with local slope gradients as steep as approximately 1.5:1 (h:v), near the northwestern property corner. Surface



Base Map: TOPO!® ©2018 National Geographic, U.S.G.S. La Jolla Quadrangle, California -- San Diego Co., 7.5 Minute, dated 1996, current, 1982.



Base Map: Google Maps, Copyright 2022 Google, Map Data Copyright 2022 Google

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SITE LOCATION MAP

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Figure 1

drainage is primarily accommodated by sheet-flow runoff that follows surficial topography to the north and northwest.

The northwestern parcel (Lot 2) is undeveloped; whereas the other parcel (Lot 17) shares a tennis court and driveway with the adjacent northeastern property. Vegetation on Lot 2 generally consists of a thicket of pine trees and shrubbery. Vegetation within Lot 17 mostly consists of ornamental shrubbery and grass that surrounds the tennis court.

Proposed Development

Based on our review of the project architectural plans prepared by Island Architects, ([IA], 2022a), GSI understands that the proposed development within the subject parcels includes razing the existing tennis court and preparing the site to receive a new single-family residence and accessory dwelling unit (ADU) with an associated swimming pool, retaining walls, and vehicular and pedestrian pavements.

The proposed single-family residence will include a total of four (4) floor levels with one (1) underground level. The proposed ADU will consist of one (1) above-ground floor level and one (1) underground floor level. IA (2022a) shows that cut and fill grading will be necessary to achieve the design grades with maximum planned cuts and fills on the order of 14½ and 9½ feet, respectively. It appears that retaining walls will primarily be used to accommodate grade transitions. IA (2022a) indicates that the maximum planned retained soil height will be on the order of 23½ feet.

GSI anticipates that the proposed buildings will consist of a combination of wood-frame and masonry construction supported by shallow foundations with concrete slab-on-grade floors. Building loads are currently unavailable but assumed to be similar to typical of relatively lightly loaded residential construction. Ingress/egress to the proposed development will occur through a private driveway that extends from the Coast Walk roadway, near its intersection with N. Torrey Pines Road. GSI anticipates that underground utilities servicing the proposed driveway will traverse the aforementioned private driveway. Sanitary sewage disposal will be tied into the municipal system.

SITE EXPLORATION

On June 2 and 3, 2022, a GSI field representative conducted surficial geologic mapping and performed subsurface exploration within the subject parcels. Near-surface soil and geologic conditions were explored with two (2) exploratory borings, advanced with a limited-access (tripod) drill rig. The approximate locations of the borings are shown on Plate 1 (Geotechnical Map), which has been adopted from IA (2022a, 2022b). Three (3) geologic cross sections depicting the subsurface conditions in profile view are provided on Plates 2 and 3. The logs of the exploratory borings are provided in Appendix B.

PHYSIOGRAPHIC AND REGIONAL GEOLOGIC SETTINGS

Physiographic Setting

The site is located in the coastal plain physiographic section of San Diego County. The coastal plain section is characterized by pronounced marine wave-cut terraces intermittently dissected by stream channels that convey water from the eastern highlands to the Pacific Ocean.

Regional Geologic Setting

San Diego County lies within the Peninsular Ranges Geomorphic Province of southern California. This province is characterized as elongated mountain ranges and valleys that trend northwest (Norris and Webb, 1990). The province extends from the base of the east-west aligned Santa Monica - San Gabriel Mountains, and continues south into Baja California, Mexico. The mountain ranges within this province are underlain by basement rocks consisting of pre-Cretaceous metasedimentary rocks, Jurassic metavolcanic rocks, and Cretaceous plutonic (granitic) rocks.

As indicated by Kennedy and Tan (2008), a relatively thick (> 1,000 meters) succession of Upper Cretaceous-, Tertiary-, and Quaternary-age sediments unconformably overlie basement rocks in southwestern San Diego County. The Upper Cretaceous units are composed of marine turbidites and continental fan deposits assigned to the Rosario Group. After the deposition of the Rosario Group, the coastal margin was uplifted and eroded until the middle Eocene. Subsequently, several major transgressive-regressive cycles led to the deposition of nine (9) partially intertonguing middle and upper Eocene sequences. Following deposition of the Eocene sediments, the margin was subjected to tectonic uplift and dissection until the Oligocene when continental and shallow marine sediments of the Otay Formation were deposited. Following the Oligocene time, the coastal margin was uplifted causing considerable erosion. In the Pliocene, major marine transgression resulted in the deposition of marine sandstone and transitional marine and continental pebble and cobble conglomerate, belonging to the San Diego Formation.

After the deposition of the San Diego Formation and continuing to present day, the coastal margin has experienced relatively steady uplift. This has resulted in the emergence of continually evolving marine terraces. Dissection of these terraces has led to the accumulation of alluvial deposits within the major drainage courses and beach deposits along the shoreline areas.

Regional geologic mapping by Kennedy and Tan (2008) indicates that most of the subject parcels are immediately underlain by Quaternary-age old paralic deposits (unit 6). These deposits were formerly referred to as the Quaternary-age Bay Point Formation on the previous regional geologic map prepared by Kennedy (1975). Both Kennedy (1975) and Kennedy and Tan (2008) show Cretaceous-age sedimentary bedrock, belonging to the

Point Loma Formation, underlying the aforementioned surficial deposits and close to the surface near the northwestern margin of Lot 2, and the southeastern boundary of Lot 17. Kennedy and Tan (2008) describe the old paralic deposits as, "poorly sorted, moderately permeable, reddish-brown, interfingered strandline, beach, estuarine and colluvial deposits composed of siltstone, sandstone, and conglomerate" that were deposited on the approximately 120,000-year old Nestor Terrace. Kennedy and Tan (2008) characterize the Point Loma Formation as, "Interbedded, fine-grained, dusky-yellow sandstone and olive-gray siltstone."

ONSITE GEOLOGIC UNITS

The onsite geologic units that were encountered in the exploratory borings included a Quaternary-age residual soil at the surface that was imprinted upon the Quaternary-age old paralic deposits. Although not encountered in the borings nor observed during our onsite mapping, based on our observations of outcrops along the nearby coastal bluff, our reviews of Kennedy (1975) and Kennedy and Tan (2008), and oblique aerial photographs obtained from the "California Coastal Records Project" (www.californiacoastline.org), as well as our past work experience with nearby sites on Coast Walk and Torrey Pines Road (Appendix A), the Cretaceous-age Point Loma Formation underlies the old paralic deposits. Undocumented artificial fill from the nearby Coast Walk roadway embankment may also encroach into the northwest corner of APN 350-131-02-00. These earth materials are further described below. The general distribution of the geologic units across the subject parcels and adjacent coastal bluff are shown in plan view on Plate 1 and in profile view on Plates 2 and 3.

Undocumented Artificial Fill (Map Symbol - Afu)

Based on our surficial observations and mapping, undocumented artificial fill may occur at the surface near the northwestern property corner. The fill may be associated with the construction of the nearby embankment for the Coast Walk roadway. Although not explored, the undocumented fill likely consists of a mixture of silty sand, clayey sand, sand, and clay with rounded gravels derived from the near-surface geologic units in the surrounding area.

Quaternary Residual Soil (Not Mapped)

A Quaternary-age residual soil (colluvium) was encountered at the surface in the borings. It extended to depths of approximately1 foot to 2 feet below the existing grades. The residual soil in the borings consisted of light-olive-brown silty sand that was damp and dense. The residual soil contained traces of gravel, locally.

Quaternary Old Paralic Deposits (Map Symbol - Qop)

Quaternary-age old paralic deposits were encountered in the borings at depths ranging between approximately 1 foot and 2 feet below the existing grades. As observed therein, the old paralic deposits consisted of interbedded light-olive-brown, dark-olive-brown, and olive-gray clayey sand; olive-brown, light-olive-brown, olive-gray, and reddish-yellow sand; and olive-brown, light-olive-brown, olive-gray, reddish-yellow, and medium gray clay, and contained trace rounded gravel, locally. The old paralic deposits were generally damp to moist but became wet to saturated at an approximate depth of 15 feet below the existing grade. The old paralic deposits contained rounded gravels, locally. Based on data obtained from previous subsurface exploration on a nearby property (GSI, 2015), we infer that the old paralic deposits may be a paleo-talus deposit along a relict seacliff/shore platform junction when sea level was higher than at present.

Cretaceous Point Loma Formation (Map Symbol - Kp)

As previously stated, the Cretaceous-age Point Loma Formation underlies the old paralic deposits. GSI's observations of the Point Loma Formation in a large-diameter boring we advanced and logged, as part of an investigation of a nearby property (GSI, 2015), indicated that this unit generally consisted of interbedded grayish-brown, dark-gray, and olive-brown sandy claystone and sandstone with trace concretions. Based our review of oblique aerial photographs, our past work experience within a nearby property, and assuming a relatively gentle seaward inclination of the geologic contact between the old paralic deposits and the Point Loma Formation (approximately 2 degrees from the horizontal plane), we estimate that the Point Loma Formation generally occurs between approximate elevations 62 and 71 feet above MSL throughout most of the project area (see Plates 2 and 3). However, we infer that it may be within a couple of feet of the ground surface near the southeastern property boundary of Lot 17, coincident with a relict coastal bluff associated with a higher sea level stand during the Pleistocene.

GEOLOGIC STRUCTURE

The site is situated upon the faulted northeastern limb of a northwest-trending syncline that Kennedy (1975) refers to as the Pacific Beach Syncline. Kennedy (1975) and Kennedy and Tan (2008) show the northwest-trending Country Club fault to the west of the subject site and a shorter subsidiary, northwest-trending fault to the east of the site. Oblique aerial photographs clearly show that Point Loma Formation beds, exposed in the coastal bluff, are inclined toward the southwest at moderate angles. Kennedy (1975) indicates Point Loma Formation beds inclined 35 to 40 degrees toward the southwest in the vicinity of the parcels. Whereas, Kennedy and Tan (2008) show Point Loma Formation beds inclined 20 to 30 degrees to the southwest in the site vicinity. However, Kennedy and Tan (2008) do not map these bedding attitudes within the fault-bounded block upon which the parcels are located. During subsurface investigation on a nearby property on Coast Walk, performed in preparation of GSI (2015), GSI recorded Point Loma Formation beds inclined approximately 23 to 29 degrees toward the southwest. Bedding orientation is not considered adverse relative to the deep-seated stability of the nearby coastal bluff.

GROUNDWATER

GSI encountered perched groundwater seepage in Boring B-2 at an approximate depth of 19 feet below the existing grade or approximately 65 feet above MSL. The seepage is likely the result of groundwater accumulating near the geologic contact between the old paralic deposits and the Point Loma Formation, owing to the contrasting permeabilities of these units. Regional groundwater is anticipated to occur near sea level and likely fluctuates a few feet with the tides.

Based on our understanding of the proposed site development, groundwater is not anticipated to be a significant geotechnical factor. However, our findings and conclusions regarding groundwater reflect the site conditions at the time of our recent subsurface exploration and do not preclude future changes in local groundwater conditions from meteorological or climatic factors, excessive irrigation, damaged underground utilities, or other circumstances that were not obvious during our field exploration.

Due to the nature of the onsite earth materials, seepage or perched groundwater conditions may develop in other locations and elevations within the subject property, both during and following the proposed development. Perched groundwater is likely to occur along boundaries of contrasting permeabilities and densities (i.e., sandy/clayey fill lifts, geologic contacts, bedding, discontinuities, weathered/unweathered zones, etc.), and should be anticipated. This potential should be disclosed to all interested/affected parties. Should perched groundwater be encountered, this office can evaluate the conditions and provide recommendations for mitigation. Mitigation commonly consists of the installation of subdrains and cut-off barriers.

ONSITE SOILS AND STORM WATER INFILTRATION FEASIBILITY

According to the United States Department of Agriculture / Natural Resources Conservation Service's (USDA/NRCS's) "Web Soil Survey" website (http://websoilsurvey.sc.egov. usda.gov), the onsite soils consist of Urban land. Due to disturbance from urbanization, the attributes of this soil unit are unknown.

Based on the findings from our subsurface exploration and laboratory testing, it is the opinion of GSI that the infiltration of storm water into the onsite earth materials for permanent post-construction storm water best management practices (BMPs) has a high potential to accumulate along sand and clay beds within the old paralic deposits and along the geologic contact between the old paralic deposits and the Point Loma Formation, resulting in perched groundwater (groundwater mounding). Perched groundwater would likely migrate laterally and enter the adjacent properties, and seep from the nearby coastal bluff, owing to the seaward-dipping geologic contact between the old paralic deposits and the Point Loma Formation. The lateral migration of perched groundwater could induce swelling of expansive soils and fill settlement within the subject property and the adjacent parcels. Perched groundwater exiting the bluff face would also contribute to spring sapping and reduced bluff stability. Lastly, the proposed project includes numerous retaining walls. Lateral migration of perched groundwater could increase moisture transmission through these walls. Thus, the infiltration of storm water into the onsite earth materials for permanent post-construction storm water BMPs is <u>not</u> considered sound engineering practice and is <u>not</u> recommended from a geotechnical perspective.

GEOLOGIC HAZARDS EVALUATION

GSI has evaluated the parcels relative to geologic and seismic hazards that could affect the proposed development. According to the "City of San Diego Seismic Safety Study - Geologic Hazards and Faults" (City of San Diego Development Services Department [SDDSD], 2008), much of the subject parcels are located within Geologic Hazard Category 53. Geologic Hazard Category 53 includes sites with level or sloping terrain with unfavorable geologic structure that present low to moderate risk for development. The northwestern, approximately one-half of Lot 17 lies within Geologic Hazard Category 43, which includes generally unstable coastal bluffs due to unfavorable jointing and local high erosion rates.

Mass Wasting/Landslide Susceptibility

Mass wasting refers to the various processes by which earth materials are moved down slope in response to the force of gravity. Examples of these processes include slope creep, surficial failures, and deep-seated landslides. Creep is the slowest form of mass wasting and generally involves the outer 5 to 10 feet of a slope surface. During heavy rains, such as those in El Niño years, creep-affected materials may become saturated, resulting in a more rapid form of downslope movement (i.e., landslides or surficial failures).

According to regional landslide susceptibility mapping by Tan (1995), the subject parcels are located within Relative Landslide Susceptibility Subareas 3-1and 4-1. Subarea 3-1 is characterized as being "generally susceptible" to landsliding due to a combination of weak earth materials and steep slopes. Tan (1995) indicates that although most slopes in Subarea 3-1, do not currently contain landslides, localized slope failures can be expected when slopes are adversely altered. Tan (1995) describes Subarea 4-1 as being most susceptible to landslides. Tan (1995) states that sites within Subarea 4-1 are generally located outside the boundaries of definite mapped landslides but contain visibly unstable slopes underlain by both weak materials and adverse geologic structure. Sites within Subarea 4-1 also contains inferred landslides and oversteepened high coastal bluffs subject to active marine erosion.

Our review of regional geologic mapping by Tan (1995), Kennedy (1975), and Kennedy and Tan (2008) did not reveal the presence of landslides within the subject parcels. In addition, we did not observe evidence of landslides or deep-seated instability within the parcels during our field investigation. Moreover, geomorphic features indicative of past mass wasting events (i.e., scarps, hummocky terrain, debris cones, arcuate drainage patterns, etc.) were not identified within the subject parcels during our review of stereoscopic aerial photographs (Fairchild Aerial Surveys, 1952; Park Aerial Surveys, 1953). The coastal bluff seaward of Lot 2 (APN 350-131-02-00) is not actively retreating due to marine erosion, but in the historic past, uncontrolled irrigation and runoff has caused some subaerial erosion. To that end, GSI has evaluated coastal bluff retreat rates and global stability over the 75-year design life of the proposed development, as discussed later in this report.

The onsite soils are considered erodible. Properly designed and regularly maintained surface drainage is recommended to mitigate erosion.

Subsidence

The subject parcels are not located in an area of known subsidence associated with fluid withdrawal (groundwater or petroleum); therefore, the potential for subsidence due to the extraction of fluids is considered negligible.

Hydrocollapse / Hydroconsolidation

The subject parcels are generally underlain by geologic units that are not considered susceptible to hydrocollapse and hydroconsolidation. In addition, the residual soil and any undocumented fill will either be removed during the planned excavations or improved by the recommended remedial grading. Thus, the potential for the proposed development to experience significant settlements due to hydrocollapse or hydroconsolidation is considered negligible.

FAULTING AND REGIONAL SEISMICITY

Regional Faults

Our review indicates that there are no known Holocene-active faults (i.e., faults that have ruptured in the last 11,700 years) crossing the subject parcels (Jennings and Bryant, 2010; SDDSD, 2008), and the site is not located within an Alquist-Priolo Earthquake Fault Zone (California Department of Conservation, California Geological Survey [CGS], 2018). However, the site is situated in a region subject to periodic earthquakes along Holocene-active faults. The Rose Canyon fault (part of the Newport-Inglewood - Rose Canyon fault zone [NIRCFZ]) is the closest known Holocene-active fault to the site, located at a distance of approximately 0.39 miles (0.62 kilometers) to the northeast. This fault should have the greatest effect on the site in the form of strong ground shaking, should the design earthquake occur. Cao, et al. (2003) indicate the slip rate on the Rose Canyon fault is 1.5 (±0.5) millimeters per year (mm/yr) and the fault is capable of a maximum magnitude 7.2 earthquake. The location of the Rose Canyon fault and other major faults within 100 kilometers of the site are shown on the "California Fault Map" in Appendix C. The possibility of ground acceleration, or shaking at the site, may be considered as approximately similar to the southern California region as a whole.

Local Faulting

A review of available regional geologic maps (Treiman, 1993; Kennedy, 1975; Kennedy and Tan, 2008; Jennings and Bryant, 2010; CGS, 2018) and SDDSD (2006) did not indicate the presence of faults, Holocene-active or otherwise, crossing the subject parcels.

Surface Rupture

Owing to the lack of known Holocene-active faults crossing the subject parcels, the potential for the proposed development to be adversely affected by surface rupture from fault displacement is considered low.

Seismicity

The acceleration-attenuation relation of Bozorgnia, Campbell, and Niazi (1999) has been incorporated into the computer program EQFAULT, developed by Thomas F. Blake (Blake, 2000a). EQFAULT performs deterministic seismic hazard analyses using digitized California faults as earthquake sources.

The program estimates the closest distance between each fault and a given site. If a fault is found to be within a user-selected radius, the program estimates peak horizontal ground acceleration that may occur at the site from an upper bound (formerly "maximum credible earthquake"), on that fault. Upper bound refers to the maximum expected ground acceleration produced from a given fault. Site acceleration (g) was computed by one user-selected acceleration-attenuation relation that is contained in EQFAULT. Based on the EQFAULT program, a peak horizontal ground acceleration from an upper bound event on the Rose Canyon fault may be on the order of 0.83 g. The computer printouts of pertinent portions of the EQFAULT program are included within Appendix C.

Historical site seismicity was evaluated with the acceleration-attenuation relation of Bozorgnia, Campbell, and Niazi (1999), and the computer program EQSEARCH (Blake, 2000b, updated to May 8, 2021). This program performs a search of the historical earthquake records for magnitude 5.0 to 9.0 seismic events within a 100-kilometer radius, between the years 1800 through May 8, 2021. Based on the selected acceleration-attenuation relationship, a peak horizontal ground acceleration is estimated, which may have affected the site during the specific time frame. Based on the available data and the attenuation relationship used, the estimated maximum (peak) site acceleration during the period 1800 through May 8, 2021 was about 0.25 g. A historic earthquake epicenter map and a seismic recurrence curve was also estimated/generated from the historical data. Computer printouts of the EQSEARCH program are presented in Appendix C.

Seismic Shaking Parameters

The following table summarizes the site-specific seismic design criteria obtained from the 2019 CBC, Chapter 16 Structural Design, Section 1613, Earthquake Loads (CBSC, 2019) and American Society of Civil Engineers (ASCE 7-16 [ASCE, 2017]). The computer program Seismic Design Maps, provided by the California Office of Statewide Health Planning and Development (OSHPD) and the Structural Engineers Association of California (SEAOC) has been used to aid in design (https://seismicmaps.org). The short spectral response uses a period of 0.2 seconds. Based on the findings from our onsite subsurface exploration and our past work experience with a nearby site underlain by similar geologic conditions (GSI, 2015), it is our opinion that Site Class "C" conditions are applicable to the proposed development.

2019 CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	**VALUE per OSHPD/SEAOC SEISMIC DESIGN MAPS**	**2019 CBC or REFERENCE**
Risk Category*	I, II, or III	Table 1604.5
Site Class	C	Section 1613.2.2/Chap. 20 ASCE 7-16 (p. 203-204)
Spectral Response - (0.2 sec), S_s	1.386 g	Section 1613.2.1 Figure 1613.2.1(1)
Spectral Response - (1 sec), S_1	0.485 g	Section 1613.2.1 Figure 1613.2.1(2)
Site Coefficient, F_a	1.2	Table 1613.2.3(1)
Site Coefficient, F_v	1.5	Table 1613.2.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.663 g	Section 1613.2.3 (Eqn 16-36)
Maximum Considered Earthquake Spectral Response Acceleration (1 sec),S_{M1}	0.728 g	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	1.109 g	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.485 g	Section 1613.2.4 (Eqn 16-39)

2019 CBC SEISMIC DESIGN PARAMETERS		
PARAMETER	VALUE per OSHPD/SEAOC SEISMIC DESIGN MAPS	2019 CBC or REFERENCE
PGA_M - Probabilistic Vertical Ground Acceleration may be assumed as about 50% of this value.	0.758 g	ASCE 7-16 (Eqn 11.8.1)
Seismic Design Category	D	Section 1613.2.5/ASCE 7-16 (p. 85: Table 11.6-1 or 11.6-2)
* - Risk Category to be confirmed by the project architect.		

SECONDARY SEISMIC HAZARDS

Liquefaction/Lateral Spreading

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in relatively cohesionless soils. These soils may thereby acquire a high degree of mobility, which can lead to vertical deformation, lateral movement, lurching, sliding, and as a result of seismic loading, volumetric strain and manifestation in surface settlement of loose sediments, sand boils and other damaging lateral deformations. This phenomenon occurs only below the water table, but after liquefaction has developed, it can propagate upward into overlying non-saturated soil as excess pore water dissipates.

One of the primary factors controlling the potential for liquefaction is the depth to groundwater. Typically, liquefaction has a relatively low potential at depths greater than 50 feet and is unlikely or will produce vertical strains well below 1 percent at depths below 60 feet when relative densities are 40 to 60 percent and effective overburden pressures are two or more atmospheres (i.e., 4,232 pounds per square foot [Seed, 2005]).

The condition of liquefaction has two principal effects. One is the consolidation of loose sediments with resultant settlement of the ground surface. The other effect is lateral sliding. Significant permanent lateral movement generally occurs only when there is significant differential loading, such as fill or natural ground slopes within susceptible materials. No such loading conditions exist at the site.

Liquefaction susceptibility is related to numerous factors and the following five conditions should be concurrently present for liquefaction to occur: 1) sediments must be relatively young in age and not have developed a large amount of cementation; 2) sediments must generally consist of fine- to medium-grained, relatively cohesionless sands; 3) the sediments must have low relative density; 4) free groundwater must be present in the sediment; and 5) the site must experience a seismic event of a sufficient duration and magnitude, to induce straining of soil particles. Only about one to perhaps two of these five necessary conditions have the potential to affect the site, concurrently.

Summary

It is the opinion of GSI that the susceptibility of the proposed project area to experience damaging deformations from seismically-induced liquefaction and lateral spreading is relatively low owing to the dense/hard nature of the old paralic deposits and Point Loma Formation that underlie the site in the near-surface and the depth to the regional groundwater table. In addition, our recommendations for remedial earthwork and foundation design, and construction would further mitigate liquefaction/lateral spread potential.

Tsunami

Tsunami are a series of waves caused by a rapid displacement of water volume within a body of water. This accelerated change in volume can be caused by displacement of the seafloor due to faulting or other factors such as volcanic eruptions, landslides, glacier calving, meteorite impacts, and underwater explosions. According to tsunami inundation mapping by the California Emergency Management Agency, et al. (2009), the subject parcels are not located within a tsunami inundation zone. Thus, the proposed development is at low risk for tsunami inundation. However, the coastal bluff seaward of Lot 2 (APN 350-131-02-00) is located within a tsunami inundation zone, and could experience some erosion from a tsunami impact.

Historical records indicate the frequency of tsunami reaching the San Diego County coastline is relatively low and the height of historical tsunami have been within the normal tidal range. Thus, effects from a tsunami would be generally similar to those created by storm waves.

Other Geologic/Secondary Seismic Hazards

The following list includes other geologic/seismic related hazards that have been considered during our evaluation of the site. The hazards listed are considered negligible or mitigated as a result of site location, soil characteristics, and typical site development procedures:

- Coseismic deformation (ground lurching or shallow ground rupture)
- Seiche

COASTAL BLUFF GEOMORPHOLOGY

The typical profile of coastal bluffs may be divided into three zones: 1) the shore platform; 2) a lower near-vertical cliff surface, termed the seacliff; and 3) an upper-bluff slope generally ranging in inclination between about 20 and 80 degrees or more (measured from the horizontal plane). The bluff top or bluff edge is the boundary between the upper bluff and the relatively flat-lying to moderately sloping coastal terrace. The coastal bluff adjacent to APN 350-131-02-00 is generally consistent with the previously described typical bluff profile.

Offshore from the seacliff is an area of indefinite extent termed the near-shore zone. The bedrock surface in the near-shore zone, which extends out to sea from the base of the seacliff, is the shore platform. As pointed out by Trenhaile (1987), worldwide, the shore platform may vary in inclination from near horizontal to as steep as 3:1 (h:v). The boundary between the seacliff and the shore platform is called the cliff-platform junction, or sometimes the shoreline angle. Within the near-shore zone, is a subdivision called the inshore zone, where the waves begin to break. This boundary varies with time because the point at which waves begin to break changes dramatically with changes in wave size and tidal level. During low tides, large waves will begin to break further away from shore. During high tides, waves may not break at all, or they may break directly on the lower seacliff. Closer to shore is the foreshore zone, or the portion of the shore lying between the upper limit of wave wash at high tide and the ordinary low water mark. Both of these boundaries often lie on a sand or cobble beach. The foreshore zone in the vicinity of the subject parcels extends from low water to the lower face of the bluff. The La Jolla Submarine Canyon lies directly offshore and influences the wave environment.

Emery and Kuhn (1982) developed a global system of classification of coastal bluff profiles, and applied that system to the San Diego County coastline from San Onofre State Park to the southerly tip of Point Loma. Emery and Kuhn (1982) designated the La Jolla coastline adjacent to the subject parcels as "active" with an A-a classification. The letter "A" designates coastal bluffs generally composed of homogenous earth materials. The relative effectiveness of marine erosion compared to subaerial erosion of the bluff produces a characteristic bluff profile. The letter "a" indicates that the rate of marine erosion is much greater than the rate of subaerial erosion. Based on our observations, we classify the coastal bluff as C-b to C-c. The letter "C" describes a coastal bluff that is composed of a more resistant geologic unit along its base, with a more erodible unit in the upper-bluff portion. The letters "b" or "c" indicate that the rate of marine erosion is more than the rate of subaerial erosion, or is equal to the rate of subaerial erosion, respectively.

Marine Erosion

The factors contributing to "Marine Erosion" processes are described below:

Mechanical and Biological Processes

Mechanical erosion processes at the cliff-platform junction include water abrasion, rock abrasion, cavitation, water hammer, air compression in joints/fractures, breaking-wave shock, and alternation of hydrostatic pressure with the waves and tides. All of these processes are active in backwearing. Downwearing processes include all but breaking-wave shock (Trenhaile, 1987). Backwearing and downwearing, by the mechanical processes described above, are both augmented by bioerosion, the removal of rock by the direct action of organisms (Trenhaile, 1987). Backwearing is assisted by algae in the intertidal and splash zones and by rock-boring mollusks in the tidal range. Algae and associated small organisms bore into rock up to several millimeters. Mollusks may bore several centimeters into the rock. Chemical and salt weathering also contribute to the erosion process.

Water Depth, Wave Height, and Platform Slope

The key factors affecting the marine erosion component of bluff retreat are water depth at the base of the cliff, breaking wave height, and the slope of the shore platform. Along the entire coastline, unarmored seacliffs are subject to periodic attack by breaking and broken waves, which create the dynamic effects of turbulent water and the compression of entrapped air pockets. When acting upon a jointed and fractured seacliff, the "water-hammer" effect tends to cause hydraulic fracturing, which exacerbates seacliff erosion. Erosion associated with breaking waves is most active when water depths at the cliff-platform junction coincide with the respective critical incoming wave height, such that the water depth is approximately equal to 1.3 times the wave-height.

Marine Erosion at the Cliff-Platform Junction

The cliff-platform junction contribution to retreat of the overall seacliff is from marine erosion, which includes mechanical, chemical, and biological erosion processes. Marine erosion operates horizontally (backwearing) on the cliff as far up as the top of the splash zone, and vertically (downwearing) on the shore platform (Emery and Kuhn, 1980; Trenhaile, 1987). Backwearing and downwearing typically progress at rates that will maintain the existing gradient of the shore platform.

Subaerial Erosion

"Subaerial Erosion" processes are discussed as follows:

Groundwater

The primary erosive effect of groundwater seepage on the formational materials within the coastal bluff is by spring sapping, or the mechanical erosion of individual grains by groundwater exiting the bluff face. Chemical solution is also a significant contributor, especially on carbonate matrix material. Groundwater approaching the bluff face typically infiltrates near-surface, stress-relief, bluff-parallel joints/fractures. Hydrostatic loading of joints/fractures near the bluff face is an important cause of block-toppling on steep-cliffed lower bluffs, especially where basal notching is present (Kuhn and Shepard, 1980). There was no evidence of groundwater seeping from the face of the coastal bluff in the oblique aerial photographs reviewed (www.californiacoastline.org).

Slope Decline

The process of slope decline typically consists of a series of steps, which ultimately cause the bluff to retreat. The base of the bluff is first weakened by wave attack and the development of wave cut nips or sea caves. As the weakened seacliff fails by blockfall or rockfall, an over-steepened bluff face is left, with the debris at the toe of the seacliff. Ultimately, the rockfall/blockfall debris is removed by wave action, and the marginal support for the upper bluff is thereby removed. Progressive surficial slumping and failure of the bluff will occur until a condition approaching the angle of repose is established over time, and the process begins anew.

Surface Drainage

Uncontrolled concentrated surface drainage can result in significant upper-bluff erosion. These "top down" type bluff failures are characterized by small "V"-shaped erosional gullies, a few feet across, that extend down the bluff face but terminate above the wave runup line. Gullies are present in bluff northeast and southwest of the subject site.

HISTORIC COASTAL-BLUFF RETREAT

Most of San Diego County's coastline has experienced a measurable amount of erosion in the last 110 years or so, with more rapid erosion occurring during periods of heavy storm surf (Kuhn and Shepard, 1984). The seacliff portions of the coastal bluffs are exposed to direct wave attack along most of the coast. The waves commonly erode the seacliff by impact on bedding planes and small joints/fractures, and fissures in the bedrock units, and by water-hammer effects. The upper bluffs, which often support little or no vegetation, are subject to wave spray and splash, sometimes causing saturation of the outer layer and subsequent sloughing of over-steepened slopes. Wind, rain, irrigation, and uncontrolled surface runoff contribute to the subaerial erosion of the upper coastal bluff, especially on the more exposed over-steepened portions.

Historic Coastal Bluff Retreat Summary

Numerous studies have been undertaken to analyze coastal bluff retreat along the San Diego coastline. However, the most in-depth study to date consists of a 1999 assessment by Benumof and Griggs (1999). This study presents erosion rates for coastal bluffs in different sections of the San Diego County coastline. The erosion rates published by these workers were obtained by analyzing a combination of factors including overall rock mass strengths, obtained through Schimdt Hammer testing; visual assessments of discontinuity orientation, spacing, width, and infilling; earth material weathering and fatigue; groundwater seepage; and wave impact at the seacliff. These data were compared to the historical bluff edge locations observed in ortho-corrected aerial photographs of the coast for the years 1932, 1949, 1952, and 1956. A 1994 aerial photograph served as the base imagery for the entire coastline.

For the La Jolla coast section, which began south of La Jolla Cove beach and terminated near the Children's Pool, Benumof and Griggs (1999) arrived at a mean bluff recession rate of 3.06 centimeters per year (cm/yr), which equates to approximately 1.20 inches per year (in/yr) or approximately 0.10 feet per year (ft/yr), with a standard deviation of 1.5 cm/yr (0.59 in/yr or 0.049 ft/yr). Their findings indicated an approximate retreat rate of 0.1 ft/yr for this reach. This rate is similar to the site-specific historical retreat rate we obtained (see below), and in contrast to the upper bound rate of bluff retreat for the entire La Jolla study area of 7 cm/yr (2.76 in/yr), or 0.23 ft/yr, concluded by Benumof and Griggs (1999)

Hapke and Reid (2007), provided rates of coastal bluff retreat along much of the California coast in their publication, "Historical Coastal Cliff Retreat Along the California Coast." That study extended from the border between California and Oregon to San Diego Harbor. These workers determined a rate of <u>less than</u> 0.04 meters/year of marine erosion (1.05 inches/year or 0.088 feet/year), for the subject site.

In order to evaluate the historical retreat of the coastal bluff fronting the subject parcels, GSI reviewed stereoscopic aerial photographs taken in 1953 (Park Aerial Surveys, Inc., 1953) and oblique aerial photographs for the years 1972, 1979, 1989, 2002, 2004, 2006, 2008, 2010, and 2013 taken by Kenneth and Gabrielle Adelman and catalogued on the "California Coastal Records Project" website (www.californiacoastline.org). The coastal bluff edge in the photographs were compared to the position of the Coast Walk trail and roadway, which were present in all of the photographs. Based on our aerial photograph review, we estimate that approximately 5 feet of bluff edge retreat has occurred since 1953, a period 69 years. This equates to a retreat rate of approximately 2.2 cm/yr (roughly 0.87 in/yr or 0.07 ft/yr). This rate is similar to the mean rate obtained by Benumof and Griggs (1999) for their La Jolla study area and the site-specific rate concluded by Hapke and Reid (2007).

It is our opinion that the low recession rate of the coastal bluff, fronting the subject site, is attributed to the resistant Point Loma Formation in the seacliff. In addition, the broad shore platform and the protective head of (entrance to) the La Jolla Submarine Canyon, located immediately offshore, shield the coastal bluff from most direct wave impact. The shallow tributary valleys of the La Jolla Submarine Canyon are located very close to the shoreline, in water only about 30 feet deep. The tributary valleys intersect the canyon at an axial depth of approximately 60 feet, about 2,000 feet from its head. In addition, failed blocks of Point Loma Formation, over time, has formed a berm along the shoreline, seaward of the bluff toe. This berm serves as a natural revetment, dissipating wave energy before it impacts the coastal bluff. Accordingly, since the bluff is located in a relatively passive cove and protected by a broad shore platform, a submarine canyon, and a berm of failed blocks of Point Loma Formation, GSI concludes that the historical retreat rate of 0.088 ft/yr, determined by Hapke and Reid (2007), is reasonable and the best available science in this regard. This rate also corroborates our historical aerial photograph reviews that show very little bluff edge retreat since 1953, or over the last 69 years.

LONG-TERM SEA LEVEL CHANGE

Long-term (geologic) sea level change is the major factor determining coastal evolution (Emery and Aubrey, 1991). Three general sea level conditions have been recognized: rising (typically interglacial), falling (typically glacial), and stationary (although of a transient nature). The rising and falling stages result in massive sediment release and transport, while the stationary stage allows time for adjustment and reorganization toward equilibrium. Overall, planet Earth has experienced a long decline in temperatures. Beginning 3.5 million years ago, a series of 45 ice ages began. This long period of increasing cold initiated with ice ages occurring on a 41,000-year cycle and included 33 separate glacial events. For the last 1.25 million years, we have been in a more severe 100,000-year cycle, in which glaciations occurred during 13 ice ages typically lasting 90,000 years, with interglacial warm periods lasting about 10,000 years (Carter, 2011). It is intuitively obvious that the warming and cooling of the Earth have natural causes (Milankovitch cycles, solar insolation cycles, etc), and those natural sources did not suddenly halt at the start of the Industrial Revolution, when it is theorized anthropogenic activities began influencing atmospheric carbon dioxide levels (Wrightstone, 2017).

Major changes in sea level of the Quaternary period were caused by worldwide climate fluctuation resulting in at least 17 glacial and interglacial stages in the last 800,000 years and many before then (Shakelton and Opdyke,1976), as indicated in Figure 2. Figure 2 shows that each of the last interglacial warming periods (as we are in today), was significantly warmer than our current temperature (Jouzel and Masson-Delmotte, 2007; Wrightstone, 2017).

Worldwide sea level rise associated with the melting of continental glaciers is commonly referred to as "glacio-eustatic" or "true" sea level rise. During the past 200,000 years, eustatic sea level has ranged from more than approximately 350 feet below to possibly as high as about 31 feet above the present level . The latter suggests it was <u>hotter</u> at that time <u>than</u> it is <u>now</u>.

Tectonic activity can also account for significant relative changes in sea level on a local scale. Past movement along the Rose Canyon fault zone and associated faults, which served to uplift Mount Soledad and formed Point La Jolla, also created a zone of structural weakness along which the La Jolla Submarine Canyon has been incised. The Torrey Pines block, with its relatively horizontally stratified Eocene-age formations and wave-cut terraces, has experienced more than 450 feet of tectonic uplift in the last 2 million years, while the tilted and uplifted Soledad Mountain block has undergone more than 750 feet of tectonic uplift in the same period (Kern, 1977).

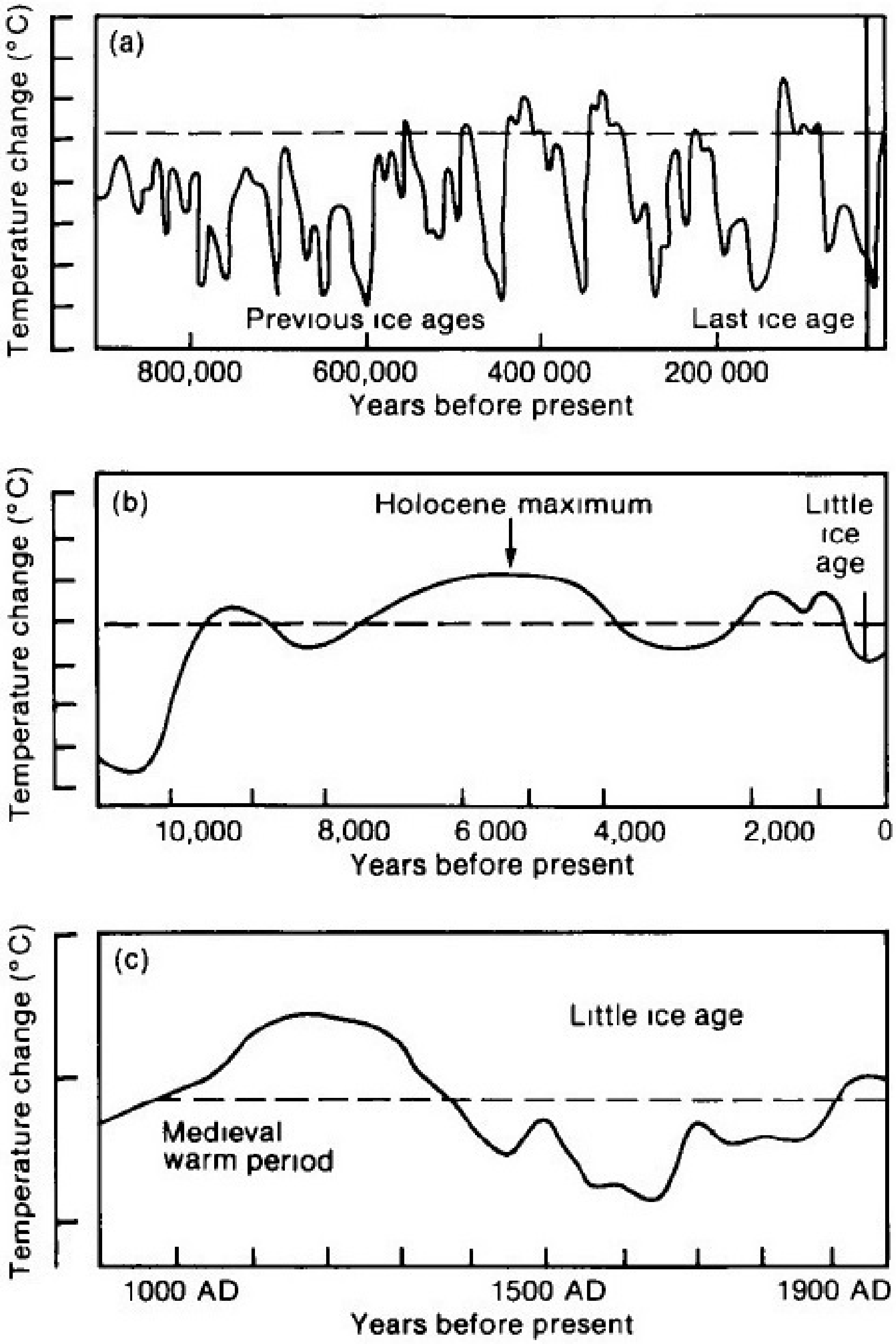


Figure 2 (from Figure 7.1 [IPCC, 1990]): Schematic diagrams of global temperature variations since the Pleistocene on three time scales (a) the last million years (b) the last ten thousand years and © the last thousand years. The dotted line nominally represents conditions near the beginning of the twentieth century.

Sea level changes during the last 20,000 years or so have resulted in an approximately 350-foot rise in sea level when relatively cold global climates of the Wisconsin ice age started to become warmer; melting a substantial portion of the continental ice caps (Curray, 1960 and 1961; CLIMAP, 1976). As shown in Figure 2, following the peak of the Last Glacial Maximum (LGM) about 18,000 to 20,000 years ago, Earth entered the present interglacial warm period (which usually last 10,000 to 15,000 years [the current one is about 11,000 years old [Wrightstone, 2017]). Interestingly, during the last 10,000 years, there have been at least 10 significant instances of sea level rise (SLR) and fall. Contrary to popular belief, both the rate of SLR and the associated global temperature were greater during those events, than the late 20th century period of SLR (Ally, 2004), which has been cited as "unprecedented" in order to justify political agendas. Global sea level rose very rapidly at rates as high as 50 millimeters per year (mm/yr) or about 1.97 in/yr, with a mean rate of about 10 mm/yr (0.39 in/yr) between the Late Pleistocene (about 15,000 years ago) and mid-Holocene time.

About 7,500 to 6,000 years ago, sea level was roughly 1 to 7.2 meters (approximately 3.2 to 23.6 feet) above the current level (Hein, et al., 2014; Yu, et al., 2007), and has since fallen, and risen to a lesser degree, but has never remained static for long periods. During the past 3,000 to 2,000 years, the rate appears to have fluctuated and slowed, haltingly, to approximately 0.1 to 0.2 mm/yr (Intergovernmental Panel on Climate Change [IPCC], 2001). The National Academy of Sciences (National Research Council, 2012) indicates that in the 20th century, SLR was about 1.7 mm/yr (0.067 in/yr), and has concluded that from about 1992 to 2012, SLR had increased to about 3.1 mm/yr (approximately 0.12 in/yr), requiring increases of 3 to 4 times the current rate needed to realize a scenario of 1 meter (3.2 feet) of SLR by 2100.

It is estimated that sea level in La Jolla rose approximately 0.67 feet over the past century, where annual mean sea levels were measured at the La Jolla tide gauge, starting in 1925 (tidesandcurrents.noaa.gov.sltrends.sltrends.html). As indicated above, for about 60% of the current interglacial warming period, it was warmer then than it is today (see Figure 2 [IPCC, 1990; Ally, 2004; Box, et al., 2009; and Wrightstone, 2017]). Again, contrary to popular belief, the earth has been in a warming trend for approximately the last 350 years (see Figure 2 [from IPCC, 1990]), commencing about 100 years (~1650 AD) before the Industrial Revolution (~1750 AD).

FUTURE SEA LEVEL RISE

Currently, there is a wide range of predicted rates in SLR over the next century, from several inches to over 14 feet. This wide range makes it extremely difficult for the design of coastal development. The amount and magnitude of SLR is not settled scientifically (see Nerem, 2005; Nerem, et al., 2006, Nerem, et al., 2018; Wrightstone, 2017), has a wide field of uncertainty at the 2100- to 2150-year end range, and is driven by the variables in the model selected.

In 2006, the California Climate Change Center produced a "white paper" entitled "Projecting Future Sea Level" (Cayan et al., 2006). The purpose of that report was not to set a development standard, but rather to play out a range of scenarios of sea level rise and discuss potential impacts. The paper reports that sea level along the west coast of the United States has been rising at a rate of about 0.08 inches/year in the last century. The authors of the white paper refined their work and produced a scientific paper in 2008 entitled "Climate Change Projections of Sea Level Extremes Along the California Coast." This paper provides a range in sea level rise from 11 cm (4.3 in) to 72 cm (28 in) over the next 100 years. Even though there is no scientific consensus (Wrightstone, 2017), modeling of future climates drives a change in the calculated rate of sea level rise.

With regard to sea level rise for coastal engineers, Chapter 5 of the 2009 United States Army Corps of Engineers (USACE) "Coastal Engineering Manual" (CEM) provides an extensive discussion of water levels used for design. A summary of the CEM conclusions regarding sea level rise and climate change are reproduced below:

- The primary conclusion was, with some regional exceptions, sea level is not rising at a rate to cause undue concern. Results of the report indicate an average sea level rise over the past century of approximately 30 cm/century on the United States (US) east coast, and 11 cm/century on the US west coast, and a range along the US Gulf of Mexico coast of less than 20 cm/century for the west coast of Florida to more than 100 cm/century in parts of the Mississippi delta plain.

- The USACE uses a 4.3-inch (11 cm) rise for the US west coast sea level over the next 100 years.

More detailed planning and engineering policy in 2011 was followed by the release of the current guidance, USACE (2013), that requires consideration of three scenarios. Practitioners, however, also are allowed to consider a higher rate of sea-level change (e.g., a global rise of 2.0 m at 2100 global scenario), if justified by project conditions (USACE, 2013). In addition, the flexibility to use even higher scenarios, when justified, can account for changes in statistically significant trends and new knowledge about SLR. In 2014, the USACE published technical guidance for adaptation to SLR, including examples of how to incorporate the effects of sea-level change on coastal processes, project performance, and project response within a tiered, risk-based planning framework.

Moreover, web-based tools have been developed to automate the computation of SLR scenarios and provide the desired consistency with repeatable analytical results. One tool is described briefly below.

Sea-Level Change Curve Calculator

The "Sea-Level Change Curve Calculator" (Version 2022.55 [(https://cwbi-app.sec.usace.army.mil/rccslc/slcc_calc.html)] is a web tool that allows the user to visualize the USACE and other authoritative sea level rise projections for any tide gauge that is part of the National Oceanographic and Atmospheric Administration's (NOAA's) "National Water Level Observation Network" (NWLON). The SLR change curve in Figure 3 was generated from data derived from Gauge: 9410230, La Jolla, California. While the curve appears more asymptotic near the 2100 year-end, there are three major breaks in slope that align in a curvilinear fashion over a 75-year design life: from the year 2022 to the year 2058; from 2059 to 2083, and from 2084 to 2097 (the end of the design life). These three linear portions are discussed further, later in the text.

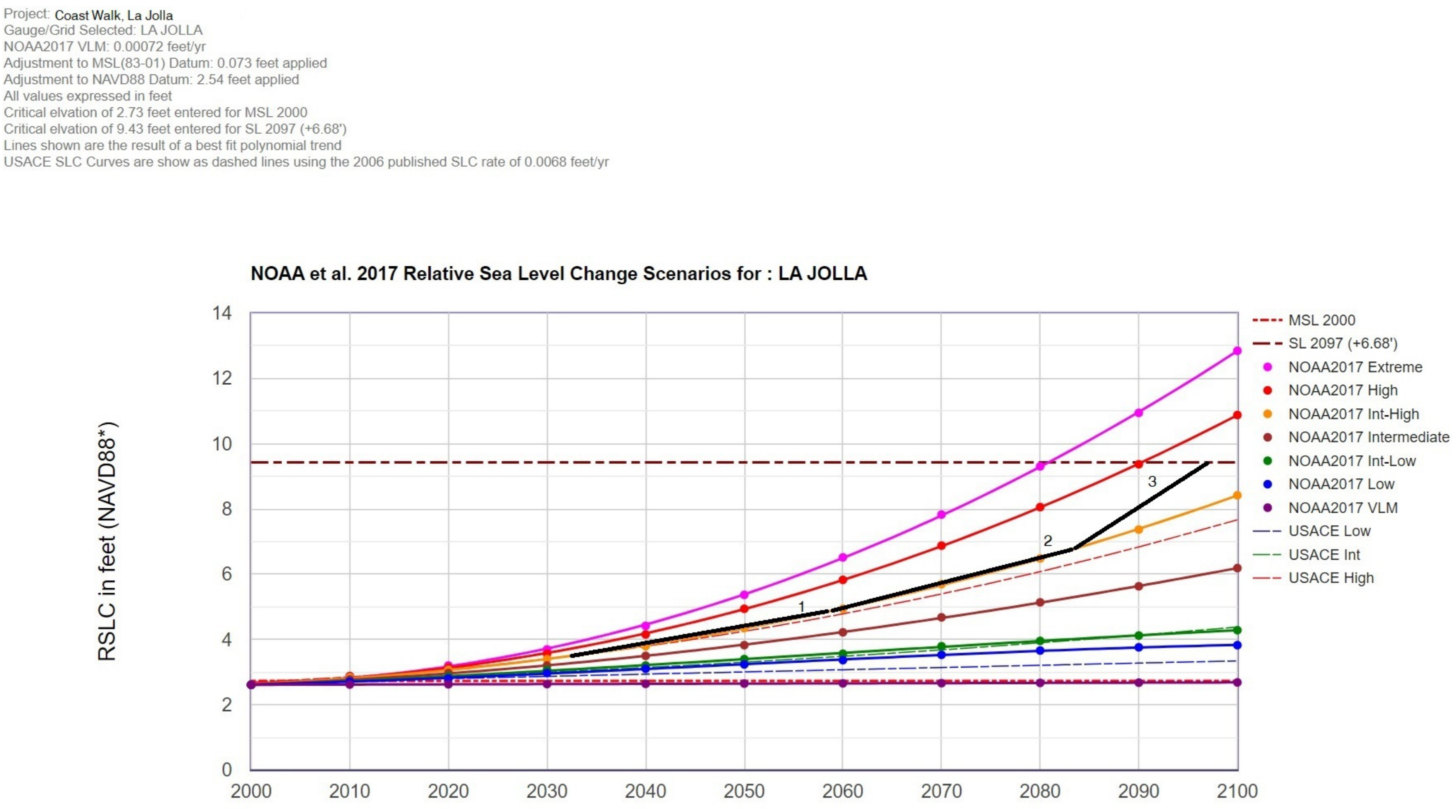


Figure 3 - Sea-Level Change Curve Calculator

Computer climate models make an enormous range of assumptions and have not been able to accurately predict short-term observed climate changes. These models use assumptions that are manipulated, and parameters that are adjusted to produce a range of SLR scenarios. Whether all this tampering and adjusting really collectively add up to a realistic representation of the atmosphere is open to conjecture, since the primary greenhouse gas driver, water vapor, is largely ignored.

The most current Environmental Protection Agency (EPA) global sea level rise prediction is available on their website. The EPA approximate range for global sea level rise in 2100 is 0.6 meters (2 feet) to 2.2 meters (7.2 feet) above present sea level (Sweet et al., 2022).

Recently adopted guidelines by the California Coastal Commission ([CCC], 2018) indicate that the planning scenario for a "medium-high risk aversion" (based on greenhouse gas emissions), should be considered for residential coastal development, and further point out that the high risk scenario follows current greenhouse emissions tracking. CCC (2018) indicates that this range of SLR is the "best available science" in spite of the lack of scientific consensus. In fact, CO_2 has a 140 million-year trend of decreasing atmospheric concentration (Berner, 2001; Wrightstone, 2017), to historic and current levels (approximately 285 to 405 ppm), as indicated on Figure 4. The predicted large rise in sea level comes from computer climate models predicated on greenhouse gas emissions (primarily <u>CO_2</u>, which approximately comprises a <u>mere 6 percent of all greenhouse gases</u>) causing global temperature to rise (rather than the other way around), regardless of the gross lack of correlation of that relationship during geologic time (see Figure 4).

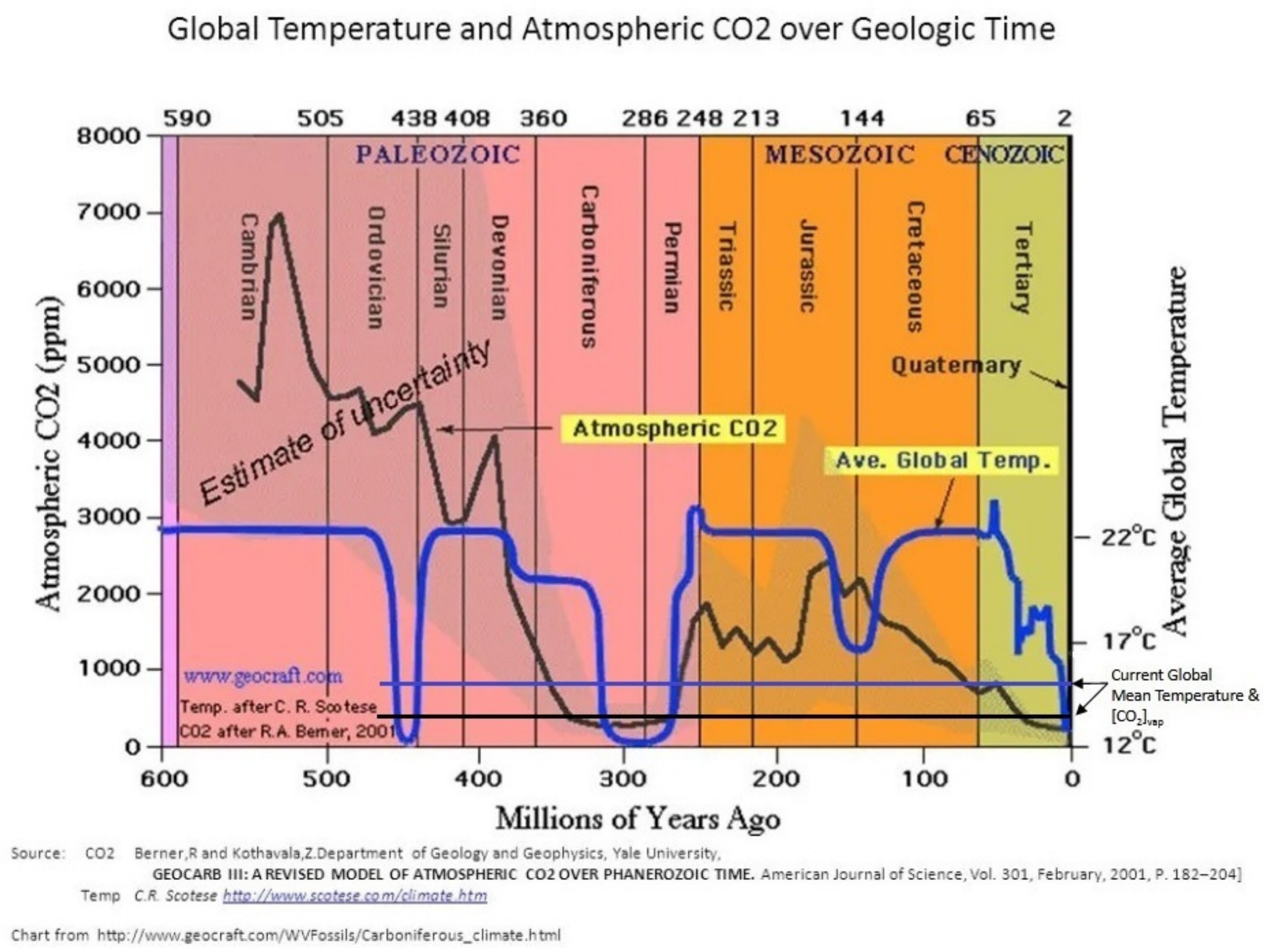


Figure 4 - Geologic Timescale: Concentration of CO_2 and Temperature Fluctuations.

Clearly, as indicated previously, other natural cyclic factors, besides atmospheric carbon, influence earth temperatures and global warming. Again, these natural cycles did not just suddenly halt at the commencement of the Industrial Revolution. Regardless, using the CCC guidance document (CCC, 2018), the "Medium-High risk aversion scenario" (equivalent to 0.5% probability that SLR exceeds this amount), yields an approximate sea level rise of 7.1 feet above current sea level by the year 2100 . Extrapolating for a 75-year design life of the proposed residential structures, this is equivalent to about 6.7 feet above current sea level at La Jolla (closest available projection in CCC [2018]). In contrast, Scripps Institution of Oceanography indicates current SLR is tracking along the lower-intermediate to low-curve shown in Figure 3, which predicts that sea level in the year 2100 will be approximately 4.3 feet higher than at present.

FUTURE LONG-TERM BLUFF RETREAT RATE

CoSMoS 3.0 Computer Application

Recently, the CCC has been using the online computer application, CoSMos 3.0, developed by the United States Geological Survey (Barnard et al., 2018) as a tool for evaluating the magnitude of coastal bluff erosion under SLR. In order to test the validity of the CoSMoS 3.0 computer modeling for coastal bluff retreat at the subject site and vicinity, GSI performed an analysis with 25 cm (about 0.8 ft) of SLR. The analysis indicated that the coastal bluff would retreat slightly landward of the Coast Walk roadway, where it fronts Lot 2, or roughly 60 feet landward of its present condition (Figure 5). Interestingly, there would be less retreat along the coastal bluff to the northwest of the site, seaward of

Figure 5 - Limits of coastal bluff retreat with 25 cm of SLR, based on CoSMoS 3.0 projections.

Prospect Place, where pervasive sea caves are present. Thus, it is the opinion of this firm that the CoSMos 3.0 computer modeling does not accurately predict coastal bluff retreat resulting from SLR. In fact, the United States Geological Survey (USGS) expressly discourages the use of CoSMos 3.0 for regulatory decisions and permitting. Clearly, CoSMos 3.0 is not the appropriate tool for assigning site-specific rates of future coastal bluff retreat.

Future Retreat

Assuming an increased retreat rate in the future, per CCC guidelines, the rate should transition from the current rate to the future rate. To account for the possible added effects from SLR over the aforementioned time period, GSI has reasonably assumed that the rate of bluff retreat over the next 75 years should be similar to the past, for several reasons: 1) as sea level rises, the indurated Point Loma Formation in the seacliff would be occasionally impacted by waves, as it is now, and should have very little effect on bluff retreat (see Plates 2 and 3); and 2) the plots of SLR approach asymptotic near the end of the 75-year design life. In contrast, the curves are much more linear toward the beginning of the design life.

Additionally, rather than becoming inundated by SLR, the shoreline and near-shore will readjust to the new sea level over time such that waves and tides will see the same profile that exists today. This is the principle of beach equilibrium (Dean, 1990), and is the reason why we have shorelines today, even though sea level has risen over 300 feet in the last ~20,000 years. Thus, it can be expected that under most normal conditions, incoming waves will break and their energy will attenuate before impacting the bluff. Under high tides/storm conditions, incoming waves will continue to impact the resistant Point Loma Formation, as they do at present, only at a slightly higher elevation within the bluff profile (see Plates 2 and 3).

Simplified Numerical Model of Shoreline Evolution

The CCC now observes the simplified numerical models developed by Ashton, et al. (2011) and Young, et al. (2014) as “state-of-the-art” tools for assessing the long-term retreat of coastal bluffs relative to current SLR projections. These simplified models build upon and generally follow the core principles of the “Soft Cliff and Platform Erosion” (SCAPE) developed by Walkden and Hall (2005) and Walkden and Dickson (2008). SCAPE consists of a two-dimensional/quasi three-dimensional modeling tool used to replicate the geomorphic evolution of eroding soft rock shorelines (including platform, beach, waves, tides, cliff, and engineering interventions) over timescales of years to millennia.

Unlike the SCAPE model, which uses randomly determined wave inputs, fluctuating tidal cycles, and heterogenous erosion relationships, the simplified numerical models fit these parameters into a “zone” of wave -induced erosion concentrated around sea level and with predetermined vertical range, and erosive potential. In other words, the vertical range of erosion is representative of both the tidal range and the varying heights of incoming waves. Within the tidally averaged surf zone, the bedrock profile is eroded at a rate proportional to its slope. Points above the zone of active marine erosion stay landward of the top of the wave-cut platform, thus, maintaining an arbitrarily vertical cliff. The bedrock shore profile located below the zone of wave attack does not change within the model configuration; and therefore, are representations of abandoned relict slopes. The model is carried out by raising sea level at a constant rate that is varied between simulations.

The simplified model produces a dynamic equilibrium profile of an eroded shoreline, similar to the SCAPE model, whereby the erosion rate is a function of the velocity of cliff retreat. More specifically, the model initially shows a direct relationship between erosion and SLR, but for higher rates of SLR, the erosion rates begin to diminish as the equilibrium erosion profile steepens.

The simplified numerical model equation is defined as:

$$R_2 = R_1 (S_2/S_1)^m$$

Where:
R_2 = Future retreat rate
R_1 = Historical retreat rate
S_1 = Historical rate of sea level rise
S_2 = Future rate of sea level rise
m = Site-specific response parameter

According to Ashton, et al. (2011), the parameter "m" is dependent on the feedbacks between the shore profile geometry and erosion. An instant or linear feedback ($m=1$) represents an eroding shoreline where the erosion rate and SLR rate increase linearly (see Figure 6 [their Fig 12]). Potential examples of eroding shorelines exhibiting an instant response are dominated by sediment flux gradients and include coasts with bluffs and cliffs with high sediment yields. A negative feedback or nonlinear system ($0<m<1$) includes eroding shorelines with negative feedbacks, such as high earth material strengths or a protective beach that reduces erosion. Potential examples of negative feedback systems are shorelines dominated by wave-driven erosion, such as rocky shore platforms and coastal bluffs adjacent to low volume beaches. A no feedback system ($m=0$) includes eroding shorelines where the magnitude of erosion is independent of SLR. Potential examples of no feedback systems include shorelines comprised of hard rock without shore platforms, shorelines dominated by bioerosion, or shorelines subjected to low wave energy. Lastly, an inverse feedback system ($m<1$) represents a shoreline where the erosion rates could decrease as SLR rates increase. Potential environments include shorelines subjected to bioerosion and reflective coastal bluffs.

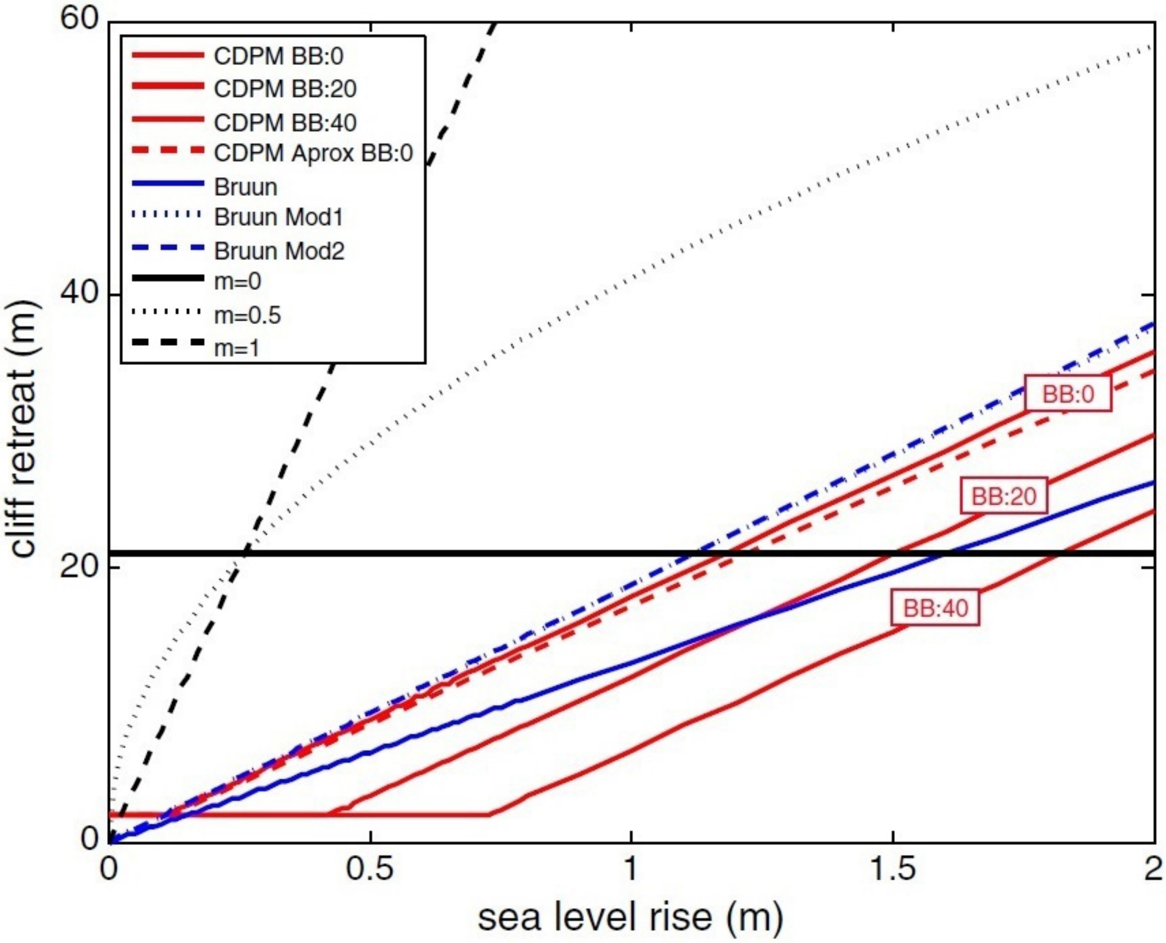


Fig. 12. Comparison of the conditionally decoupled profile model (CDPM) with 0, 20, and 40 m beach buffers (BB) and original Bruun, modified Bruun (Bruun Mod1 and Mod2), no feedback ($m = 0$), approximate SCAPE ($m = 0.5$), and linear extrapolation ($m = 1$). Exponent m models are based on historical cliff and MSLR, while others are sediment balance based.

Figure 6 - Sea Level Rise (meters) and Cliff Retreat (meters).

Model Limitations

Ashton, et al. (2011) indicate that the simplified numerical model is limited to evaluating shoreline erosion along rocky coasts with low volume beaches and coastal bluffs that do not contribute significant beach accreting sediment. Moreover, these researchers state that the simplified numerical model is best suited for evaluating shoreline erosion over long timescales, such as millennia, and not appropriate for shorter time periods under the purview of most coastal management applications. Lastly, the simplified numerical model does not consider longshore sediment transport, which can either build or decay protective beaches.

Coastal Bluff Lithology

The lithology of the nearby coastal bluff likely provides the greatest dampening effect on marine erosion. As shown on Plates 2 and 3, wave attack will still be focused on the more resistant Point Loma Formation rather than the more erodible, overlying old paralic deposits within the 75-year design life of the currently proposed residential structures, even during astronomical high tides. A review of Figures 6(a) and 6(b) in Benumof and Griggs (1999) indicates that the Point Loma Formation within the La Jolla section (appropriate for the subject site) exhibited the second highest mean Schmidt Hammer rebound values of their studied San Diego County coastal bluffs. Only coastal bluffs, mostly composed of Point Loma Formation in the Sunset Cliffs section, displayed higher rebound values.

Presence of a Protective Beach, Shore Platform, and Submarine Canyon

The shoreline along the toe of the coastal bluff, fronting APN 350-131-02-00, is generally composed of Point Loma Formation with sporadic cobbles and failed, boulder-sized fragments of Point Loma Formation. These shoreline deposits are more concentrated seaward of the bluff toe, forming a shingle rampart. This quasi-revetment helps dissipate in-coming wave energy before it can impact the coastal bluff, and will equilibrate in step with SLR over the 75-year design life of the proposed residential structures (Dean,1990). The La Jolla Submarine Canyon also helps to reduce wave runup by decreasing wave amplitude, shoreward of the canyon head. Lastly, the broad shore platform attenuates in-coming wave energy prior to impacting the coastal bluff, also limiting runup.

Most of the time, the shoreline is wider than 20 feet, similar to a conditionally decoupled profile model (CDPM) curve BB:0 (see Figure 6, which is Figure 12 of Young, et al., 2014). Curve BB:0, which is below the m= 0.5 (or ½) curve of the simplified numerical equation, and closer to m=0, near the 2 meter SLR endpoint (when the design 6.7 feet of SLR will have occurred). Given the proximity to the BB:0 (m=0) line and the aforementioned geologic and bathymetric factors that limit marine-induced bluff erosion, we judge that m = 0.1 (or 1/10) appears appropriate for the coastal bluff adjacent to APN 350-131-02-00

FUTURE BLUFF RETREAT SUMMARY

The calculated long-term rate of future bluff retreat using the simplified numerical model equation is presented below, based on the aforementioned three curvilinear sections and:

1. Historical retreat rate based on our review of aerial photographs (0.088 ft/yr) = R_1

2. Average SLR trend over 97 years (1924 to 2021), based on NOAA (Scripps Pier, La Jolla) is 2.04 mm/yr (0.007 ft/yr) = S1

3. Future SLR rate (2097), under *medium-high risk aversion scenario* = 6.7 ft/75 yrs = 0.089 ft/yr = S_2

4. m = 1/10

GSI's assignment of the value for the exponent "m" is reasonable based on the response of the nearby coastal bluff to increased rates of SLR, and would lie close to the no feedback (m=0) system discussed in Ashton, et al. (2011); and therefore likely close to zero.

The premises discussed previously should largely allow the retreat rate to remain unaffected in reality. However, GSI has reasonably assumed SLR will mimic the historical bluff retreat rate for the next 37 years (through 2058). We have used 0.088 ft/yr for this time interval. The erosion rate should marginally increase for the following 25 years (2059 through 2083), and we have reasonably added ⅓ of the change in the erosion rate in 2097 to the initial erosion rate (Δ, see below). During the more asymptotic SLR end of the 75-year design life (2084 through 2097), the bluff retreat rate should be closer to the site-specific upper bound bluff retreat rate for this time interval, even though only the more resistant Point Loma Formation would be impacted by SLR.

Both the low and high site-specific historic bluff erosion rates are indicated in the calculations below:

Site-Specific Future Retreat Rate

At year 2097, under *medium-high risk aversion scenario (0.5% Probability)*,

$$R_2 = R_1\ (S_2/S_1)^m$$
$$R_2 = (0.088\text{ ft/yr})\ (0.089\text{ ft/yr}/0.007\text{ ft/yr})^{1/10}$$
$$R_2 = (0.088)\ (12.71)^{1/10}$$
$$R_2 = (0.088)(1.29) = 0.114\text{ ft/yr in the year 2097.}$$

Based on the above, the retreat rate will change from 0.088 ft/yr to 0.114ft/yr, and the difference between the 75-year commencement and end of the design life, Δ= 0.026 ft/yr, from 2022 to 2097.

FUTURE BLUFF RETREAT BASED ON SLR CURVE INCREMENTS - Low			
APPLICABLE DATES	BLUFF RETREAT RATE (FT/YR)	DURATION (YEARS)	BLUFF RETREAT (FEET)
2022-2058 (0.088 ft/yr) current SLR rate	0.088	37	3.26
2059-2083 (0.088 ft/yr + ⅓[Δ] = (0.088 ft/yr +⅓[[0.026 ft/yr])= 0.097 ft/yr increase in SLR rate	0.097	25	2.43
2083-2097 (Calculated SLR rate in 2096 = 0.114 ft/yr)	0.114	13	1.48
	Totals	**75**	**7.17**

As shown above, the nearby coastal bluff may experience approximately 7.2 feet of retreat over the 75-year design life of the proposed residential structures in the unlikely event that sea level is 6.7 feet higher that present day in the year 2097. This retreat distance is illustrated in plan view on Plate 1. Plates 2 and 3 also show the lack of the effects of SLR on the bluff face, along with a hypothetical representation of the eroded coastal bluff profile at the end of 75 years or in the year 2097, based on the calculated retreat.

LABORATORY TESTING

General

Laboratory tests were performed on relatively undisturbed and representative bulk samples of the onsite earth materials in order to evaluate their physical characteristics. The test procedures used and results obtained are presented below and in Appendix D.

Classification

Soils were classified visually according to the Unified Soils Classification System (Sowers and Sowers, 1979). The soil classifications are shown on the Boring Logs in Appendix B.

Moisture-Density Relations

The field moisture contents and dry unit weights were evaluated in the laboratory for relatively undisturbed samples of the site earth materials, collected from the borings. Testing was performed in general accordance with ASTM D 2937 and ASTM D 2216. The dry unit weight was reported in pounds per cubic foot (pcf), and the field moisture content was reported as a percentage of the dry weight. The results of these tests are shown on the Boring Logs in Appendix B.

Atterberg Limits

An Atterberg limits test was performed on a bulk sample of the old paralic deposits collected from Boring B-1 to evaluate the sample's liquid limit, plastic limit, and plasticity index. The Atterberg limits test was conducted in general accordance with ASTM D 4318. The test results are presented in the following table and in Appendix D. Testing indicates that the sample is subject to plastic deformation with a U.C.S.C. designation of CL.

SAMPLE LOCATION AND DEPTH (FT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
B-1 @ 10	42	12	30

Direct Shear

Shear testing was performed on a relatively undisturbed sample of the old paralic deposits collected from Boring B-1. The shear test was performed in general accordance with ASTM Test Method D 3080 in a direct shear machine of the strain control type. The shear test results are presented as follows and in Appendix D:

SAMPLE LOCATION AND DEPTH (FT)	PRIMARY		RESIDUAL	
	COHESION (PSF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)	FRICTION ANGLE (DEGREES)
B-1 @ 3	447	32	116	31

Soil pH, Saturated Resistivity, Soluble Sulfates and Soluble Chlorides

Testing was conducted on a representative bulk sample of the near-surface onsite earth materials, collected from Boring B-2, for an evaluation of general soil corrosivity and soluble sulfates, and soluble chlorides. Testing was performed in general accordance with California Test Methods (CTMs) 643-99, 417, and 422. The test results are presented in Appendix D and the following table:

SAMPLE LOCATION AND DEPTH (ft)	pH	SATURATED RESISTIVITY (ohm-cm)	SOLUBLE SULFATES (% by weight)	SOLUBLE CHLORIDES (ppm)
B-2 @ 0-5	7.0	1,200	0.045	140

Corrosion Summary

The laboratory testing indicates that the tested sample of the onsite soils is neutral with respect to soil acidity/alkalinity; is corrosive to exposed, buried metals when moist; presents negligible sulfate exposure to concrete (Exposure Class S0 per Table 19.3.1.1 of American Concrete Institute [ACI] 318-14 [ACI, 2014]), and has a slightly elevated concentration of soluble chlorides that is below action levels.

GSI does not consult in the field of corrosion engineering. Therefore, additional comments and recommendations may be obtained from a qualified corrosion engineer based on the level of corrosion protection required for the project, as determined by other members of the design team. The site is located with a corrosive environment created by the Pacific Ocean. This should be considered in the project design and construction. The mix design for structural concrete that may come into contact with sea spray should conform to the guidelines in Table 19.3.2.1 of ACI 318-14 (ACI, 2014) for Exposure Class C2 conditions, if determined necessary by the project architect.

SLOPE STABILITY ANALYSIS

GSI performed slope stability analyses along Geologic Cross Section A-A' using the geologic conditions encountered during our onsite field exploration and our past work experience on a nearby property (GSI, 2015). The analysis was conducted using the two-dimensional limit-equilibrium slope stability software program "GEOSTASE" version 4.30.31, developed by Gregory Geotechnical (2019). A general summary of the "GEOSTASE" program is included in Appendix E.

GSI analyzed 2 potential failure scenarios. One case involved a circular-type failure through the old paralic deposits in the upper bluff. The other included a non-circular gross failure through the entire bluff. The analyses were conducted using the Spencer Method since it satisfies both force and moment equilibrium.

Shear strength properties applied to the earth materials considered in the analyses were based on the results of soil strength testing performed on a relatively undisturbed sample of the onsite old paralic deposits and samples of the Point Loma Formation from a nearby property (GSI, 2015, 2016), as well as our professional judgement. The soil strengths assigned to the geologic units included in the analyses are summarized in Appendix E. Anisotropic soil strength values were applied to the old paralic deposits and the Point Loma Formation.

Owing to the perched groundwater seepage encountered in Boring B-2, a groundwater surface was modeled near the geologic contact between the old paralic deposits and the Point Loma Formation. Another groundwater surface was placed at MSL to model the regional groundwater table.

In order to model loads applied by HS20 fire apparatus within the Coast Walk roadway, we incorporated a distributed load equivalent to 300 pounds per square foot (psf). Although Geologic Cross Section A-A’ does not traverse the proposed residential buildings, for reasonable conservatism, we included a distributed load of 1,500 psf to model loading applied by the primary residential structure.

Pseudo-static (seismic) slope stability analyses were performed for the upper-bluff failure case to evaluate the effects of seismic loading on the stability of the coastal bluff.

Summary of Slope Stability Analyses

The results of our slope stability analyses are discussed below. Computer-generated printouts from the slope stability analyses are included in Appendix E.

Upper-Bluff Failure

The analyses demonstrated that the theoretical failure surface with a static factor-of-safety (FOS) of 2.8 against upper-bluff failures would daylight the ground surface along Geologic Cross Section A-A’ at an approximate distance of 29 feet landward of the coastal bluff edge. The analyses also showed that the theoretical failure surface with a seismic FOS of 2.0 would daylight the ground surface along Geologic Cross Section A-A’ at an approximate distance of 35 feet from the bluff edge. Since the gross failure analysis was more onerous, it was used to determine the Geologic Setback for the project.

Gross Bluff Failure

The analyses showed that the theoretical failure surface with a static FOS of 2.3 against gross bluff failure would daylight the ground surface along Geologic Cross Section A-A’ at an approximate distance of 43 feet landward of the coastal bluff edge. Using linear interpolation, GSI estimates that the theoretical failure surface with a static FOS of 1.5 against gross bluff failure would occur approximately 17 feet landward of the coastal bluff edge. Owing to the extremely high static FOS we obtained for the gross bluff failure case and the unlikelihood of gross bluff failure, it is our opinion that seismic analyses are not warranted for the gross failure case (California Department of Conservation, California Geological Survey, 1997). The locations where the static FOS against gross bluff failure equals 1.5 are shown on the Geotechnical Map (Plate 1) and Geologic Cross Sections, A-A’ through C-C’ (Plates 2 and 3).

Surficial Slope Stability

Based on published and accepted erosion rates, our analyses, and our observations, a coastal bluff is inherently surficially unstable. However, based on our aforementioned findings regarding site-specific coastal bluff retreat, a remodeled residential structure should not be adversely affected from retreat over its 75-year design life.

GEOLOGIC SETBACK FOR PROPOSED DEVELOPMENT

Based on the results of our coastal bluff retreat evaluation and slope stability analyses, it is our opinion that the City of San Diego minimum 25-foot development setback from the coastal bluff edge is appropriate for the proposed improvements. This geologic setback is considered sufficient mitigation against coastal bluff retreat and bluff instability over the 75-year design life of the proposed residential structures without the need for shoreline protection measures. The recommended geologic setback is shown on Plates 1 through 3.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on our site-specific field exploration, laboratory testing, and geotechnical engineering analyses, and our past work experience on a nearby coastal bluff property (GSI, 2015), it is our opinion that the site appears suitable to receive the proposed residential development from geotechnical engineering and geologic viewpoints, provided that the recommendations, presented in this report, are properly incorporated into the design and construction phases of site development. The primary geotechnical concerns with respect to the proposed remodel/redevelopment are:

- Coastal bluff stability and retreat throughout the design life of the proposed improvements.
- Earth material characteristics and the depth to suitable bearing materials below existing grades.
- On-going expansion/corrosion potentials of the onsite soils.
- The proximity of the site to a corrosive environment (i.e., Pacific Ocean).
- The infeasibility of storm water infiltration into the onsite earth materials for permanent, post construction storm water BMPs.
- Potential for perched groundwater to occur both during and after development.
- Non-structural zone for unmitigated perimeter areas (improvements subject to distress).
- Temporary slope stability.
- Regional seismic activity.

The recommendations presented herein consider these as well as other aspects of the site. The engineering analyses, performed, concerning site preparation and the recommendations presented herein recognize the information provided and obtained during our field work within the subject parcels and a nearby Coast Walk property. In the event that any significant changes are made to the proposed site development, the conclusions and recommendations contained in this report shall not be considered valid unless the design changes are reviewed and the recommendations of this report are evaluated or modified in writing by this office. Foundation design parameters are considered preliminary until the foundation design, layout, and structural loads are provided to this office for review.

1. Geotechnical observation and testing services should be provided during earthwork to aid the contractor in removing unsuitable soils and in his effort to compact the fill.

2. Geologic observations should be performed during any earthwork to verify or further evaluate the exposed geologic conditions. Although unlikely, if adverse geologic conditions or structures are encountered, supplemental recommendations and earthwork may be warranted.

3. Quantitative slope stability analyses modeling upper-bluff and gross bluff failures, and our evaluation of long-term retreat of the nearby coastal bluff indicate that the City of San Diego minimum 25-foot development setback from the coastal bluff edge would be sufficient mitigation against coastal bluff failures and retreat over the 75-year design life of the proposed residential structures, without the need for shoreline protection measures.

4. The residual soil that occurs within the upper approximately 1 foot to 2 feet of the existing grades is considered potentially compressible in its existing state; and therefore, should not be relied upon for the support of the planned settlement-sensitive improvements (i.e., new foundations, new slab-on-grade floors, underground utilities, walls, pavements, swimming pool, etc.) and new planned fills without mitigation. Unless extracted during the planned excavations, the residual soil should be removed and reused as properly compacted fill for structural support. If encountered during earthwork construction, any undocumented artificial fills should receive similar remedial treatment.

5. Based on our observations and laboratory testing of the onsite earth materials, and our previous work experience on a nearby property (GSI, 2015), we estimate that the soils encountered during the proposed development will be medium to high in expansion potential (E.I. = 51 to 130). In accordance with Section 1808.6 of the 2019 CBC, foundations for buildings and structures constructed upon expansive soils (i.e., soils with an expansion index [E.I.] greater than 20) will require specific structural design to mitigate their adverse shrink/swell effects. Alternatively, Section 1808.6 of the 2019 CBC indicates that expansive soils within the influence of the building foundations can be removed and replaced with soils that are very low in expansion potential (E.I. of 20 or less and a plasticity index [P.I.] of 14 or less), or stabilized in place (i.e., cement stabilization). The treatment of expansive soils through earthwork will require selective grading. Preliminary geotechnical recommendations for post-tensioned slab and mat-slab foundation systems are included in this report for the structural mitigation of expansive soils. However, given the different finish floor elevations throughout the proposed residential structures, post-tensioned slab foundations may not be feasible from a structural engineering perspective. The client/developer may consider value engineering studies for expansive soil mitigation.

6. The results of soil pH, saturated resistivity, soluble sulfates, and soluble chlorides testing indicate that a representative sample of the onsite, near-surface soils is neutral with respect to soil acidity/alkalinity; is corrosive to exposed, buried metals when saturated; presents negligible sulfate exposure to concrete (Exposure Class S0 per Table 19.3.1.1 of ACI 318-14 [ACI, 2014]), and has a slightly elevated concentration of soluble chlorides that is below action levels.

   GSI does not consult in the field of corrosion engineering. Therefore, additional comments and recommendations may be obtained from a qualified corrosion engineer based on the level of corrosion protection required for the project, as determined by the project architect, civil engineer, and structural engineer. The site is located with a corrosive environment created by the Pacific Ocean. This should be considered in the project design and construction. The mix design for structural concrete that may come into contact with sea spray should conform to the guidelines in Table 19.3.2.1 of ACI 318-14 (ACI, 2014) for Exposure Class C2 conditions, if determined to be warranted by the project architect.

7. Based on our observations during site-specific field exploration, the results of laboratory testing conducted on the onsite soils, and our past work experience with similar sites, storm water infiltration into the onsite earth materials for permanent post-construction storm water BMPs is not considered feasible from a geotechnical standpoint. The infiltration of storm water has the potential to result in the accumulation of perched groundwater. Lateral migration of perched groundwater could induce swelling of expansive soils and fill settlement within the subject property and the adjacent parcels. Perched groundwater exiting the bluff face would also contribute to spring sapping and reduced bluff stability. It could also increase moisture transmission through the proposed retaining walls and any retaining walls on adjacent properties. Thus, the infiltration of storm water is not recommended from a geotechnical perspective. If onsite treatment and detention of storm water are required, they should occur within fully contained systems or basins lined with impermeable membranes.

8. Based upon our understanding of the currently proposed development and the available subsurface data, groundwater is not expected to be a significant geotechnical factor. However, there is potential for perched groundwater conditions to manifest along zones of contrasting permeabilities (i.e., sandy/clayey fill lifts, geologic contacts, bedding, discontinuities, etc.) during and after construction. The potential for perched groundwater to occur should be disclosed to all interested/affected parties.

9. The 2019 CBC (CBSC, 2019) indicates that remedial grading be performed across all areas of the site covered by the grading permit, and not just within the influence of the proposed residential structures. Relatively thick unsuitable soils may also necessitate a special zone of consideration, on perimeter/confining areas. This

zone would be approximately equal to the thickness of the potentially compressible earth materials, if remedial grading cannot be performed onsite and offsite. The width of this zone would be considered equal to the thickness of the unsuitable soils adjacent to property boundaries or existing improvements that need to remain in service, both during and following construction. Any planned settlement-sensitive improvements, constructed within this zone, may require deepened foundations, additional reinforcement, etc., or will retain some potential for settlement and associated distress. This will require proper disclosure to all interested/affected parties, should this condition exist at the conclusion of grading. Based on the available subsurface data, the width of this zone may be on the order of 1foot to 2 feet.

10. On a preliminary basis, unsupported temporary slopes 20 feet or less in gross overall height, that do not expose saturated soils, groundwater seepage, running sands, or other adverse conditions may be constructed in accordance with CAL/OSHA guidelines for Type "B" soils (i.e., a 1:1 [h:v] temporary slope gradient). All temporary slopes should be observed by a licensed engineering geologist or engineer prior to worker entry. Should hazardous conditions be exposed, temporary slopes may need to be altered to flatter gradients or shored. Shoring or slot grading will likely be necessary where existing improvements and property boundaries do not allow for the recommended temporary slope gradients.

11. Site soils are considered erodible. Thus, the proper control of surface drainage is considered essential and should be maintained over the life of the proposed development. Surface runoff should not be directed toward the bluff edge, the tops of any graded slopes, or foundations.

12. The subject site is susceptible to moderate to strong ground shaking from an earthquake occurring on any of the regional Holocene-active fault systems. Therefore, the seismicity-acceleration values, provided herein, should be considered during the design and construction of the proposed residential buildings.

13. General Earthwork and Grading Guidelines are provided at the end of this report as Appendix F. Specific recommendations are provided below.

EARTHWORK CONSTRUCTION RECOMMENDATIONS

General

All earthwork should conform to the guidelines presented in Appendix J of the 2019 CBC (CBSC, 2019), the requirements of the City of San Diego, and the Grading Guidelines presented in Appendix F, except where specifically superceded in the text of this report.

In the event that the grading codes and the recommendations in this report are found to be in conflict, the most conservative approach should be undertaken. Prior to grading, a GSI representative should be present at the preconstruction meeting to provide additional grading guidelines, if needed, and to review the earthwork schedule.

During earthwork construction, all site preparation and the general grading procedures of the contractor should be observed and the fill selectively tested by a representative(s) of GSI. If unusual or unexpected conditions are exposed in the field, they should be reviewed by this office and, if warranted, modified or additional recommendations will be offered. All applicable requirements of local and national construction and general industry safety orders, the Occupational Safety and Health Act, and the Construction Safety Act should be met.

Site Preparation

Any demolition debris, vegetation, and other deleterious material should be removed from the development area prior to the start of earthwork construction.

Remedial Excavation (Removal of Unsuitable Earth Materials)

If not extracted by the planned excavations, the residual soil should be removed to expose suitable, unweathered old paralic deposits. Any undocumented artificial fill encountered during earthwork construction should receive similar remedial excavation. The excavated soils may be reused as compacted fill following the removal of any organic matter and deleterious materials. Based on the available subsurface data, remedial excavations are anticipated to extend to depths of approximately 1 foot to 2 feet below the existing grades. However, variations are possible and potentially compressible earth materials may extend to greater depths, locally, and require deeper remedial excavation. Remedial excavations should be completed below a 1:1 (h:v) plane projected down and away from the bottom, outer edge of the proposed settlement-sensitive improvements or the limits of new planned fills unless constrained by property lines or existing improvements that are to remain in service, both during and following construction. Remedial excavations should be observed by the geotechnical consultant. Once approved, the bottom of the remedial excavations should be scarified at least 6 to 8 inches, uniformly moisture conditioned to 2 to 3 percent above the soil's optimum moisture content, and then be compacted to a minimum relative density of 90 percent of the laboratory standard (per ASTM D 1557).

Overexcavation

In order to provide uniform support, GSI recommends that any unweathered old paralic deposits located within 48 inches of pad grade or 24 inches below the lowest foundation element (whichever is greater) be overexcavated (undercut) to provide for a vertical section of compacted fill that is at least 48 inches thick within the building pad areas and 24 inches thick below all building footings, including elevator pit foundations. The maximum to minimum fill thickness across the building pad should not exceed a ratio of 3:1 (maximum:minimum). The overexcavation bottoms should be observed by the geotechnical consultant, sloped away from the structures, scarified at least 6 to 8 inches, uniformly moisture conditioned to 2 to 3 percent above the soil's optimum moisture content, and then compacted to a minimum relative density of 90 percent of the laboratory standard (per ASTM D 1557).

Earthwork Mitigation of Expansive Soils

As an alternative to using specialized structural design for reducing shrink/swell deformations imparted by the onsite expansive soils, earthwork mitigation may be performed. Earthwork mitigation would include removing and replacing expansive soils with soils that are very low in expansion potential (E.I. of 20 or less and P.I. of 14 or less), such that the upper 15 feet of soil below the proposed pad grades has a weighted P.I. less than 15. Additional subsurface exploration and laboratory testing is recommended to evaluate the feasibility of this alternative method of expansive soil mitigation.

Compacted Fill Placement and Compaction

Compacted fill materials should be cleansed of major vegetation and debris, uniformly moisture conditioned to at least 2 to 3 percent above the soil's optimum moisture content, placed in relatively thin 6- to 8-inch lifts, and mechanically compacted to obtain a minimum relative density of 90 percent of the laboratory standard (ASTM D 1557). Fill materials placed within 10 feet of pad grade should not contain rocks with sizes greater than 12 inches in any dimension. The geotechnical consultant should perform observations and field density testing during fill placement and compaction.

Import Fill Materials

Any proposed import fill materials should be observed and determined suitable by the geotechnical consultant prior to delivery to the site. Import fill material (if required) should have an E.I. of 50 or less. If the import will be used for expansive soil mitigation, it should have an E.I. of 20 or less and a P.I. of 14 or less. The structural design of the proposed improvements may require modification if import fill materials have a greater expansion potential than the onsite soils. If the import fill materials originate from a site other than a quarry, the environmental documents for the source site should be provided to GSI for review. At least three (3) business days are recommended for import submittal reviews and compliance testing, prior to importation.

Temporary Slopes

On a preliminary basis, temporary slopes greater than 4 feet, but less than 20 feet in overall height, completed into the onsite earth materials should conform to CAL/OSHA and OSHA requirements for Type "B" soil conditions (i.e., 1:1 [h:v] temporary slope gradient) provided that saturated soils, groundwater, running sands, or other adverse geologic conditions are not present. Heavy equipment storage and traffic and the stockpiling of soils, demolition, debris, and building materials should not occur within a horizontal distance of “H” from the tops of temporary slopes, where “H” equals the height of the temporary slope. All temporary slopes should be observed by a licensed engineering geologist or engineer prior to worker entry. If temporary slopes conflict with property boundaries or essential existing improvements, shoring or slot grading may be necessary. The need for shoring or slot grading should be further evaluated during the grading plan review stage of development. Preliminary recommendations for shoring and slot grading are included herein.

Slot Grading

Slot grading may be performed as an alternative to shoring when excavating below a 1:1 (h:v) plane projected down from property lines or existing improvements that are to remain in service. On a preliminary basis, the maximum depth of the slots should not exceed 10 feet and the width of an open slot should be no greater than 6 feet. Multiple slots may be excavated simultaneously provided that open slots are separated by a minimum 12-foot wide section of undisturbed soils or tested and approved compacted fill. Open slots should be observed by GSI prior to backfill.

Graded Slopes

GSI understands that grade transitions will be accommodated by the planned retaining walls. Thus, graded slope construction is not anticipated at this time. If graded slopes are included in the proposed development, GSI can provide recommendations for the construction of such after grading plans have been provided for GSI review.

Excavation Observation and Monitoring (All Excavations)

When excavations are made adjacent to an existing improvement (i.e., underground utility, wall, road, building, etc.), there is a risk of some damage even if a well-designed system of excavation is planned and executed. We therefore recommend that a systematic program of observations be made before, during, and after construction to determine the effects (if any) of the excavation on existing improvements.

We believe that this is necessary for two reasons. First, if excessive movements (i.e., more than ½-inch) are detected early enough, remedial measures can be taken which could possibly prevent serious damage to existing improvements. Second, the responsibility for damage to the existing improvement can be evaluated more equitably if the cause and extent of the damage can be determined more precisely.

Monitoring should include the measurement of any horizontal and vertical movements of the existing structures/improvements. Locations and types of monitoring devices should be selected prior to the start of construction. The program of monitoring should be agreed upon between the pertinent members of the project team, prior to excavation.

Reference points should be provided on existing walls, buildings, and other settlement-sensitive improvements. These points should be placed as low as possible on the walls and buildings adjacent to the excavation. Exact locations may be dictated by critical points, such as bearing walls or columns for buildings; and surface points on roadways or curbs, near the top of the excavation.

For a survey monitoring system, an accuracy of a least 0.01 foot should be required. Reference points should be installed and read initially prior to excavation. The readings should continue until all construction below ground has been completed and the permanent backfill has been brought to finish grade.

The frequency of readings will depend upon the results of previous readings and the rate of construction. Weekly readings could be assumed throughout the duration of construction with daily readings during rapid excavation near the bottom of the excavation. The readings should be plotted by the project surveyor/civil engineer and then reviewed by the geotechnical consultant. In addition to the monitoring system, it would be prudent for the geotechnical consultant and the contractor to make a complete inspection of the existing structures and improvements both before and after construction. The inspection should be directed toward detecting any signs of damage, particularly those caused by settlement. Notes should be made and pictures should be taken where necessary.

Observation

All excavations should be observed by a licensed engineering geologist or engineer. Should the observation reveal any unforseen hazard, the engineering geologist or engineer will recommend treatment. Please inform GSI at least 24 hours prior to any required site observation.

Earthwork Balance (Shrinkage/Bulking)

The volume change of excavated materials, upon compaction as engineered fill, is anticipated to vary with material type and location. The overall earthwork shrinkage and bulking of the earth materials anticipated to be encountered during site grading may be approximated by using the following parameters:

Undocumented Fill and Residual Soil . . . . 5% to 10% shrinkage
Quaternary Old Paralic Deposits . . . . 2% to 5% bulking

The above factors are estimates only, based on preliminary data. The residual soil and undocumented fill may achieve higher shrinkage if organics or clay content is higher than anticipated, if a high degree of porosity is encountered, or if compaction averages more than 90 percent of the laboratory standard (per ASTM D 1557). In addition, extensive rodent burrowing may result in higher shrinkage. Final earthwork balance factors could vary. In this regard, it is recommended that balance areas be reserved where grades could be adjusted up or down near the completion of grading in order to accommodate any yardage imbalance for the project.

PRELIMINARY RECOMMENDATIONS - FOUNDATIONS

General

The following preliminary recommendations for the design and construction of building foundations or slab-on-grade floors are based on our understanding of the proposed development, the assumed loading conditions, and the subsurface and laboratory data we have obtained from site-specific and nearby studies. Final foundation and slab-on-grade floor design recommendations will be provided at the conclusion of grading based on the actual loading conditions and the E.I. and P.I. of soils located near the finished pad grades.

In the following sections, GSI provides preliminary "minimum" design and construction recommendations for foundations underlain by soils that are medium to high in expansion potential (E.I. = 51 to 130). As previously indicated herein, foundation systems constructed within the influence of expansive soils should be designed to resist shrink/swell deformations per Section 1808.6 of the 2019 CBC.

The information and recommendations presented in this section are not meant to supercede design by the project structural engineer or a civil engineer specializing in structural design. Upon request, GSI could provide additional input/consultation regarding soil parameters, as related to foundation design.

POST-TENSIONED SLAB FOUNDATION SYSTEMS

If feasible, given the multiple floor levels of the proposed buildings, post-tensioned slab foundation systems may be used to mitigate the damaging shrink/swell effects of the onsite expansive soils. Recommendations for the design and construction of post-tensioned slab foundation systems are provided in the following sections.

The post-tensioned slab foundation designer may elect to exceed the minimum recommendations, provided herein, in order to increase slab stiffness performance. Post-tensioned (PT) slab foundation design may be either ribbed or mat-type. The former uses reinforced internal, concrete beams to assist with rigidity. The latter is also referred to as a uniform thickness foundation (UTF). The use of a UTF is an alternative to the traditional ribbed-type. The UTF offers a reduction in the number or surface area of the internal concrete beams. That is to say a UTF typically uses a single perimeter grade beam and "shovel" footings for hold-downs, but has a thicker slab than the ribbed-type.

Post-tensioned slab foundations should be designed using sound engineering practice and be in accordance with local building codes, 2019 CBC requirements, and Post Tensioning Institute (PTI) methodologies (PTI; 2004, 2008, 2012, 2013, and 2014). Upon request, GSI can provide additional data/consultation regarding soil parameters, as they relate to post-tensioned slab foundation design.

From a soil expansion/shrinkage standpoint, a common contributing factor to distress of structures using post-tensioned slab foundations is a "dishing" or "arching" of the slabs. This is caused by the fluctuation of the moisture content in the soils below the perimeter of the slab, primarily due to onsite and offsite irrigation practices, climatic and seasonal changes, and the presence of expansive soils. When the soil environment surrounding the exterior of the slab has a higher moisture content than the area beneath the slab, moisture tends to migrate inward, underneath the slab edges to a distance beyond the slab edges referred to as the moisture variation distance. When this migration of water occurs, the volume of the soils beneath the slab edges expands and causes the slab edges to lift in response. This is referred to as an edge-lift condition. Conversely, when the outside soil environment is drier, the moisture transmission regime is reversed and the soils underneath the slab edges lose their moisture and shrink. This process leads to dropping of the slab at the edges, which results in what is commonly referred to as the center-lift condition. A well-designed, post-tensioned slab foundation having sufficient stiffness and rigidity provides a resistance to excessive bending that results from non-uniform swelling and shrinking slab subgrade soils, particularly within the moisture variation distance, near the slab edges. Other mitigation techniques typically used in conjunction with post-tensioned slab foundations consist of a combination of specific soil pre-saturation and the construction of a perimeter "cut-off" wall/grade beam. Soil pre-saturation consists of moisture conditioning the slab subgrade soils prior to the post-tensioned slab foundation construction. This effectively reduces soil moisture migration from the area located outside the building toward the soils underlying the post-tensioned slab foundation. Perimeter cut-off walls are thickened edges of the concrete slab that impede both outward and inward soil moisture migration.

Slab Subgrade Pre-Soaking

Pre-moistening of the slab subgrade soil is recommended to reduce the potential for post-construction soil heave. The moisture content of the subgrade soils should be greater than optimum moisture to a depth equivalent to the perimeter grade beam or cut-off wall depth in the slab areas (typically 18 or 24 inches deep for soils that are medium or high in expansion potential, respectively).

Pre-moistening or pre-soaking should be evaluated by the geotechnical consultant 72 hours prior to vapor retarder placement. In summary:

EXPANSION POTENTIAL	PAD SOIL MOISTURE	CONSTRUCTION METHOD	SOIL MOISTURE RETENTION
Medium (E.I. = 51-90)	Upper 18 inches of pad grade soil moisture 2 percent over optimum	Berm and flood or wetting and reprocessing	Periodically wet or cover with plastic after trenching. Evaluation within 72 hours prior to placement of vapor retarder and underlayment section.
High (E.I. = 91-130)	Upper 24 inches of pad grade soil moisture 3 percent over optimum	Berm and flood or wetting and reprocessing	Periodically wet or cover with plastic after trenching. Evaluation within 72 hours prior to placement of vapor retarder and underlayment section.

Perimeter Cut-Off Walls

Perimeter cut-off walls should be at least 18 or 24 inches deep for soils that are medium or high in expansion potential, respectively. The cut-off walls may be integrated into the post-tensioned slab foundation or independent of the foundation. The cut-off walls should be a minimum of 6 inches thick (wide). The bottom of the perimeter cut-off wall should be designed to resist tension, using cable or steel reinforcement per the project structural engineer.

Post-Tensioned Slab Foundation Design

The following recommendations for the design of post-tensioned slab foundations have been prepared in general conformance with the requirements of the recent Post Tensioning Institute's (PTI's) publication titled "Design of Post-Tensioned Slabs on Ground, Third Edition" (PTI, 2004), together with it's subsequent addendums and errata (PTI; 2008, 2012, 2013, and 2014).

Post-Tensioned Slab Foundation Soil Support Parameters

The recommendations for soil support parameters have been provided based on the typical soil index properties for soils that are low to very high in expansion potential. The soil index properties are typically the upper bound values based on our experience and practice in the southern California area. Additional testing is recommended either during or following grading, and prior to foundation construction to further evaluate the soil conditions within the upper 7 to 15 feet of pad grades. The following table presents suggested minimum coefficients to be used in the Post-Tensioning Institute design method:

Thornthwaite Moisture Index	-20 inches/year
Correction Factor for Irrigation	20 inches/year
Depth to Constant Soil Suction	7 feet or overexcavation depth to bedrock
Constant soil Suction (pf)	3.6
Moisture Velocity	0.7 inches/month
Effective Plasticity Index (P.I.)*	20-40
* - The weighted plasticity index should be evaluated for the upper 15 feet of foundation soils either during or following rough grading and prior to foundation construction.	

Based on the above, the recommended post-tensioned slab foundation soil support parameters are tabulated below:

DESIGN PARAMETERS	MEDIUM EXPANSION (E.I. = 51-90)	HIGH EXPANSION (E.I. = 91-130)
e_m center lift	8.7 feet	8.5 feet
e_m edge lift	4.5 feet	3.75 feet
y_m center lift	0.66 inches	0.75 inches
y_m edge lift	1.3 inch	1.7 inches
Bearing Value(1)	1,000 psf	1,000 psf
Lateral Pressure(2)	100 psf	100 psf
Subgrade Modulus (k)	85 pci/inch	70 pci/inch
Lateral Sliding Resistance (Cohesion)(3)	130 psf	130 psf
Minimum Perimeter Footing Embedment(4)	18 inches	24 inches

(1) The bearing value of load-bearing perimeter and internal grade beams of the post-tensioned slab foundation may be increased to 1,500 psf if the beams are a minimum of 12 inches wide and founded at least 12 inches below the lowest adjacent grade into tested and approved compacted fill, overlying suitable old paralic deposits. Allowable bearing values may be increased by one-third for short-term seismic and wind loads.
(2) The upper 6 inches of passive pressure should be neglected if not confined by slabs or pavement.
(3) Cohesion value to be multiplied by the contact area. The lateral sliding resistance should not exceed one-half of the dead load.
(4) As measured below the lowest adjacent compacted subgrade surface without landscape layer or sand underlayment.
Note: The use of open bottomed raised planters adjacent to foundations will require more onerous design parameters.

The parameters are considered minimums and may not be adequate to represent all expansive soils and site conditions such as adverse drainage or improper landscaping and maintenance. The above parameters are applicable provided the grades around the proposed residential buildings provide positive drainage that is maintained away from the building foundations. In addition, no trees with significant root systems are to be planted within 15 feet of the perimeter of foundations. Therefore, it is important that information regarding drainage, site maintenance, trees, settlements, and effects of expansive soils be passed on to all interested/affected parties. The values tabulated above may not be appropriate to account for possible differential settlement of the slab due to other factors, such as excessive settlements. If a stiffer slab is desired, alternative Post-Tensioning Institute ([PTI] third edition) parameters may be recommended. All exterior columns not supported by the post-tensioned slab foundation should be supported by 24 square inch isolated footings extending at least 24 inches into approved compacted fill overlying suitable old paralic deposits. Exterior column footings should be tied to the post-tensioned slab foundation with 12 square inch, reinforced grade beams in at least two directions.

MAT-SLAB FOUNDATION

A mat-slab foundation may also be used to support the proposed residential buildings for the mitigation of expansive soils (E.I. $\geq$ 21 and P.I. $\geq$ 15). The project structural engineer may supercede the following recommendations based on the planned building loads and use. Wire Reinforcement Institute (WRI) methodologies (Snowden, 1981, 1996) may be used in the mat-slab foundation design.

For a mat-slab foundation bearing uniformly on approved compacted fill that has been placed directly upon tested and approved compacted fill overlying suitable, unweathered old paralic deposits, a maximum allowable net bearing capacity of 1,000 psf is recommended. Additional vertical bearing capacity up to 1,500 psf may be used for load-bearing perimeter and internal grade beams, incorporated into the mat-slab foundation, that have a minimum width of 12 inches and extend at least 12 inches below the lowest adjacent grade into tested and approved compacted fill overlying suitable old paralic deposits. These values may be increased by one-third for short-term loads including wind or seismic.

Mat-slab foundation reinforcement should be designed in accordance with local codes and structural considerations, including the intended use. The mat-slab foundation may be either ribbed or uniform thickness (UTF) with perimeter grade beams. In order to reduce soil moisture transmission between the interiors and exteriors of the mat-slab foundations, the perimeter grade beams should be at least 6 inches wide and extend a minimum of 18 or 24 inches below the lowest adjacent grade for slab subgrades that are medium to high in expansion potential, respectively. The need and arrangement of internal grade beams will be in accordance with the project structural engineer's recommendations. The passive resistance and lateral sliding resistance values recommended in the preceding "Post-Tensioned Slab Foundation Systems" section of this report should also be considered in the design of mat-slab foundations. The mat-slab foundation should support any columns or posts for overhang structures.

All exterior columns not supported by the mat-slab foundation should be supported by 24 square-inch isolated spread footings extending at least 24 inches below the lowest adjacent grade into approved compacted fill overlying suitable old paralic deposits. These exterior column footings should be tied to the post-tensioned slab foundation in at least two directions with reinforced grade beams that are at least 12 inches square in cross section.

The modului of subgrade reaction (K_S) and effective plasticity index (P.I.) for consideration in the mat-slab foundation design for a slab subgrade that is medium and high in expansion potential are presented in the following table:

MEDIUM EXPANSION (E.I. = 51-90)	HIGH EXPANSION (E.I. = 91-130)
K_S = 85 pci/inch, PI = 30 to 39	K_S = 70 pci/inch, PI = 40 to 45

The modulus of subgrade reaction is a unit value for a 1-foot square footing and should be reduced in accordance with the following equation when used with the design of larger foundations.

$$K_R = K_S\left[\frac{B+1}{2B}\right]^2$$

where: K_S = unit subgrade modulus
K_R = reduced subgrade modulus
B = foundation width (in feet)

Slab Subgrade Pre-Soaking

Slab subgrade pre-soaking should conform to the recommendations previously provided in the "Post-Tensioned Slab Foundation Systems" section of this report.

FOUNDATION SETBACKS

New foundations associated with the proposed development should be setback at least 25 feet landward of the nearby coastal bluff edge. The horizontal separation between the outside, bottom edges of foundations and any adjacent descending slopes within the subject parcels should not be less than 7 feet. Greater setbacks would be recommended if foundations occur in proximity to descending slopes with gradients steeper than 2:1 (h:v). This should be further evaluated during the grading plan review stage of project design.

FOUNDATION SETTLEMENT

Provided that the earthwork and foundation recommendations in this report are followed, shallow foundations bearing on tested and approved compacted fill overlying suitable old paralic deposits should be designed to accommodate a maximum total settlement of 1½ inches and a differential settlement of ¾-inch over a 40-foot horizontal span (angular distortion = 1/640).

SOIL MOISTURE TRANSMISSION CONSIDERATIONS

GSI has evaluated the potential for moisture or water vapor transmission through the proposed concrete slab-on-grade floors, in light of typical floor coverings and improvements. Slab moisture emission rates range from about 2 to 27 lbs/24 hours/1,000 square feet from a typical slab (Kanare, 2005), while floor covering manufacturers generally recommend about 3 lbs/24 hours as an upper limit. The recommendations in this section are not intended to preclude the transmission of moisture or water vapor through the building foundations or slab-on-grade floors. Foundation systems and concrete slab-on-grade floors shall not allow moisture or water vapor to enter into the structures so as to cause damage to another building component or to limit the installation of the type of flooring materials typically used for the particular application (State of California, 2022). These recommendations may be exceeded or supplemented by a "water proofing" consultant, the project architect, or the structural consultant. Thus, the client will need to evaluate the following in light of a cost vs. benefit basis (owner expectations and repairs/replacement), along with disclosure to all interested/affected parties. It should also be noted that moisture or water vapor transmission will occur in a new concrete slab-on-grade floor as a result of chemical reactions taking place within the curing concrete. Moisture or water vapor transmission through concrete floor slabs as a result of concrete curing has the potential to adversely affect sensitive floor coverings depending on the thickness of the concrete floor slab and the duration of time between the placement of concrete, and the floor covering installation. It is possible that a slab moisture sealant may be needed prior to the placement of sensitive floor coverings if a thick slab-on-grade floor is used and the time frame between concrete and floor covering placement is relatively short.

Considering the expansive nature of the onsite soils, the known soil conditions in the region, the anticipated typical moisture or water vapor transmission rates, floor coverings, and improvements (to be chosen by the client and project architect) that can tolerate moisture or water vapor transmission rates without significant distress, the following alternatives are provided:

- Construct a thicker concrete slab-on-grade floor.

- Concrete slab-on-grade floor underlayment should consist of a 15-mil vapor retarder, or equivalent, with all laps sealed per the 2019 CBC and the manufacturer's recommendations. The vapor retarder should comply with ASTM E 1745 - Class A criteria (i.e., Stego Wrap or approved equivalent), and be installed in accordance with ACI 302.1R-15 and ASTM E 1643.

- The 15-mil vapor retarder (ASTM E 1745 - Class A) should be installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.).

- The concrete slab-on-grade floor should be immediately underlain by a sand cushion consisting of 2 inches of clean sand (SE $\geq$ 30), placed atop a 15-mil vapor retarder (ASTM E-1745 -Class A, per Engineering Bulletin 119 [Kanare, 2005]) that is installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.). The manufacturer shall provide instructions for lap sealing, including minimum width of lap, method of sealing, and either supply or specify suitable products for lap sealing (ASTM E 1745), and per code.

  ACI 302.1R-15 (ACI, 2015) states, "Experience has shown, however, that the greatest level of protection for floor coverings, coatings, or building environments is provided when the vapor retarder/barrier is placed in direct contact with the slab. Placing concrete in direct contact with the vapor retarder/barrier eliminates the potential for water from sources such as rain, saw-cutting, curing, cleaning, or compaction to become trapped within the fill course. Wet or saturated fill above the vapor retarder/barrier can significantly lengthen the time required for a slab to dry to a level acceptable to the manufacturers of floor coverings, adhesives, and coatings. A fill layer sandwiched between the vapor retarder/barrier and the concrete slab-on-grade floor also serves as an avenue for moisture to enter and travel freely beneath the slab, which can lead to an increase in moisture within the slab once it is covered. Moisture can enter the fill layer through voids, tears, or punctures in the vapor retarder/barrier." Therefore, additional observation and testing will be necessary for the cushion or sand layer for moisture content, and relatively uniform thicknesses, prior to the placement of concrete.

  Conversely, ACI 302.1R-15 indicates that placing concrete directly upon the vapor retarder requires additional design and construction considerations to avoid potential slab-related problems, such as excessive concrete settlement and significantly larger length change during casting and drying shrinkage, and when the concrete is subject to environmental changes. In addition, dominant joint behavior can be made worse when the slab is placed in direct contact with the vapor retarder. Further, settlement cracking over reinforcing steel is more likely because of increased settlement resulting from a longer bleeding period. There is also a potential for enhanced slab curl. Lastly, if rapid surface drying conditions are present, the surface of the concrete (i.e., top fraction of an inch [millimeter]) placed directly upon the vapor retarder would have a greater propensity to dry and crust over leaving the underlying concrete relatively less stiff or unhardened. This may impact surface flatness of the concrete slab and result in blistering or delamination. Design and construction measures should be implemented to offset or reduce these effects.

Given the above, GSI recommends that all responsible parties participate in a risk/benefit evaluation regarding the specified location of the vapor retarder during project design.

- The vapor retarder should be directly underlain by a capillary break consisting of at least 4 inches of clean crushed gravel with a maximum dimension of ¾ inch (less than 5 percent passing the No. 200 sieve) that has been placed upon a properly compacted and moisture conditioned slab subgrade.

- Concrete used in the construction of the building foundations and slab-on-grade floors should have a maximum water to cement ratio (W/C) of 0.50. This does not supercede Table 19.3.2.1 of American Concrete Institute 318-14 ([ACI], 2014) for corrosion or other corrosive requirements. Additional concrete mix design recommendations should be provided by the structural consultant or waterproofing consultant. Concrete finishing and workability should be addressed by the structural consultant and a waterproofing consultant.

- Where slab water to cement ratios are as indicated herein, or admixtures used, the structural consultant should also make changes to the concrete in the grade beams and footings in kind, so that the concrete used in the foundation and floor slab is designed or treated for more uniform moisture protection.

- The owner should be specifically advised which building areas are suitable for tile flooring, vinyl flooring, or other types of moisture/vapor-sensitive flooring and which building areas are not suitable for these types of flooring applications. In all planned floor areas, flooring shall be installed per the manufacturer's recommendations.

- Additional recommendations regarding moisture or water vapor transmission should be provided by the architect/structural engineer/slab or foundation designer and should be consistent with the specified floor coverings indicated by the architect.

Regardless of the mitigation, some limited moisture/ water vapor transmission through the foundations and slab-on-grade floors cannot be entirely precluded and should be anticipated. Construction crews may require special training for installation of certain product(s), as well as concrete finishing techniques. The use of specialized product(s) should be approved by the slab designer, project architect, and the waterproofing consultant. A technical representative of the flooring contractor should review the slab and moisture retarder plans and provide comment prior to the construction of the foundation or improvement. The vapor retarder contractor should have representatives onsite during the initial installation.

PRELIMINARY RETAINING WALL DESIGN PARAMETERS

General

The following preliminary recommendations are provided for the design and construction of conventional masonry (concrete masonry unit [CMU]) and cast-in-place concrete [CIPC]) retaining walls. IA (2022a) proposes retaining wall heights up to approximately 23½ feet. Based on our experience, the economic limits of reinforced cantilever retaining wall design is approximately 19½ feet. Greater cantilever retaining wall heights typically require prestressing elements (i.e., soil nails or tieback anchors), bracing, or counterforts, as evaluated by the project structural engineer. Recommendations for specialty walls (i.e., crib, earthstone, mechanical stabilized earth [MSE] retaining walls, etc.) can be provided upon request, and would be based on site-specific conditions.

Conventional Retaining Walls

The design parameters provided below assume that either select materials (typically Class 2 permeable filter material or Class 3 aggregate base), or native onsite earth materials with an E.I. of 20 or less and a P.I. of 14 or less are used to backfill any retaining wall. It is unlikely that the onsite earth materials will meet this criteria. Thus, the importation of retaining wall backfill appears necessary at this time. In order to reduce lateral earth pressures acting upon the retaining walls, soils with an E.I. of 21 or greater and a P.I. of 15 or greater should not occur above a 1:1 (h:v) plane projected up and toward the retained soils from the heel of the retaining wall footing. Otherwise, the retaining walls will need to be designed for increased lateral earth pressures. The type of backfill (i.e., select or native), should be specified by the wall designer, and clearly shown on the retaining wall plans.

Waterproofing should also be considered for all retaining walls in order to reduce the potential for unsightly efflorescence staining, spalling stucco finishes, etc. In addition, waterstops should be used between all concrete and masonry joints.

Preliminary Retaining Wall Foundation Design

The preliminary foundation design for the proposed retaining walls should incorporate the following recommendations:

Minimum Footing Embedment - 24 inches below the lowest adjacent grade into tested and approved, compacted fill overlying suitable old paralic deposits. Footing embedment excludes the landscape layer (typically the upper 6 inches of soil) and any adjacent pavements. Where potentially compressible earth materials cannot be removed and recompacted below a 1:1 (h:v) plane projected down from the bottom, outboard edge of the proposed retaining wall footings, due to property boundaries or existing improvements that need to remain in service, the wall footing should be founded into suitable old paralic deposits. This will likely require a deepened retaining wall footing, based on the available subsurface data.

Minimum Footing Width - 24 inches.

Allowable Bearing Pressure - An allowable bearing pressure of 1,500 psf may be used in the preliminary design of retaining wall foundations provided that the footing maintains a minimum width of 24 inches and extends at least 24 inches below the lowest adjacent grade into tested and approved compacted fill, overlying suitable old paralic deposits or into suitable old paralic deposits. This pressure may be increased by one-third for transient short-term wind or seismic loads.

Passive Earth Pressure - Owing to the expansive characteristics of the onsite earth materials, a passive earth pressure of 100 psf/ft (pcf) with a maximum earth pressure of 1,000 psf may be used in the preliminary design of retaining wall foundations founded into tested and approved compacted fill materials overlying suitable old paralic deposits or into suitable old paralic deposits.

Lateral Sliding Resistance - A cohesion of 130 psf multiplied by the contact area may be used for lateral sliding resistance. The lateral sliding resistance should not exceed one-half of the dead load.

Backfill Soil Density - Backfill soil densities ranging between 125 pcf and 130 pcf may be used in the design of the proposed retaining walls. This assumes the use of granular backfill with an average compaction of at least 90 percent of the laboratory standard (per ASTM D 1557).

Footing Setbacks - All retaining wall footing setbacks from slopes should comply with Figure 1808.7.1 of the 2019 CBC. GSI recommends a minimum horizontal setback distance of 7 feet, as measured from the bottom, outboard edge of the footing to the face of descending slopes.

Restrained Walls

Any retaining wall that will be restrained prior to placing and compacting backfill material or retaining walls that have re-entrant or male corners, should be designed for an at-rest equivalent fluid pressure (EFP) of 55 pcf and 65 pcf for select and very low expansive backfill, respectively. For constrained conditions (such as property boundaries), where select or very low expansive backfill cannot be placed above a 1:1 (h:v) plane projected up from the heel of the restrained retaining wall footings, the EFP may range from 100 to 150 pcf for the native expansive soils within this zone. The design should include any applicable surcharge loading. For areas of male or re-entrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall (2H) laterally from the corner.

Cantilevered Walls

Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions due to traffic, structures, seismic events, or adverse geologic conditions. When wall configurations are finalized, the appropriate loading conditions for superimposed loads can be provided upon request.

For preliminary planning purposes, the structural consultant/wall designer should incorporate the surcharge of traffic on the back of retaining walls, if traffic will occur within "H" of the backside of the retaining walls, where "H" equals the retained soil height. The traffic surcharge may be taken as 100 psf/ft in the upper 5 feet of the wall for light passenger vehicle traffic (i.e., cars, pick-up trucks, etc.). Traffic surcharge from heavy-axle trucks (HS20) should be modeled as 300 psf/ft in the upper 5 feet of the wall. This does not include the surcharge of parked vehicles which should be evaluated at a higher surcharge to account for the effects of seismic loading.

The following table provides the recommended equivalent fluid pressures to be used in cantilever retaining wall design for level and 2:1 (h:v) sloping backfill conditions. For constrained conditions (such as property boundaries), where select or very low expansive backfill cannot be placed above a 1:1 (h:v) plane projected up from the heel of the cantilever retaining wall footings, the EFP may range from 100 to 150 pcf for the native expansive soils within this zone. These values assume level backfill conditions and would be higher for 2:1 (h:v) sloping backfill.

SURFACE SLOPE OF RETAINED MATERIAL (HORIZONTAL:VERTICAL)	EQUIVALENT FLUID WEIGHT P.C.F. (SELECT BACKFILL)[2]	EQUIVALENT FLUID WEIGHT P.C.F. (NATIVE BACKFILL)[3]
Level[1]	38	50
2 to 1	55	65

[1] Level backfill behind a retaining wall is defined as compacted earth materials, properly drained, without a slope for a distance of 2H behind the wall, where H is the height of the wall.
[2] SE $\geq$ 30, P.I. < 15, E.I. < 21, and $\leq$ 10% passing No. 200 sieve. Probably not sufficiently present onsite.
[3] E.I. = 0 to 50, SE $\geq$ 30, P.I. < 15, E.I. < 21, and $\leq$ 15% passing No. 200 sieve.

Design parameters for retaining walls less than 3 feet in height may be superceded by San Diego Regional Standard Design (SDRSD). SDRSD retaining walls require the use of select backfill materials (i.e., clean sand or gravel, or mixtures of the aforementioned with U.S.C.S. designations of GW, GP, SW, or SP) owing to the low equivalent fluid pressure used in their design. In addition, the use of standard design retaining walls are not permitted on sites where unstable coastal bluffs and unfavorable geologic structure with sloping topography (SDDSD, 2020). As previously indicated in the "Geologic Hazards Evaluation" section of this report, the SDDSD recognizes these geologic hazards within the subject parcels. Thus, for preliminary planning, it is recommended that the project architect and civil engineer contact the SDDSD regarding the feasibility of incorporating regional standard design retaining walls into the proposed project.

Seismic Surcharge

For retaining walls incorporated into the buildings, site retaining walls with more than 6 feet of retained materials, as measured vertically from the bottom of the wall footing at the heel to daylight; retaining walls that could present ingress/egress constraints for emergency vehicles and personnel in the event of failure; or retaining walls that could damage a nearby building upon failure, GSI recommends that the walls be evaluated for seismic surcharge in general accordance with 2019 CBC requirements. The retaining walls in this category should maintain an overturning Factor-of-Safety (FOS) of approximately 1.25 when the seismic surcharge (seismic increment), is applied. For restrained walls, the seismic surcharge should be applied as a uniform surcharge load from the bottom of the footing (excluding shear keys) to the top of the backfill at the heel of the wall footing. For cantilevered walls, the seismic surcharge should be applied as an inverted triangular pressure distribution for the portion of the wall located above 0.6H up from the bottom of the footing to the top of the retained soils, where "H" equals the retained soil height. For the evaluation of the seismic surcharge, the bearing pressure may exceed the static value by one-third, considering the transient nature of this surcharge. This is for local wall stability only.

This seismic surcharge may be taken as 25H, where "H" is the height of the retaining wall, as measured from the bottom of the footing. The 25H is derived from the guidelines set forth in City of Los Angeles Department of Building and Safety (LADBS) Information Bulletin Document No.: P/BC 2020-83 (LADBS, 2020), which are based on Seed and Whitman (1970).

$$\gamma_{EFP\ (seismic)} = \tfrac{3}{4} k_h \gamma_{soil}$$

Where:

$\gamma_{EFP\,(seismic)}$ is the seismic increment expressed as equivalent fluid pressure (pounds per cubic foot [pcf]);

k_h is the seismic lateral earth pressure coefficient equivalent to one-half of two-thirds of PGA_M (0.758 g x ⅔ x ½ = 0.254 g);

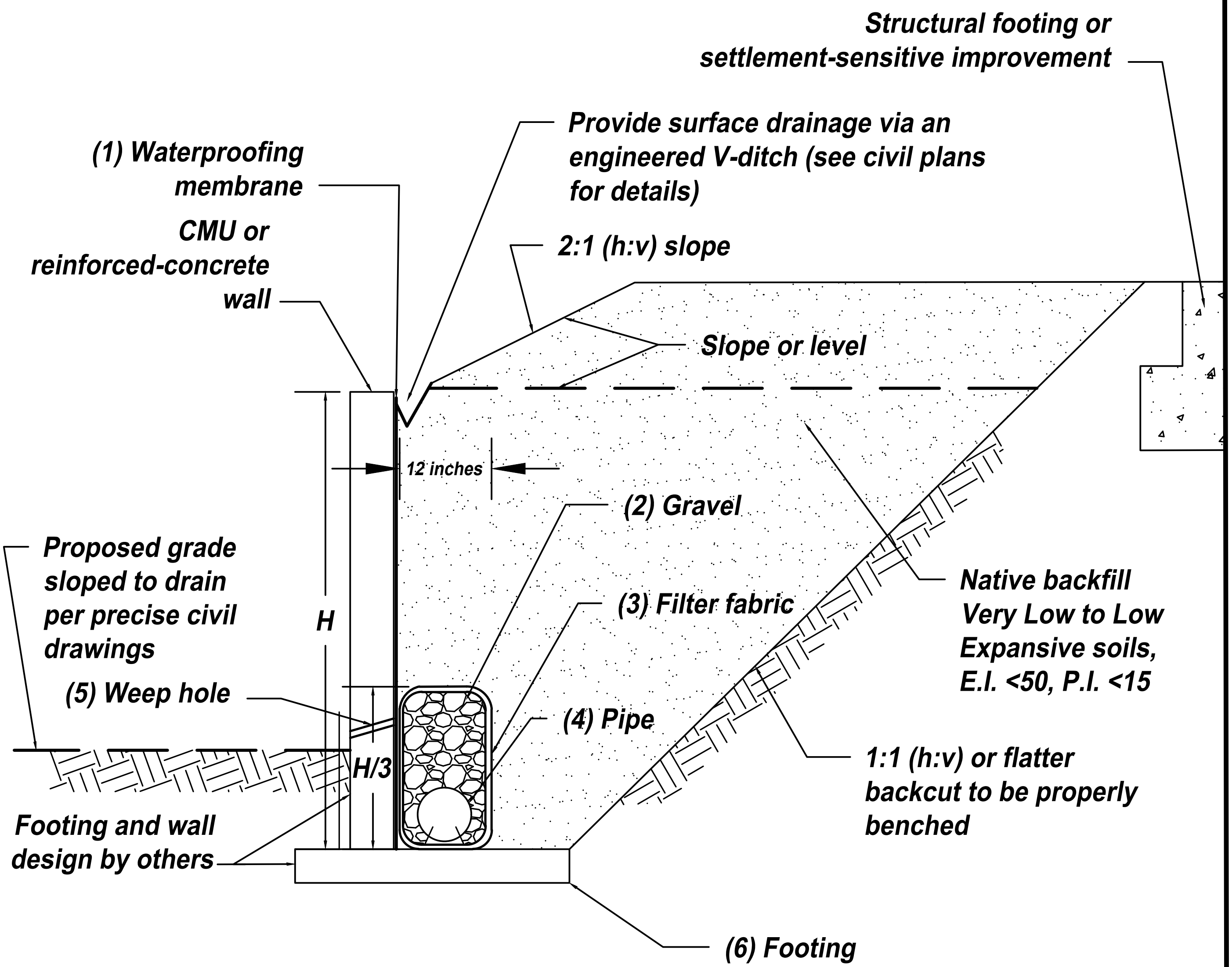
γ_{soil} is the total unit weight of the retained soils (130 pcf)

Thus, for the proposed retaining walls:

$$\gamma_{EFP\ (seismic)} = \tfrac{3}{4} \times \tfrac{1}{2} \times \tfrac{2}{3} \times 0.758 \times 130 \text{ pcf} = 24.8 \text{ pcf (use 25 pcf [25H])}$$

Retaining Wall Backfill and Drainage

Positive drainage should be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. A backdrain system is recommended for retaining walls that are 2 feet or greater in height. Details 1, 2, and 3, present the backdrainage options discussed below. Backdrains should consist of a 4-inch diameter perforated PVC or ABS drain pipe encased in either Class 2 permeable filter material or ¾-inch to 1½-inch gravel wrapped in approved filter fabric (Mirafi 140N or equivalent). The backdrain should flow via gravity (minimum 1 percent fall) toward an approved drainage facility, identified by the project civil engineer. For select backfill, the filter material should extend a minimum of 1 horizontal foot behind the base of the walls and upward at least 1 foot. For native backfill that has an E.I. of 20 or less and a P.I. of 14 or less, continuous Class 2 permeable drain materials should be used behind the wall. This material should be continuous (i.e., full height) behind the wall, and it should be constructed in accordance with the enclosed Detail 1 ("Alternative A"). For limited access and confined areas, (panel) drainage behind the wall may be constructed in accordance with Detail 2 (Alternative B). For more onerous expansive situations, backfill and drainage behind the retaining wall should conform with Detail 3 (Alternative C). Materials with an E.I. greater than 20 and P.I. greater than 14 should not be used as backfill for retaining walls. Otherwise, more rigorous wall design will be necessary. Retaining wall backfill should be uniformly moisture conditioned to at least optimum moisture content, placed in relatively thin lifts, and compacted to a minimum relative density of 90 percent of the laboratory standard (ASTM D 1557). Compaction of retaining wall backfill to at least 95 percent of the laboratory standard (ASTM D 1557) may be recommended if an overlying improvement spans between dense, unweathered old paralic deposits and retaining wall backfill.



(1) Waterproofing membrane.

(2) Gravel: Clean, crushed, $^3\!/_4$ to $1^1\!/_2$ inch.

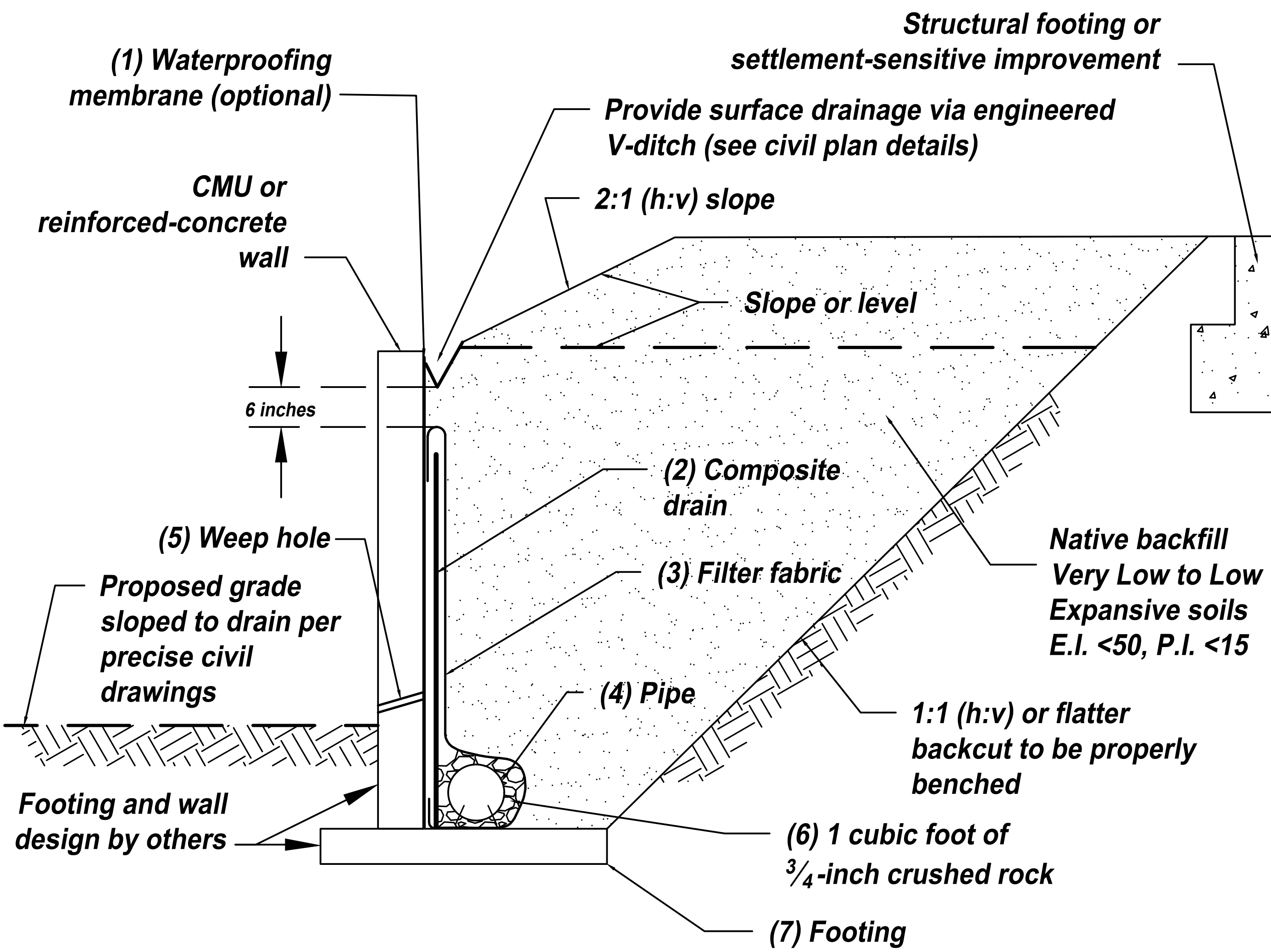
(3) Filter fabric: Mirafi 140N or approved equivalent.

(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient sloped to suitable, approved outlet point (perforations down).

(5) Weep holes: For CMU walls, Omit grout every other block, at or slightly above finished surface. For reinforced concrete walls, minimum 2-inch diameter weep holesspaced at 20 foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Footing: If bench is created behind the footing greater than the footing width using level fill or cut natural earth materials, an additional "heel" drain will likely be required by geotechnical consultant.

GeoSoils, Inc. GSI	RETAINING WALL DETAIL — ALTERNATIVE A	Detail 1



(1) Waterproofing membrane (optional): Liquid boot or approved mastic equivalent.

(2) Drain: Miradrain 6000 or J-drain 200 or equivalent for non-waterproofed walls; Miradrain 6200 or J-drain 200 or equivalent for waterproofed walls (all perforations down).

(3) Filter fabric: Mirafi 140N or approved equivalent; place fabric flap behind core.

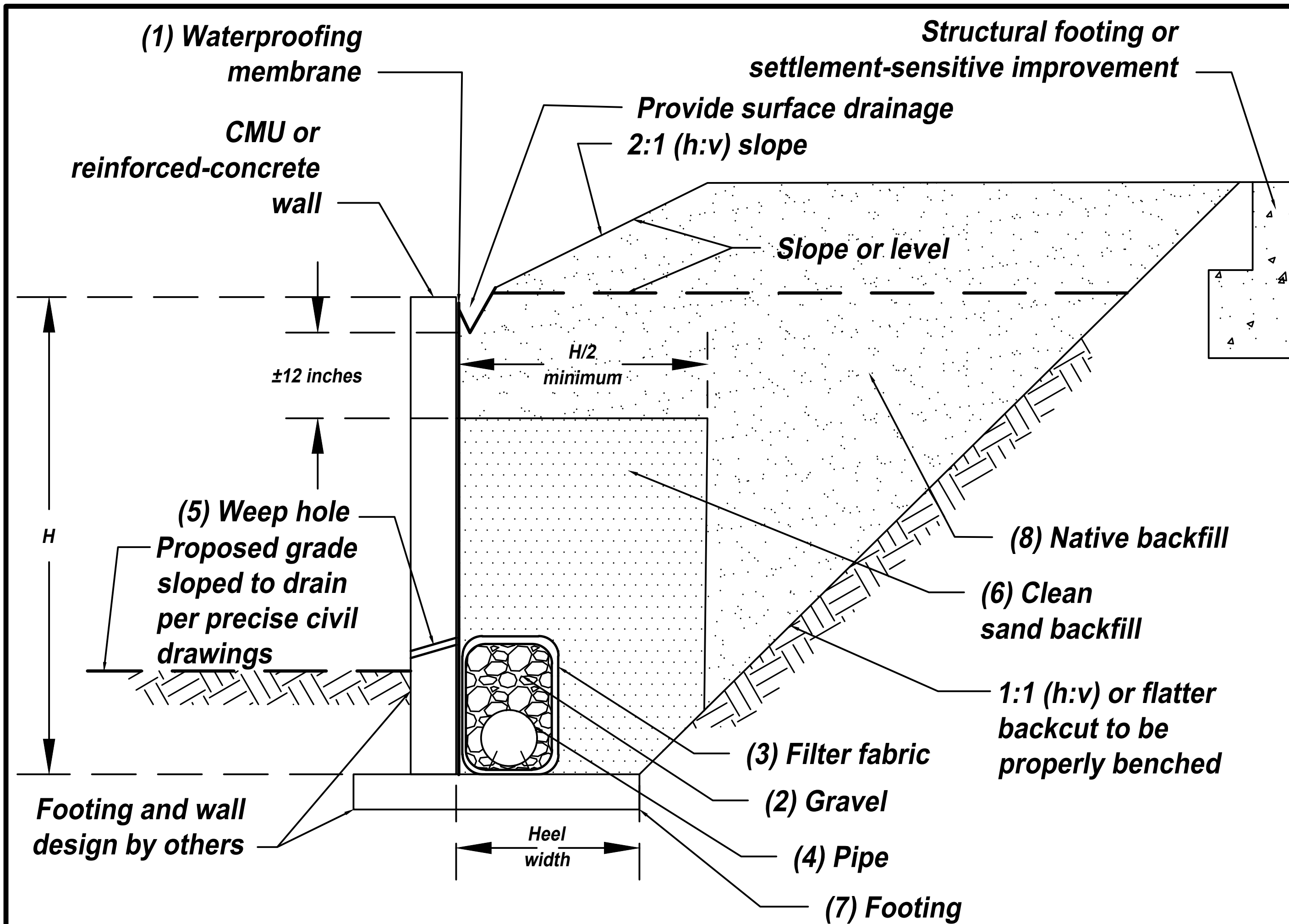
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(5) Weep holes: For CMU walls, Omit grout every other block, at or slightly above finished surface. For reinforced concrete walls, minimum 2-inch diameter weep holesspaced at 20 foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Gravel: Clean, crushed, ¾ to 1½ inch.

(7) Footing: If bench is created behind the footing greater than the footing width using level fill or cut natural earth materials, an additional "heel" drain will likely be required by geotechnical consultant.

GeoSoils, Inc.	RETAINING WALL DETAIL — ALTERNATIVE B	Detail 2



(1) Waterproofing membrane: Liquid boot or approved masticequivalent.

(2) Gravel: Clean, crushed, $^3/_4$ to $1^1/_2$ inch.

(3) Filter fabric: Mirafi 140N or approved equivalent.

(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(5) Weep hole: For CMU walls, Omit grout every other block, at or slightly above finished surface. For reinforced concrete walls, minimum 2-inch diameter weep holesspaced at 20 foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Clean sand backfill: Must have sand equivalent value (S.E.) of 35 or greater; can be densified by water jetting upon approval by geotechnical engineer.

(7) Footing: If bench is created behind the footing greater than the footing width using level fill or cut natural earth materials, an additional "heel" drain will likely be required by geotechnical consultant.

(8) Native backfill: If E.I. <21 and S.E. $\geq$35 then all sand requirements also may not be required and will be reviewed by the geotechnical consultant.

GeoSoils, Inc.	RETAINING WALL DETAIL — ALTERNATIVE C	Detail 3

Outlets should consist of a 4-inch diameter solid PVC or ABS drain pipe spaced no greater than approximately 100 feet apart, with a minimum of two outlets, one on each end of the wall. Discharge points should be non-erodible. The use of weep holes, only, in retaining walls higher than 2 feet is not recommended. The surface of the backfill should be sealed by pavement or the top 18 inches of the backfill should consist of compacted native soil (E.I. of 50 or less). Proper surface drainage should also be provided. For additional mitigation, consideration should be given to applying a water-proof membrane to the back of all retaining structures. The use of a waterstop should be considered for all concrete and masonry joints.

Wall/Retaining Wall Footing Transitions

Retaining walls are anticipated to be founded on footings designed in accordance with the recommendations in this report. Should wall footings transition from compacted fill to old paralic deposits, the wall designer may specify either:

a) A minimum of a 2-foot overexcavation and replacement of the old paralic deposits with compacted fill for a distance of 2H, from the point of transition. The overexcavation should be measured relative to the bottom of the wall footing.

b) Increase the amount of reinforcing steel and wall detailing (i.e., expansion joints or crack control joints) such that an angular distortion of 1/360 for a distance of 2H on either side of the transition may be accommodated. Expansion joints should be placed no greater than 20 feet on-center, in accordance with the structural engineer's/wall designer's recommendations, regardless of whether or not transition conditions exist. Expansion joints should be sealed with a flexible, non-shrink grout.

c) Embed the footings entirely into unweathered old paralic deposits (i.e., deepened footings).

If transitions from cut to fill transect the wall footing alignment at an angle of less than 45 degrees (plan view), then the designer should follow recommendation "a" (above) and until such transition is between 45 and 90 degrees to the wall alignment.

PRELIMINARY SHORING DESIGN AND CONSTRUCTION

Shoring of Excavations

Temporary cantilevered shoring systems, deriving passive support from reinforced soldier piles (i.e. typically steel "H" beams placed in drilled excavations and encased concrete, with timber, steel, or concrete lagging), can be used to retain adjacent property or existing improvements during the planned and remedial earthwork. Shoring of excavations is typically performed by specialty contractors with knowledge of the City of San Diego ordinances, and current building codes, as well as the local area soil conditions.

Since the design of shoring systems is sensitive to surcharge pressures behind the excavation, we recommend that this office be consulted if unusual load conditions are uncovered during the placement/installation of the shoring. To that end, GSI should perform field reviews during shoring construction. This would include logging the drilled excavations for soldier pile installation and periodic reviews of survey monitoring data. Care should be exercised when excavating into the onsite soils, especially near property lines and existing improvements since caving or sloughing of the earth materials or the displacement of any construction-related debris (if present) is possible. Special inspections/testing should be performed in accordance with the requirements of the shoring designer during shoring construction.

Shoring of the excavation is the responsibility of the contractor. Extreme caution should be undertaken to reduce damage to existing improvements caused by settlement or reduction of lateral support. Accordingly, we recommend a system of surveying and monitoring until the permanent design grade is achieved, in order to evaluate the effects of the shoring on the existing improvements the system is intended to retain. Pre-construction photographic and video documentation are also advised. Unless incorporated into the shoring design, construction equipment storage or traffic, and soil or construction material stockpiles should not occur within “H” of the top of any shored excavations (where ‘H’ equals the height of the retained earth). Temporary and permanent provisions should be made to direct any potential runoff away from the top of shored excavations. All applicable surcharge from vehicular traffic and existing structures within “H” of a shored excavation should be evaluated.

Lateral Earth Pressures for Shoring Design

1. Pressure diagrams showing the recommended application of active and passive earth pressures, and uniform and live load surcharges on temporary shoring systems are illustrated in Figure 7.

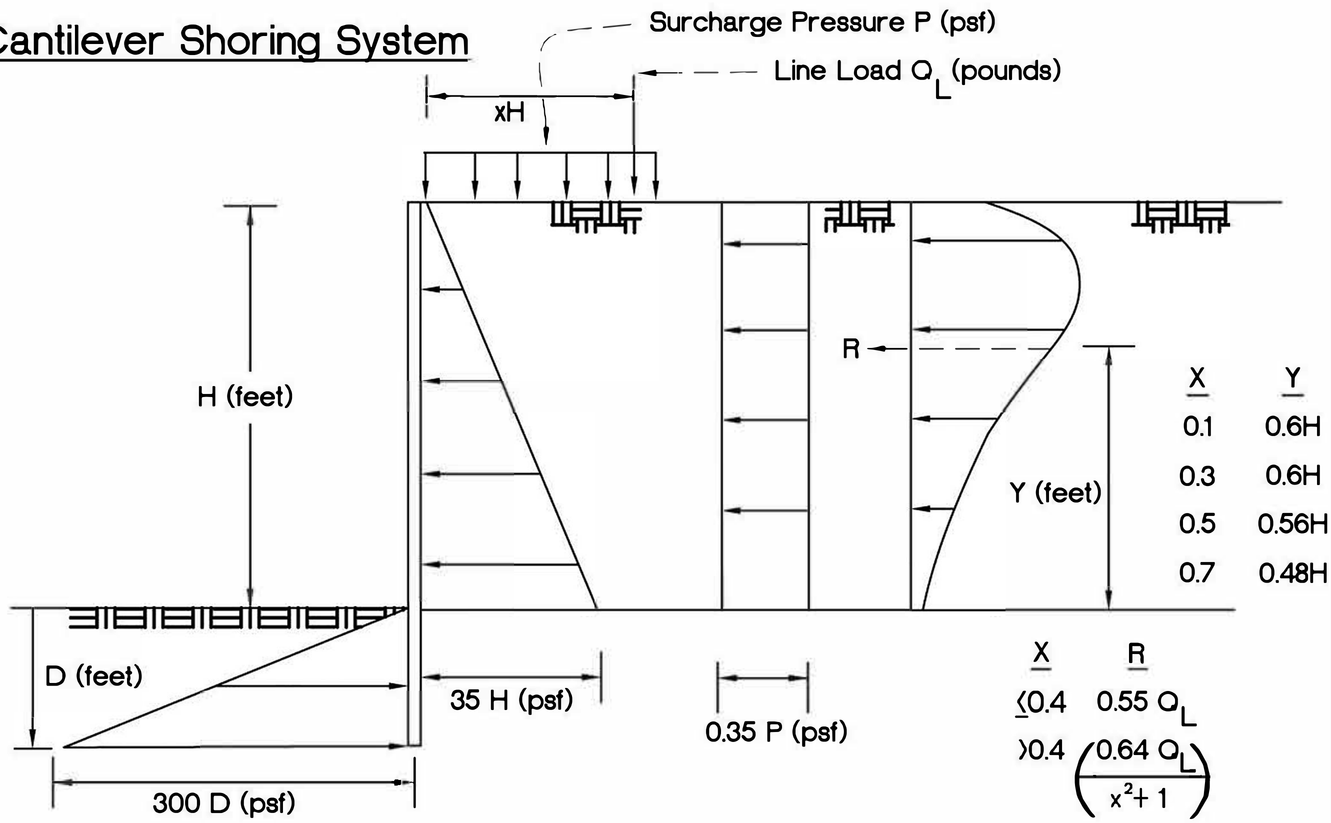
2. The active pressure to be used for temporary shoring design may be computed by the triangular pressure distribution shown in Figure 7.

3. Passive pressure for the design of temporary shoring may be computed as an equivalent fluid having a given density shown in Figure 7.

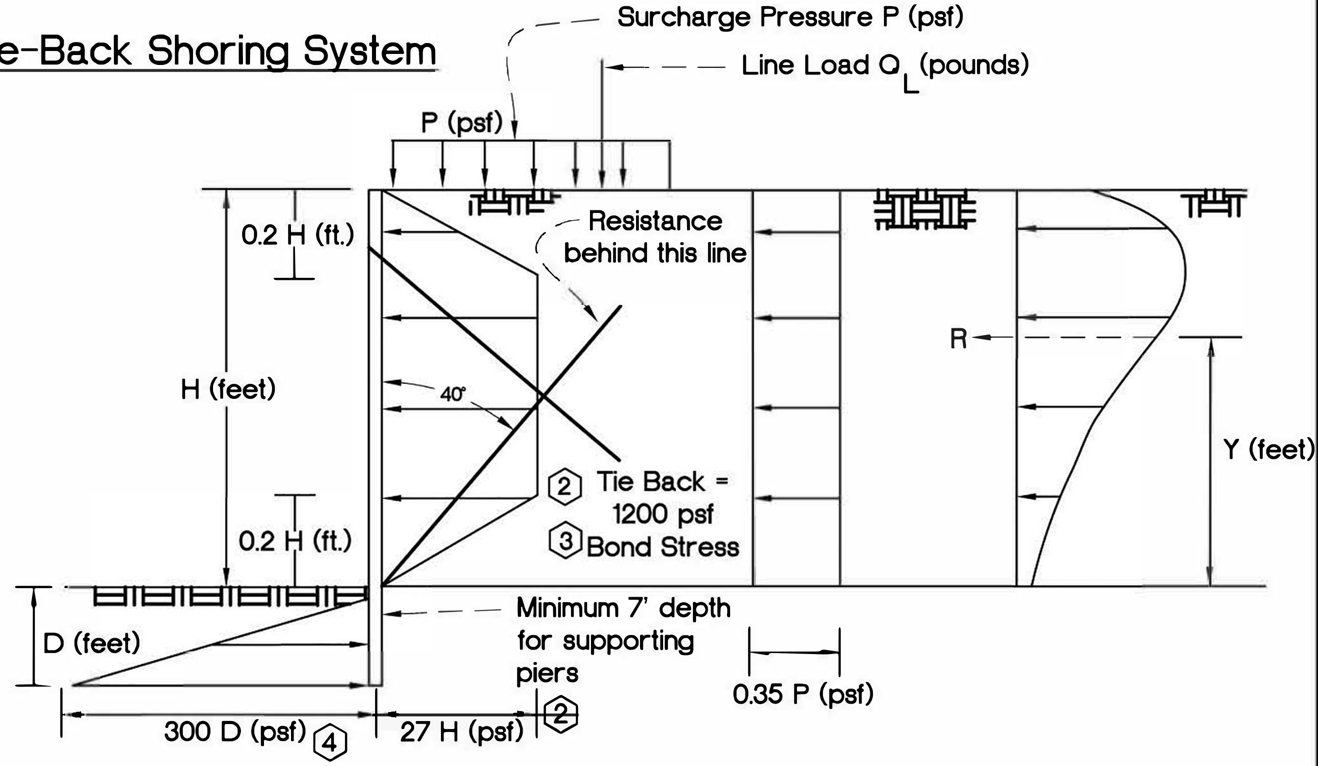
4. The above criteria assumes that hydrostatic pressure is not allowed to build up behind the shoring.

5. These recommendations are for a temporary shoring system with retained soil heights up to approximately 15 feet. Bracing or the use of rakers, or walers would likely be necessary for shoring of greater heights. The use of lateral resisting elements such as tieback anchors or soil nails are likely infeasible near property lines unless permission is granted to extend these components onto adjacent properties.

Cantilever Shoring System



Tie-Back Shoring System



NOTES

1. Include groundwater effects below groundwater level.
2. Include water effects below groundwater level.
3. Grouted length greater than 7 feet; field test anchor strength.
4. Neglect passive pressure below base of excavation to a depth of one pier diameter.

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LATERAL EARTH PRESSURES FOR TEMPORARY SHORING SYSTEMS

Figure 7

W.O. 8358-A-SC	DATE 08/22	SCALE None

6. An empirical equivalent fluid pressure approach may be used to compute the horizontal pressure against the shoring. Appropriate fluid unit weights are provided for specific slope gradients of the retained material; these do not include other superimposed loading conditions such as traffic, structures, seismic events, expansive soils or adverse geologic conditions. Traffic surcharge for shoring should be minimally applied as 100 psf/ft for light passenger vehicle traffic loads and 300 psf/ft for heavy (HS20) axle loads. The appropriate traffic surcharge should be applied in the upper 5 feet of the shoring if traffic will occur within “H” of the back of the wall (where “H” equals the height of the retained earth). It is not recommended to allow sloping surcharge (other than level backfill) within a distance of “H” behind the shoring system from either stockpiled soils, stockpiled building materials, or temporary/permanent graded slopes, where “H” equals the height of the retained earth. Steeper slope gradients (more than level) will increase the lateral earth pressure applied to the shoring and significantly increase the shoring design and costs. Regrading, if possible, is recommended prior to shoring installation to reduce the potential for sloping surcharge.

7. The shoring system should be designed such that the maximum lateral deformation at the top of the soldier pile does not exceed 1 inch. The maximum lateral deformation of the soldier piles at the lowest grade level (planned grade or bottom of remedial excavation) should not exceed ½ inch.

8. Walers, struts, tieback anchors, or soil nails may be used to reduce deflection. The design of these reinforcing elements should be provided by the project structural consultant/shoring designer.

Shoring Construction Recommendations

1. The excavation and installation of the soldier piles should be observed and documented by the project geotechnical consultant to further evaluate the geologic conditions within the influence of the shoring and to ensure the soldier pile construction conforms to the requirements of the shoring plan.

2. Drilled excavations for soldier piles should be straight and plumb. The contractor should periodically recheck the drilled excavation for plumbness, especially when oversized rock constituents, debris, or cemented formational materials are encountered. Less productive drilling in the Point Loma Formation should be anticipated.

3. Casing should be provided in drilled excavations if excessive groundwater or caving conditions are encountered. The bottom of the casing should be at least 4 feet below the top of the concrete as the concrete is placed and the casing is withdrawn. Dewatering may be required for concrete placement if significant seepage or groundwater is encountered during construction. This should be considered during project planning.

4. The exact tip elevation of the soldier piles should be clearly indicated on the shoring plans.

5. All concrete should be delivered through a tremie immediately after the approval of the drilled excavation and steel placement. Care should be taken to prevent striking the walls of the excavations with the tremie during concrete placement. Concrete should not be allowed to free fall more than 5 feet. "Tailgating" concrete is not recommended.

6. Proper spacing (<u>minimum of 3 inches</u>) between the steel reinforcement and the side walls, and bottoms of the drilled excavations should be provided.

7. Excavation for lagging should not commence until the soldier pile concrete reaches its design compressive strength.

8. The height of exposed soils during vertical excavation should not exceed 4 feet. The entire excavation should receive lagging prior to the end of each workday. Alternatively, loose soil may be used as a temporary buttress located below a 1:1 (h:v) plane projected down and toward the excavation from the top of the unsupported soils. No excavation should be left unsupported overnight.

9. A complete and accurate record of all soldier pile locations, depths, concrete, strengths, quantity of concrete per pile should be maintained by the special inspector and geotechnical consultant. The shoring design engineer should be notified of any unusual conditions encountered during installation.

Monitoring of Shoring

1. A pre-construction meeting should be held between the owner/developer, project general contractor, shoring contractor, civil engineer/surveyor, shoring designer, and geotechnical consultant to discuss shoring installation and pre-construction surveys/documentation.

2. The shoring designer or their designee should make periodic inspections of the construction site for the purpose of observing the installation of the shoring system and monitoring of the survey.

3. Monitoring points should be established at the top of selected soldier piles and at intermediate intervals as considered appropriate by the geotechnical engineer and the shoring design engineer.

4. Control points should be established outside the area of influence of the shoring system to ensure the accuracy of the monitoring readings.

5. Initial monitoring and photographic documentation of all existing improvements within 3H of the shoring, where "H" equals the retained soil height, should be performed prior to any excavation.

6. Once the excavation has commenced, periodic readings should be taken weekly until the excavation is backfilled to the design grade. If the performance of the shoring system is found to be within established guidelines, the shoring engineer may permit the periodic readings to be bi-weekly. Permission to conduct bi-weekly readings should be provided by the shoring design engineer in writing, and be distributed to the project geotechnical consultant, structural consultant, civil consultant, and the shoring contractor. Once initiated, bi-weekly readings should continue until the excavation is backfilled to the design grade. Thereafter, readings can be made monthly. Additional readings should be taken when requested by the special inspector, shoring design engineer, structural consultant, or geotechnical consultant.

7. Monitoring readings should be submitted to the shoring design engineer and geotechnical consultant within three (3) business days after they are conducted. Monitoring readings should be accurate to within 0.01 feet. Results are to be submitted in tabular format showing at least the initial date of monitoring and reading, current monitoring date and reading and the difference (delta) between the two readings.

8. If the total cumulative horizontal or vertical movement (from the start of shoring construction) of a nearby existing improvement or the soldier piles reaches ½ inch or 1 inch, respectively, all excavation activities should be suspended until the geotechnical consultant and the shoring design engineer determine the cause of movement and provide corrective measures, as necessary. Excavation should not re-commence until written permission is provided by the geotechnical consultant and the shoring design engineer. Supplemental shoring or an earthen buttress should be installed/placed to eliminate further movement. Supplemental shoring design will likely require review and approval by the building official. Excavation should not re-commence until written permission is provided by the building official.

Monitoring of Structures

1. The contractor should complete written and photographic logs of the improvements located within three times the retained soil height from the back of the shoring, prior to shoring construction. A licensed land surveyor or the project civil engineer should document all existing substantial cracks (i.e., greater than ⅛ inch horizontal separation or any vertical separation) in the adjacent improvements.

2. The contractor should monitor the existing buildings and improvements for movement or cracking that may result from the adjacent shored excavation.

3. If excessive movement or visible cracking occurs, the shoring contractor should stop work and shore/reinforce the excavation in accordance with the geotechnical consultant's recommendations, and contact the shoring design engineer.

4. Monitoring of the existing, adjacent buildings and improvements should be made at reasonable intervals, as required by the shoring design engineer. Monitoring should be performed by a licensed land surveyor or the project civil engineer.

5. If in the opinion of the shoring design engineer, monitoring data indicate excessive movement or other distress, all excavation should cease until the geotechnical engineer and the shoring design engineer investigates the situation and make recommendations for remedial actions or allows continuation of work.

6. All readings and measurements should be submitted to the shoring design engineer and the geotechnical consultant.

Deeper Excavations Adjacent to Shoring Systems

If for any reason, planned or remedial excavations are needed to extend below the planned cut depth of shoring systems, the following recommendations should be followed.

1. Any excavation extending below a 2:1 (h:v) plane projected down from the planned cut depth elevation at the face of the shoring should be completed using slot excavations as previously recommended herein. The slot excavations should not extend to depths greater than 10 feet or below areas of saturated soils, groundwater seepage, or running sands without prior evaluations by the project geotechnical consultant.

2. Survey monitoring should be performed on a continuous basis while excavations are being conducted.

PRELIMINARY ASPHALTIC CONCRETE OVER AGGREGATE BASE (AC/AB) PAVEMENT DESIGN/CONSTRUCTION

In order to evaluate the preliminary design of the structural section for asphaltic concrete pavements, GSI has assumed a Traffic Index (T.I.) of 4.5 for the proposed driveway. Owing to the fine-grained nature of most of the onsite earth materials, it is our opinion that a subgrade resistance value (R-value) of 5 is appropriate for preliminary design purposes. Preliminary recommendations for the structural section of an asphaltic concrete driveway relative to the assumed T.I. and R-value are provided in the table below. The actual T.I. for the driveway should be evaluated and confirmed by the project civil engineer or traffic engineer. Final pavement design should be based on the R-value test results of the soils located near the pavement subgrade, following grading and underground utility trench backfill.

VEHICULAR TRAFFIC CLASSIFICATION	TRAFFIC INDEX (T.I.)[1]	STANDARD PAVEMENT DESIGN		
		R-VALUE[2]	AC INCHES	CLASS 2 AGGREGATE BASE[3] INCHES
Driveway	4.5	5	3.0	8.0
Driveway (Alternative Section)	4.5	5	4.0	6.0

1 - Assumed T.I. To be confirmed by the project civil engineer or traffic engineer.
2 - Assumed subgrade R-value to be re-evaluated at the conclusion of grading and underground utility backfill.
3 - Assumed R-value for Class 2 aggregate base R=78 - Caltrans standard Class 2 Aggregate Base.

PRELIMINARY VEHICULAR PORTLAND CEMENT CONCRETE PAVEMENT (PCCP) DESIGN

Preliminary recommendations for the design of vehicular PCCP are provided in the table below.

PORTLAND CONCRETE CEMENT PAVEMENTS (PCCP)					
TRAFFIC AREAS	CONCRETE TYPE	PCCP THICKNESS (inches)	TRAFFIC AREAS	CONCRETE TYPE	PCCP THICKNESS (inches)
Light Vehicles	520-C-2500	7.0	Heavy Truck Traffic	520-C-2500	9.0
	560-C-3250	6.0		560-C-3250	8.0

NOTE: All PCCP is designed as un-reinforced and bearing directly on compacted subgrade. However, a 6-inch thick layer of compacted Class 2 aggregate base may be considered for increased performance. All PCCP should be properly detailed (jointing, etc.) per the industry standard. Pavements may be additionally reinforced with #4 reinforcing bars, placed 12 inches on center, each way, for improved performance. Trash truck loading pads (aprons) shall adhere to the City of San Diego's minimum thickness and detailing.

OTHER CONSIDERATIONS REGARDING VEHICULAR PAVEMENT DESIGN

The recommended pavement sections provided above are intended as minimum guidelines. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. If the ADT (average daily traffic) or ADTT (average daily truck traffic) increases beyond that intended, as reflected by the T.I. used for design, increased maintenance and repair could be required for the pavement section. Consideration should be given to the increased potential for distress from overuse of paved street areas by heavy equipment or construction related heavy traffic (e.g., telehandlers, concrete trucks, loaded supply trucks, etc.). Best management construction practices should be followed at all times, especially during inclement weather.

VEHICULAR PAVEMENT SECTION CONSTRUCTION

General

The following recommendations should be incorporated into the construction of vehicular pavements:

Pavement Subgrade

The recommended remedial grading should occur within the vehicular pavement areas prior to subgrade preparation. The pavement subgrade should be free of any loose materials, scarified at least 6 to 8 inches, uniformly moisture conditioned to the soil's optimum moisture content, and then compacted to a minimum relative density of 95 percent of the laboratory standard (per ASTM D 1557). The pavement subgrade should be proof-rolled under the observation of the geotechnical consultant prior to placing the Class 2 aggregate base. Field density tests should be performed during the compaction of the pavement subgrade.

Class 2 Aggregate Base

The Class 2 aggregate base should be placed in lifts not exceeding 6 inches, uniformly moisture conditioned to at least optimum moisture content, and compacted to a minimum relative density of 95 percent of the laboratory standard (per ASTM D 1557). Field density tests should be performed during the compaction of the aggregate base layer. Base aggregate should be in accordance to the Caltrans or "Greenbook" specifications for Class 2 base rock (minimum R-value=78).

Asphaltic Concrete

Asphaltic concrete paving should conform to the standards in Section 302-5 of the 2021 "Greenbook" (BNI Publications, Inc., 2021). Geotechnical observations and field density testing should be conducted during asphaltic concrete paving. The asphaltic concrete should be compacted to a minimum of 95 percent of the density obtained on samples tested in accordance with California Test Methods 304 and 308, Method "A." Method "C" may be used if the absorption of the compacted specimen is less than 2 percent.

Prime coat may be omitted if all of the following conditions are met:

1. The asphaltic concrete pavement layer is placed within two weeks of completion of the aggregate base course.

2. Traffic is not routed over the completed aggregate base course before paving.

3. Construction is completed during the dry season of May through October.

4. The aggregate base is kept free of debris prior to placement of the asphaltic concrete.

If construction is performed during the wet season of November through April, prime coat may be omitted if no rain occurs between completion of the aggregate base course and paving, and the time between completion of the base and paving is reduced to three (3) days, provided the aggregate base is free of loose soil or debris. Where prime coat has been omitted and rain occurs, traffic is routed over the aggregate base course, or paving is delayed, measures shall be taken to restore the base course, and the subgrade to conditions that will meet specifications as directed by the City of San Diego or recommended by the geotechnical consultant.

FLATWORK AND OTHER IMPROVEMENTS

Most of the onsite earth materials exhibit expansive characteristics. The effects of expansive soils are cumulative, and typically occur over the lifetime of any improvement. On relatively level areas, when the soils are allowed to dry, the desiccation and swelling process tends to cause heaving and distress to flatwork and other improvements. The resulting potential for distress to improvements may be reduced, but not totally eliminated. To that end, it is recommended that the long-term potential for distress be communicated to any interested/affected parties. To reduce the likelihood of distress, the following recommendations are presented for all exterior concrete flatwork:

1. Exterior concrete slabs-on-grade should be cast entirely on properly compacted fill materials that have been approved by the geotechnical consultant or suitable, unweathered old paralic deposits. The subgrade area for the concrete slabs should be compacted to achieve a minimum 90 percent relative compaction (per ASTM D 1557), and then be presoaked to 2 to 3 percentage points above the soils' optimum moisture content to a depth of 18 inches below the subgrade. This moisture content should be maintained in the subgrade soils during concrete placement to promote uniform curing of the concrete and to reduce the development of unsightly shrinkage cracks.

2. The exterior concrete slabs-on-grade should be cast over a non-yielding surface consisting of a 4-inch layer of crushed rock, gravel, or clean sand that should be compacted and level prior to placing concrete.

3. Exterior concrete slabs-on-grade that will receive pedestrian traffic should be a minimum of 4 inches thick. Driveway approach slabs or other concrete slabs, adjacent to landscape areas, that will receive vehicular traffic should include a thickened edge extending at least 12 inches below the subgrade to help impede the transmission of landscape water under the slab.

4. The use of transverse and longitudinal control joints are recommended to help control slab cracking due to concrete shrinkage or expansion. Two ways to mitigate such cracking are: a) add a sufficient amount of reinforcing steel, increasing tensile strength of the slab; and b) provide an adequate amount of control or expansion joints to accommodate the anticipated concrete shrinkage and expansion.

   In order to reduce the potential for unsightly cracks, exterior concrete slabs-on-grade should be reinforced at mid-height with a minimum of No. 3 bars placed at 18 inches on center, in each direction. The exterior slabs should be scored or saw cut, ½ to ⅜ inches deep, often enough so that no section is greater than 10 feet by 10 feet. For sidewalks or narrow slabs, control joints should be provided at intervals of every 5 feet. The slabs should be separated from the foundations and sidewalks with expansion joint filler material.

5. No traffic should be allowed upon the new concrete slabs until they have been properly cured to within 75 percent of design strength. Concrete compression strength for concrete slabs that will only receive pedestrian traffic should be a minimum of 2,500 psi.

6. Driveways, sidewalks, and patio slabs adjacent to the proposed buildings should be separated from the structures with thick expansion joint filler material. In areas directly adjacent to a continuous source of moisture (i.e., irrigation, planters, etc.), all joints should be additionally sealed with flexible mastic.

7. Planters and walls should not be tied to the proposed buildings.

8. Overhang structures should be supported on the slabs, or structurally designed with continuous footings tied to the perimeter building foundation(s) in at least two directions.

9. Any masonry landscape or site retaining walls that are to be constructed throughout the property should be grouted and articulated in segments no more than 20 feet long. These segments should be keyed or doweled together.

10. Positive site drainage should be maintained at all times. Finish grade on the property should provide a minimum of 1 to 2 percent fall to the street, as indicated herein or conform to Section 1804.3 of the 2019 CBC (whichever is more conservative). It should be kept in mind that drainage reversals could occur, including post-construction settlement, if relatively flat yard drainage gradients are not periodically maintained by the property owner. This should be disclosed to all interested/affected parties.

11. Air conditioning (A/C) units should be supported by concrete slabs that are incorporated into the building foundation or constructed on a rigid slab with flexible couplings for plumbing and electrical lines. A/C waste water lines should be drained to a suitable non-erodible outlet.

12. Shrinkage cracks could become excessive if proper finishing and curing practices are not followed. Finishing and curing practices should be performed per the Portland Cement Association Guidelines. Mix design should incorporate rate of curing for climate and time of year, sulfate content of soils, corrosion potential of soils, and fertilizers used on site.

PRELIMINARY OUTDOOR POOL/SPA DESIGN RECOMMENDATIONS

The following preliminary recommendations are provided for consideration in outdoor swimming pool/spa design and planning. Recommendations for a swimming pool, spa, and associated deck flatwork in areas with differential settlements exceeding ¼ inch over 40 feet horizontally, will be more onerous than the preliminary recommendations presented below. If segmental retaining walls will be included with the project, the proposed swimming pool/spa and associated deck flatwork should be located below a 1:1 (h:v) plane projected up and toward the retained soils from the heel of the wall facing and the heel of the geogrid-reinforced backfill, owing to strain incompatibilities between these improvements.

General

1. Owing to the expansive nature of most of the onsite earth materials, it is recommended that the entire bottoms and sides of the pool/spa shells and their foundations be surrounded by at least 5 feet of tested and approved compacted fill that is very low in expansion potential (E.I. of 20 or less and P.I. of 14 or less) overlying suitable unweathered old paralic deposits. Pool deck slabs-on-grade should be underlain by a similar compacted fill mat that extends at least 3 feet below the slab subgrade and at least 3 feet horizontally outside the outboard edges of the deck slab. Based on the available subsurface data, the recommended compacted fill mat will require overexcavation and replacement of the old paralic deposits. The bottom of the overexcavation should be sloped away from the pool/spa area.

2. An allowable bearing value of 1,500 psf may be assumed for continuous footings, a minimum of 12 inches wide and embedded at least 12 inches below the lowest adjacent grade into tested and approved compacted fill overlying suitable old paralic deposits. Footing embedment excludes soft soils, landscape zones, slab and underlayment sections, etc. The compacted fill material should be very low in expansion potential (E.I. of 20 or less and P.I. of 14 or less).

3. The equivalent fluid pressure to be used for the swimming pool/spa design should be 60 pcf for pool/spa walls with level backfill, and 75 pcf for 2:1 (h:v) sloping backfill conditions. In addition, backdrains should be provided behind pool/spa walls subjacent to slopes. Alternatively, the pool/spa walls may be designed for full hydrostatic pressure by adding 62.4 pcf to the equivalent fluid pressures recommended above for drained conditions.

4. If laterally supported by compacted fill that is very low in expansion potential, the passive earth pressure used in the design of the pool/spa foundations may be computed as an equivalent fluid having a density of 250 pcf, with a maximum lateral earth pressure of 2,500 pounds per square foot (psf).

5. An allowable coefficient of friction between very low expansive soil and concrete of 0.35 may be used with the dead load forces.

6. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

7. Where pools/spas are planned near structures, appropriate surcharge loads need to be incorporated into design and construction by the pool/spa designer. This includes, but is not limited to landscape berms, decorative walls, footings, built-in barbeques, utility poles, etc.

8. All pool/spa walls should be designed as "free standing" and be capable of supporting the water in the pool/spa without soil support. The shape of the pool/spa in cross section and plan view may affect the performance of the pool/spa, from a geotechnical standpoint. Pools and spas should also be designed in accordance with the latest adopted Code. The bottoms of the pools/spas, should maintain a distance H/3, where H is the height of the slope (in feet), from the slope face. This distance should not be less than 7 feet, nor need not be greater than 40 feet.

9. Hydrostatic pressure relief valves should be incorporated into the pool and spa designs.

10. All fittings and pipe joints, particularly fittings in the side of the pool/spa, should be properly sealed to prevent water from leaking into the adjacent soils materials, and be fitted with slip or expandible joints between connections transecting varying soil conditions.

11. An elastic expansion joint (flexible waterproof sealant) should be installed to reduce the potential for water to infiltrate into the soil at all deck joints.

12. A reinforced grade beam should be placed around skimmer inlets to provide support and mitigate cracking around the skimmer face.

13. In order to reduce unsightly cracking, deck slabs should be a minimum of 4 inches thick, and be reinforced with No. 3 reinforcing bars at 18 inches on-center, in two perpendicular directions. All slab reinforcement should be supported by chairs to ensure proper mid-slab positioning during the placement of concrete. "Hooking" of the steel reinforcement is not recommended. Wire mesh reinforcing should not be used. Deck slabs should not be tied to the pool/spa structures. The deck subgrade should be lightly moisturized immediately prior to concrete placement to promote uniform curing and to reduce the potential for the loss of concrete moisture following placement. For increased performance, the deck slab underlayment should consist of a 1- to 2-inch thick leveling course of sand (S.E. > 30) and a minimum of 4 to 6 inches of Class 2 aggregate base compacted to a minimum relative density of 90 percent of the laboratory standard. Deck slabs within the H/3 zone, where H is the height of the slope (in feet), will have an increased potential for distress relative to other areas outside of the H/3 zone. If distress is undesirable, improvements, deck slabs or flatwork should not be constructed closer than H/3 or 7 feet (whichever is greater) from the slope face, in order to reduce, but not eliminate, this potential.

14. In order to reduce unsightly cracking, the outer edges of the pool/spa deck slab that is bordered by landscaping, and the edges immediately adjacent to the pool/spa, should be underlain by an 8-inch wide concrete cutoff shoulder (thickened edge) extending to a depth of at least 12 inches below the bottoms of the deck slab to mitigate excessive infiltration of water under the pool/spa deck slab. These thickened edges should be reinforced with two No. 4 bars, with one bar near the top and one bar near the bottom of the thickened edges.

15. Surface and shrinkage cracking of the finished pool/spa shells and deck slab may be reduced if the shotcrete/concrete has a low slump and water-to-cement ratio that are maintained during placement. Concrete used should have a minimum compressive strength of 4,000 psi. Excessive water added to concrete prior to placement is likely to cause shrinkage cracking, and should be avoided. Some shrinkage cracking, however, is unavoidable.

16. Joint and sawcut locations for the pool/spa deck slab should be determined by the design engineer and contractor. However, spacings should not exceed 6 feet on center.

17. Considering the nature of the onsite earth materials, it should be anticipated that caving or sloughing could be a factor in subsurface excavations and trenching. Shoring or excavating the trench walls/backcuts at the angle of repose (typically 25 to 45 degrees), should be anticipated. <u>All</u> excavations should be observed by a licensed engineering geologist or engineer, prior to workers entering the

excavation or trench, and minimally conform to the recommendations for temporary slopes previously provided in this report, as well as CAL/OSHA and local safety codes. Should adverse conditions exist, appropriate recommendations should be offered at that time by the geotechnical consultant.

18. It is imperative that adequate provisions for surface drainage are incorporated by the homeowners into their overall improvement scheme. Positive surface drainage should be maintained over the life of the proposed development. Ponding water, ground saturation and flow over slope faces, are all situations which must be avoided to enhance long-term performance of the pool/spa and associated improvements, and reduce the likelihood of distress.

19. If the pool/spa ever require emptying, it should be done in accordance with the recommendations of the pool/spa designer.

20. The temperature of the water lines for spas and pools may affect the corrosion properties of the onsite soils. Thus, a corrosion specialist should be retained to review all spa and pool plans, and provide mitigative recommendations, as warranted. Concrete mix design should be reviewed by a qualified corrosion consultant and materials engineer. The swimming pool/spa designer should also consider the affects of sea spray from the nearby Pacific Ocean.

21. All pool/spa underground utility trenches should be compacted to at least 90 percent of the laboratory standard, under the full-time observation and field density testing of the geotechnical consultant. Underground utility trench bottoms should be sloped away from the primary structures on the property (typically the residential structures).

22. Pool and spa underground utility lines should not cross the primary structures' underground utility lines (i.e., not stacked, or sharing of trenches, etc.).

23. The pool/spa or associated underground utilities should not intercept, interrupt, or otherwise adversely impact any area drain, roof drain, or other drainage conveyances. If it is necessary to modify, move, or disrupt existing area drains, subdrains, or tightlines, then the design civil engineer should be consulted, and mitigative measures provided. Such measures should be further reviewed and approved by the geotechnical consultant, prior to proceeding with any further construction.

24. The geotechnical consultant should review and approve all aspects of the pool/spa and flatwork design prior to construction. A design civil engineer should review all aspects of such design, including drainage and setback conditions. Prior to acceptance of the pool/spa construction, the project builder, geotechnical consultant and civil designer should evaluate the performance of the area drains and other site drainage pipes, following pool/spa construction.

25. All aspects of construction should be reviewed and approved by the geotechnical consultant, including during excavation, prior to the placement of any additional fill, and prior to the placement of any steel reinforcement and concrete.

26. Any changes in the design or location of the pool/spa should be reviewed and approved by the geotechnical and design civil engineer prior to construction. Field adjustments should not be allowed until written approval of the proposed field changes are obtained from the geotechnical and design civil engineer.

27. Disclosure should be made to all builders, contractors, and any interested/affected parties, that pools/spas built within about 15 feet of the top of a slope, and H/3, where "H" is the height of the slope <u>will</u> experience some movement or tilting. While the pool/spa shell or coping may not necessarily crack, the levelness of the pool/spa will likely tilt toward the slope, and may not be aesthetically pleasing. The same is true with decking, flatwork and other improvements in this zone.

28. Failure to adhere to the above recommendations will significantly increase the potential for distress to the pool/spa, flatwork, etc.

29. Local seismicity or the design earthquake will cause some distress to the pool/spa and decking or flatwork, possibly including total functional and economic loss.

30. The information and recommendations discussed above should be provided to any contractors and subcontractors, or homeowners, interested/affected parties, etc., that may perform or may be affected by such work.

STRUCTURAL CONCRETE MIX DESIGN

The project architect, structural engineer, and civil engineer should review the results of the corrosion tests provided in the "Laboratory Testing" section of this report and specify the appropriate mix design for structural concrete on their respective plans. The effects of sea spray from the nearby Pacific Ocean should also be considered in the mix design of structural concrete.

PERMANENT POST-CONSTRUCTION STORM WATER BEST MANAGEMENT PRACTICES

As previously indicated herein, infiltration of storm water into the onsite soils for permanent post-construction storm water BMPs is not recommended from a geotechnical perspective. Since storm water infiltration into the onsite soils is not advised, any proposed permanent post-construction storm water BMP should consist of a fully contained system or storm water filtration, or detention basins should receive an impermeable liner and an under-drain system.

Impermeable liners used in conjunction with storm water basins should consist of a 30-mil polyvinyl chloride (PVC) membrane that is covered by a minimum of 12 inches of clean soil, free from rocks and debris. The impermeable liner should extend a few inches above the 100-year flood elevation (Q_{100} elevation). In addition, the design and construction of the proposed storm water detention basin should consider the following:

1. The 30-mil impermeable liner should have the following minimum engineering properties:

   Specific Gravity (ASTM D792): 1.2 (g/cc, min.); Tensile (ASTM D882): 73 (lb/in-width, min); Elongation at Break (ASTM D882): 380 (%, min); Modulus (ASTM D882): 30 (lb/in-width, min.); and Tear Strength (ASTM D1004): 8 (lb/in, min); Seam Shear Strength (ASTM D882) 58.4 (lb/in, min); Seam Peel Strength (ASTM D882) 15 (lb/in, min).

2. Subdrains for the under-drain system should consist of a minimum 4-inch diameter Schedule 40 or SDR 35 perforated drain pipe with the perforations oriented down. The drain pipe should be sleeved with filter sock or wrapped in filter fabric (Mirafi 140N or approved equivalent).

3. Areas adjacent to, or within, the storm water basins that are subject to inundation should be properly protected against scouring, undermining, and erosion, in accordance with the recommendations of the design engineer.

4. Long-term stability of the basin slopes will require them to be constructed at gradients no steeper than 4:1 (h:v). Alternatively, the sides of the basin may be supported by retaining structures/walls designed for the appropriate earth and hydrostatic pressures. Footings for the retaining walls should extend at least 2 feet below the bottom of the basin into earth materials deemed suitable for bearing by the project geotechnical consultant. Refer to the "Retaining Wall Design Parameters" section of this report for other geotechnical recommendations for the design and construction of retaining walls.

5. Due to the potential for piping and adverse seepage conditions, a burrowing rodent control program should also be implemented onsite.

6. Any trenches for inlet/outlet piping or other subsurface utilities, located within or near the proposed basins may become saturated and induce backfill settlement. This is due to the potential for piping, water migration, or seepage along the trench line backfill. Underground utility trenches adjacent to and within basins, should be backfilled with a 1-sack sand-cement slurry.

7. Separation geotextiles or slurry backfill should be used to reduce the potential for the piping of fine soil particles into open-graded gravel backfill layers in the trenches.

8. The use of storm water basins above or near existing or planned underground utilities that might degrade/corrode with the introduction of water/seepage should be avoided. Alternatively, a corrosion consultant may provide recommendations for corrosion protection.

9. Basins should not occur below a 1:1 (h:v) plane projected down and away from foundations or within 50 feet of the tops and toes of slopes.

10. The use of storm water basins above or near existing or planned underground utilities that might degrade/corrode with the introduction of water/seepage should be avoided. Alternatively, a corrosion consultant may provide recommendations for corrosion protection.

DEVELOPMENT CRITERIA

Landscape Maintenance and Planting

Water has been shown to weaken the inherent strength of all earth materials. Slope stability, including coastal bluff stability, is significantly reduced by overly wet conditions. Positive surface drainage away from the coastal bluff should be maintained and only the amount of irrigation necessary to sustain plant life should be provided. Over-watering should be avoided as it adversely affects site improvements, and causes perched groundwater conditions. The onsite earth materials are erodible. Eroded debris may be reduced by establishing and maintaining a suitable vegetation cover soon after construction. Plants selected for landscaping should be light weight, deep rooted types that require little water and are capable of surviving the prevailing climate. Consideration should be given to the type of vegetation chosen and their potential effect upon surface improvements (i.e., some trees will have an effect on concrete flatwork with their extensive root systems). From a geotechnical standpoint, leaching is not recommended for establishing landscaping. If the surface soils are processed for the purpose of adding amendments, they should be recompacted to 90 percent minimum relative compaction, provided they are outside the building footprint and not used as retaining wall backfill. Jute-type matting or other fibrous covers may aid in allowing the establishment of a sparse plant cover. Using plants other than those recommended above will increase the potential for perched water, staining, mold, etc., to develop. A rodent control program to prevent burrowing should be implemented. Irrigation of natural slope areas is generally not recommended. These recommendations regarding plant type, irrigation practices, and rodent control should be provided to the homeowner and all interested/affected parties.

Drainage

Adequate surface drainage is a very important factor in reducing the likelihood of adverse performance of improvements. Surface drainage should be sufficient to prevent ponding of water anywhere on the property, and especially near the proposed improvements. Lot surface drainage should be carefully taken into consideration during fine grading, landscaping, and building construction. Therefore, care should be taken that future landscaping or construction activities do not create adverse drainage conditions. Positive site drainage within the property should be provided and maintained at all times. Drainage should not be directed toward the building foundations and bluff, and not allowed to pond or seep into the ground. In general, the area within 5 feet around a structure should slope away from the structure. We recommend that unpaved lawn and landscape areas have a minimum gradient of 1 to 2 percent sloping away from structures or conform to building code requirements for surficial drainage, and whenever possible, should be above adjacent paved areas. Consideration should be given to avoiding construction of planters adjacent to structures (buildings, pools, spas, etc.). Pad drainage should be directed toward the street or other approved area(s). Although not a geotechnical requirement, roof gutters, down spouts, or other appropriate means may be used to control roof drainage. Down spouts, or drainage devices should outlet a minimum of 5 feet from structures or into a subsurface drainage system. Areas of seepage may develop due to irrigation or heavy rainfall, and should be anticipated. Minimizing irrigation will lessen this potential. If areas of seepage develop, recommendations for minimizing this effect could be provided upon request.

Erosion Control

Onsite earth materials have a moderate to high erosion potential. Consideration should be given to providing hay bales and silt fences for the temporary control of surface water, from a geotechnical viewpoint.

Landscape Planters

We recommend that any proposed open-bottom planters adjacent to proposed structures be eliminated for a minimum distance of 10 feet. As an alternative, closed-bottom type planters could be used. An outlet placed in the bottom of the planter, could be installed to direct drainage away from structures or any exterior concrete flatwork. If planters are constructed adjacent to structures, the sides and bottom of the planter should be provided with a moisture barrier to prevent penetration of irrigation water into the subgrade. Provisions should be made to drain the excess irrigation water from the planters without saturating the subgrade below or adjacent to the planters.

Subsurface and Surface Water

Subsurface and surface water are not anticipated to affect site development, provided that the recommendations contained in this report are incorporated into final design and construction, and that prudent surface and subsurface drainage practices are incorporated into the construction plans. Perched groundwater conditions along zones of contrasting permeabilities may not be precluded from occurring in the future due to site irrigation, poor drainage conditions, or damaged underground utilities, and should be anticipated. Should perched groundwater conditions develop, this office could assess the affected area(s) and provide the appropriate recommendations to mitigate the observed groundwater conditions. Groundwater conditions may change with the introduction of irrigation, rainfall, or other factors.

Site Improvements

If any additional improvements are planned for the site, recommendations concerning the geological or geotechnical aspects of design and construction of said improvements could be provided upon request. This office should be notified in advance of any fill placement, grading of the site, or trench backfilling after rough grading has been completed. This includes any grading, underground utility trench and retaining wall backfills, flatwork, etc.

Tile Flooring

Tile flooring can crack, reflecting cracks in the concrete slab-on-grade floor below the tile. Although, small cracks in a slab-on-grade floor may not be significant. Therefore, the designer should consider additional reinforcement for concrete slab-on-grade floors where tile will be placed. The tile installer should consider installation methods that reduce possible cracking of the tile such as slipsheets. Slipsheets or a vinyl crack isolation membrane (approved by the Tile Council of America/Ceramic Tile Institute) are recommended between tile and concrete slabs on grade.

Additional Grading

This office should be notified in advance of any fill placement, supplemental regrading of the site, or trench and retaining wall backfilling after rough grading has been completed. This includes completion of grading in the driveway and flatwork areas.

Footing Trench Excavations

All footing excavations should be observed by a representative of this firm after trenching and prior to concrete form and steel reinforcement placement. The purpose of the observations is to evaluate that the excavations have been made into the recommended bearing material and to the minimum widths and depths recommended for construction. If loose or compressible materials are exposed within the footing excavations, a deeper footing or the removal and recompaction of the subgrade materials would be recommended at that time. Footing trench spoil and any excess soils generated from underground utility trench excavations should be uniformly moisture conditioned to at least 1 to 2 percent above the soil's optimum moisture content and compacted to a minimum relative density of 90 percent, if not removed from the site.

Trenching/Temporary Construction Backcuts

Considering the nature of the onsite earth materials, caving or sloughing could be a factor in subsurface excavations and trenching. Shoring or excavating the trench walls/backcuts at the angle of repose (typically 25 to 45 degrees [except as specifically superceded within the text of this report]), should be anticipated. All excavations should be observed by licensed engineering geologist or engineer from GSI, prior to workers entering the excavation or trench, and minimally conform to CAL/OSHA, state, and local safety codes. Should adverse conditions exist, appropriate recommendations would be offered at that time. The above recommendations should be provided to any contractors and subcontractors, or homeowners, etc., that may perform such work.

Underground Utility Trench Backfill

1. All underground utility trench backfill should be brought to at least 1 to 2 percent above the soil's optimum moisture content and then be compacted to obtain a minimum relative density of 90 percent of the laboratory standard. As an alternative for shallow (12-inch to 18-inch) under-slab trenches, sand having a sand equivalent value of 30 or greater may be used and jetted or flooded into place, if permitted by the building official. Observation, tactile probing and field density testing should be provided to evaluate the desired results.

2. Exterior trenches adjacent to, and within areas extending below a 1:1 (h:v) plane projected down from the outside, bottom edge of the footings and all trenches beneath hardscape features, should be compacted to at least 90 percent of the laboratory standard. Sand backfill, unless excavated from the trench, should not be used below the aforementioned plane. Compaction testing, selective tactile probing, and observations should be performed to evaluate the desired results.

3. Underground utilities crossing grade beams, perimeter beams, or footings should either pass below the footing or grade beam using a hardened collar or foam spacer, or pass through the footing or grade beam in accordance with the recommendations of the structural engineer.

SUMMARY OF RECOMMENDATIONS REGARDING GEOTECHNICAL OBSERVATION AND TESTING

We recommend that observation and testing be performed by GSI at each of the following construction stages:

- During grading/recertification, including remedial earthwork.

- During excavation greater than 4 feet in depth.

- During placement of subdrains or other subdrainage devices, prior to placing fill and backfill.

- After the excavation of building footings, retaining wall footings, swimming pool/spa foundations, and free-standing walls footings, prior to the placement of reinforcing steel or concrete.

- After the excavation for pool/spa shells, prior to the placement of reinforcing steel or shotcrete.

- Prior to pouring any slabs or exterior flatwork, after presoaking/presaturation of building pads and other flatwork subgrade, before the placement of concrete, reinforcing steel, capillary break (e.g., sand, pea-gravel, etc.), or vapor retarders.

- During placement of backfill for area drain, interior plumbing, and underground utility line trenches, and retaining walls.

- During slope construction/repair, including temporary slopes.

- When any unusual soil conditions are encountered during any construction operations, subsequent to the issuance of this report.

- When any future homeowner improvements are constructed, prior to construction.

- A report of geotechnical observation and testing should be provided at the conclusion of each of the above stages, in order to provide concise and clear documentation of site work, and to comply with code requirements.

OTHER DESIGN PROFESSIONALS/CONSULTANTS

The design civil engineer, structural engineer, post-tension designer, architect, landscape architect, wall designer, etc., should review the recommendations provided herein, incorporate those recommendations into all their respective plans, and by explicit reference, make this report part of their project plans. This report presents minimum design criteria for the design of slabs, foundations and other elements possibly applicable to the project. These criteria should not be considered as substitutes for actual designs by the structural engineer/designer. The structural engineer/designer should analyze actual soil-structure interaction and consider, as needed, bearing, expansive soil influence, and strength, stiffness and deflections in the various slab, foundation, and other elements in order to develop appropriate, design-specific details. As conditions dictate, it is possible that other influences will also have to be considered. The structural engineer/designer should consider all applicable codes and authoritative sources where needed. If analyses by the structural engineer/designer result in less critical details than are provided herein as minimums, the minimums presented herein should be adopted. It is considered likely that some, more restrictive details will be required. If the structural engineer/designer has any questions or requires further assistance, they should not hesitate to call or otherwise transmit their requests to GSI. In order to mitigate potential distress, the foundation and improvements' designers should confirm to GSI and the governing agency, in writing, that the proposed foundations and improvements can tolerate the amount of differential settlement and expansion characteristics and design criteria specified herein.

PLAN REVIEW

Final project plans (grading, precise grading, foundation, retaining wall, landscaping, etc.), should be reviewed by this office prior to construction, so that construction is in accordance with the conclusions and recommendations of this report. Based on our review, supplemental recommendations or further geotechnical studies may be warranted.

LIMITATIONS

The materials encountered on the project site and used for our analysis are believed representative of the area; however, soil and bedrock materials vary in character between excavations and natural outcrops or conditions exposed during mass grading. Site conditions may vary due to seasonal changes or other factors.

Inasmuch as our study is based upon our review and engineering analyses and laboratory data, the conclusions and recommendations are professional opinions. These opinions have been derived in accordance with current standards of practice, and no warranty, either express or implied, is given. Standards of practice are subject to change with time. GSI assumes no responsibility or liability for work or testing performed by others, or their inaction; or work performed when GSI is not requested to be onsite, to evaluate if our recommendations have been properly implemented. Use of this report constitutes an agreement and consent by the user to all the limitations outlined above, notwithstanding any other agreements that may be in place. In addition, this report may be subject to review by the controlling authorities. Thus, this report brings to completion our scope of services for this portion of the project.

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B

BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM					
Major Divisions			Group Symbols	Typical Names	
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines	
		Gravel with	GM	Silty gravels gravel-sand-silt mixtures	
			GC	Clayey gravels, gravel-sand-clay mixtures	
	Sands more than 50% of coarse fraction passes No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines	
			SP	Poorly graded sands and gravelly sands, little or no fines	
		Sands with Fines	SM	Silty sands, sand-silt mixtures	
			SC	Clayey sands, sand-clay mixtures	
Fine-Grained Soils 50% or more passes No. 200 sieve	Silts and Clays Liquid limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands	
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	
			OL	Organic silts and organic silty clays of low plasticity	
	Silts and Clays Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts	
			CH	Inorganic clays of high plasticity, fat clays	
			OH	Organic clays of medium to high plasticity	
Highly Organic Soils			PT	Peat, mucic, and other highly organic soils	

CONSISTENCY OR RELATIVE DENSITY

CRITERIA

Standard Penetration Test

Penetration Resistance N (blows/ft)	Relative Density
0 - 4	Very loose
4 - 10	Loose
10 - 30	Medium
30 - 50	Dense
> 50	Very dense

Standard Penetration Test

Penetration Resistance N (blows/ft)	Consistency	Unconfined Compressive Strength (tons/ft^2)
<2	Very Soft	<0.25
2 - 4	Soft	0.25 - .050
4 - 8	Medium	0.50 - 1.00
8 - 15	Stiff	1.00 - 2.00
15 - 30	Very Stiff	2.00 - 4.00
>30	Hard	>4.00

	3"		3/4"	#4	#10	#40	#200 U.S. Standard Sieve
Unified Soil Classification	Cobbles	Gravel		Sand			Silt or Clay
		coarse	fine	coarse	medium	fine	

MOISTURE CONDITIONS

Dry	Absence of moisture: dusty, dry to the touch
Slightly Moist	Below optimum moisture content for compaction
Moist	Near optimum moisture content
Very Moist	Above optimum moisture content
Wet	Visible free water; below water table

MATERIAL QUANTITY

trace	0 - 5 %
few	5 - 10 %
little	10 - 25 %
some	25 - 45 %

OTHER SYMBOLS

C	Core Sample
S	SPT Sample
B	Bulk Sample
═	Groundwater
Qp	Pocket Penetrometer

BASIC LOG FORMAT:
Group name, Group symbol, (grain size), color, moisture, consistency or relative density. Additional comments: odor, presence of roots, mica, gypsum, coarse grained particles, etc.

EXAMPLE:
Sand (SP), fine to medium grained, brown, moist, loose, trace silt, little fine gravel, few cobbles up to 4" in size, some hair roots and rootlets.

GeoSoils, Inc.

BORING LOG

PROJECT: APNS 350-131-02-00 AND -29-00
LA JOLLA, CALIFORNIA 92037

W.O. 8358-A-SC *BORING* B-1 *SHEET* 1 *OF* 1

DATE EXCAVATED 6-3-22 *LOGGED BY:* TMP *APPROX. ELEV.:* 91'

SAMPLE METHOD: Mod. Cal Sampler and Standard Penetrometer

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0				SM				**QUATERNARY RESIDUAL SOIL:** @ 0', SILTY SAND, light olive brown, damp, dense.
				SC/CL				**QUATERNARY OLD PARALIC DEPOSITS:**
		Ring	71		114.1	11.4	61.9	@ 2', CLAYEY SAND, light olive brown, damp, dense; thin interbeds of SANDY CLAY, light olive brown, damp, hard; trace GRAVEL, trace precipitates (blebs). @ 3', As per 2', moist.
5		SPT	50			11.3		@ 6', As per 3'.
10		SPT	38/ 50-5"	SP-CL		8.4		@ 10', Interbedded SAND, olive brown, damp, very dense and CLAY, light olive brown, damp, hard; trace precipitates (blebs).
15		SPT	27/ 50-4"			9.7		@ 15', As per 10'.
								Total Depth = 16' (Practical Refusal Due to Cobbles) No Groundwater or Caving Encountered. Backfilled 6-3-22.
20								
25								
30								

Standard Penetration Test
Undisturbed, Ring Sample
Groundwater
Seepage

GeoSoils, Inc.

BORING LOG

PROJECT: APNS 350-131-02-00 AND -29-00
LA JOLLA, CALIFORNIA 92037

W.O. 8358-A-SC *BORING* B-2 *SHEET* 1 *OF* 1

DATE EXCAVATED 6-3-22 *LOGGED BY:* TMP *APPROX. ELEV.:* 83'

SAMPLE METHOD: Mod. Cal Sampler and Standard Penetrometer

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0				SM				**QUATERNARY RESIDUAL SOIL:** @ 0', SILTY SAND, light olive brown, damp, dense; trace GRAVEL up to approximately 1 1/2" in dimension.
				SP-CL				**QUATERNARY OLD PARALIC DEPOSITS:** @ 1', SAND, olive gray and reddish yellow, damp, dense; thin interbeds of CLAY, light olive brown, damp, hard; trace rounded GRAVEL up to approximately 1" in dimension.
			61					@ 3', As per 1'.
5			35					@ 6', As per 1'.
10			53	SC/CL				@ 10', CLAYEY SAND, dark olive brown and olive gray, damp, very dense; interbeds of CLAY, olive gray, damp, hard; trace manganese-oxide staining.
15			30	SP				@ 15', SAND, olive gray and olive brown, wet to saturated, medium dense; fine to medium grained.
			65	CL				@ 18', CLAY, medium gray, moist to saturated, hard. @ 19', Groundwater seepage encountered.
20								Total Depth = 19 1/2' Seepage Encountered at Approximately 19'. No Caving Encountered. Backfilled 6-3-22.
25								
30								

Standard Penetration Test
Undisturbed, Ring Sample
Groundwater
Seepage

APPENDIX C

SEISMICITY DATA

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*                    *
*   E Q F A U L T    *
*                    *
*   Version 3.00     *
*                    *
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 8358-A-SC

DATE: 06-25-2022

JOB NAME: HERITAGE BRIDGE, LLC, FALCON COVE, LLC

CALCULATION NAME: 8358

FAULT-DATA-FILE NAME: C:\Users\Ryan\Documents\EQFAULT1\CGSFLTE.DAT

SITE COORDINATES:
SITE LATITUDE: 32.8483
SITE LONGITUDE: 117.2668

SEARCH RADIUS: 62.2 mi

ATTENUATION RELATION: 12) Bozorgnia Campbell Niazi (1999) Hor.-Soft Rock-Cor.
UNCERTAINTY (M=Median, S=Sigma): S Number of Sigmas: 1.0
DISTANCE MEASURE: cdist
SCOND: 0
Basement Depth: 5.00 km Campbell SSR: 1 Campbell SHR: 0
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\Users\Ryan\Documents\EQFAULT1\CGSFLTE.DAT

MINIMUM DEPTH VALUE (km): 3.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
ROSE CANYON	0.4(0.6)	7.2	0.830	XI
CORONADO BANK	12.1(19.4)	7.6	0.434	X
NEWPORT-INGLEWOOD (Offshore)	23.3(37.5)	7.1	0.175	VIII
ELSINORE (JULIAN)	38.1(61.3)	7.1	0.106	VII
ELSINORE (TEMECULA)	39.5(63.5)	6.8	0.083	VII
EARTHQUAKE VALLEY	46.0(74.0)	6.5	0.058	VI
PALOS VERDES	49.3(79.4)	7.3	0.093	VII
ELSINORE (COYOTE MOUNTAIN)	53.2(85.6)	6.8	0.060	VI
SAN JOAQUIN HILLS	54.5(87.7)	6.6	0.073	VII
ELSINORE (GLEN IVY)	55.1(88.7)	6.8	0.058	VI
SAN JACINTO-ANZA	60.5(97.4)	7.2	0.070	VI
SAN JACINTO-COYOTE CREEK	61.0(98.1)	6.6	0.046	VI

****

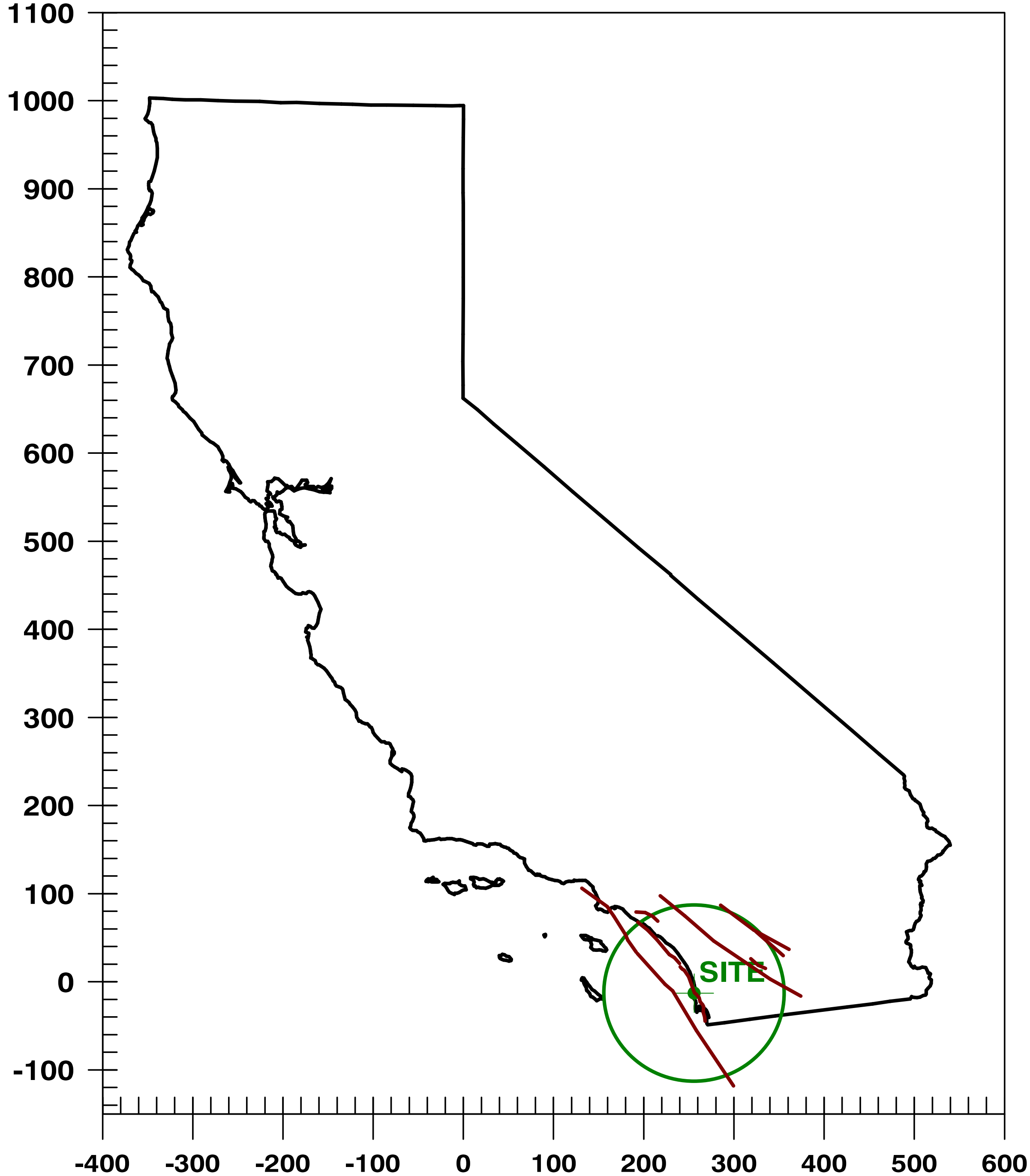
-END OF SEARCH- 12 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ROSE CANYON FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 0.4 MILES (0.6 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.8297 g

CALIFORNIA FAULT MAP

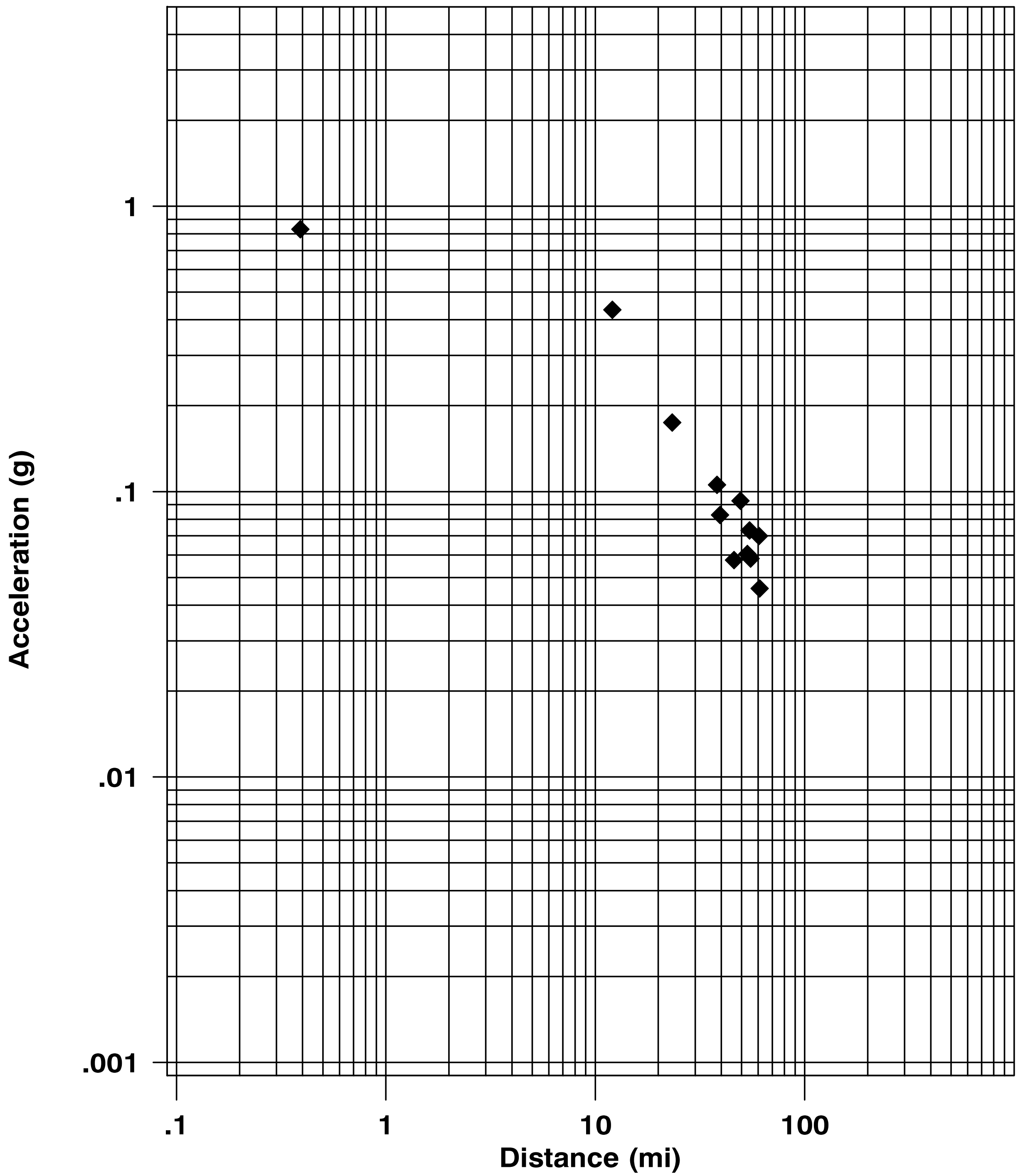
HERITAGE BRIDGE, LLC, FALCON COVE, LLC



W.O. 8358-A-SC
PLATE C-3

MAXIMUM EARTHQUAKES

HERITAGE BRIDGE, LLC, FALCON COVE, LLC



W.O. 8358-A-SC
PLATE C-4

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*                       *
*    E Q S E A R C H    *
*                       *
*     Version 3.00      *
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ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 8358-A-SC

DATE: 06-25-2022

JOB NAME: HERITAGE BRIDGE, LLC, FALCON COVE, LLC

EARTHQUAKE-CATALOG-FILE NAME: C:\Users\Ryan\Documents\EQSEARCH\ALLQUAKE-2021.DAT

MAGNITUDE RANGE:
MINIMUM MAGNITUDE: 5.00
MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:
SITE LATITUDE: 32.8483
SITE LONGITUDE: 117.2668

SEARCH DATES:
START DATE: 1800
END DATE: 2021

SEARCH RADIUS:
62.2 mi
100.1 km

ATTENUATION RELATION: 12) Bozorgnia Campbell Niazi (1999) Hor.-Soft Rock-Cor.
UNCERTAINTY (M=Median, S=Sigma): S Number of Sigmas: 1.0
ASSUMED SOURCE TYPE: SS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
SCOND: 0 Depth Source: A
Basement Depth: 5.00 km Campbell SSR: 1 Campbell SHR: 0
COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 3.0

EARTHQUAKE SEARCH RESULTS

Page 1

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.102	VII	10.2(16.5)
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.253	IX	10.6(17.1)
DMG	32.7000	117.2000	05/27/1862	20 0 0.0	0.0	5.90	0.170	VIII	10.9(17.6)
T-A	32.6700	117.1700	10/21/1862	0 0 0.0	0.0	5.00	0.079	VII	13.5(21.8)
T-A	32.6700	117.1700	12/00/1856	0 0 0.0	0.0	5.00	0.079	VII	13.5(21.8)
T-A	32.6700	117.1700	05/24/1865	0 0 0.0	0.0	5.00	0.079	VII	13.5(21.8)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.058	VI	18.7(30.0)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.060	VI	27.3(43.9)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.035	V	36.0(57.9)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.026	V	40.8(65.7)
T-A	32.2500	117.5000	01/13/1877	20 0 0.0	0.0	5.00	0.024	V	43.5(70.0)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.027	V	45.6(73.4)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.022	IV	49.4(79.6)
GSP	32.3290	117.9170	06/15/2004	222848.2	10.0	5.30	0.024	IV	52.1(83.9)
DMG	32.7000	116.3000	02/24/1892	720 0.0	0.0	6.70	0.052	VI	57.0(91.8)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.018	IV	59.3(95.4)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.018	IV	59.3(95.4)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.032	V	59.3(95.4)
GSG	33.4200	116.4890	07/07/2010	235333.5	14.0	5.50	0.023	IV	59.8(96.3)
DMG	32.0000	117.5000	06/24/1939	1627 0.0	0.0	5.00	0.017	IV	60.1(96.8)
DMG	32.0000	117.5000	05/01/1939	2353 0.0	0.0	5.00	0.017	IV	60.1(96.8)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.023	IV	60.4(97.2)
DMG	32.2000	116.5500	11/05/1949	43524.0	0.0	5.10	0.018	IV	61.2(98.5)
DMG	32.2000	116.5500	11/04/1949	204238.0	0.0	5.70	0.026	V	61.2(98.5)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.019	IV	61.8(99.5)

****

-END OF SEARCH- 25 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2021

LENGTH OF SEARCH TIME: 222 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 10.2 MILES (16.5 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 6.7

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.253 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:

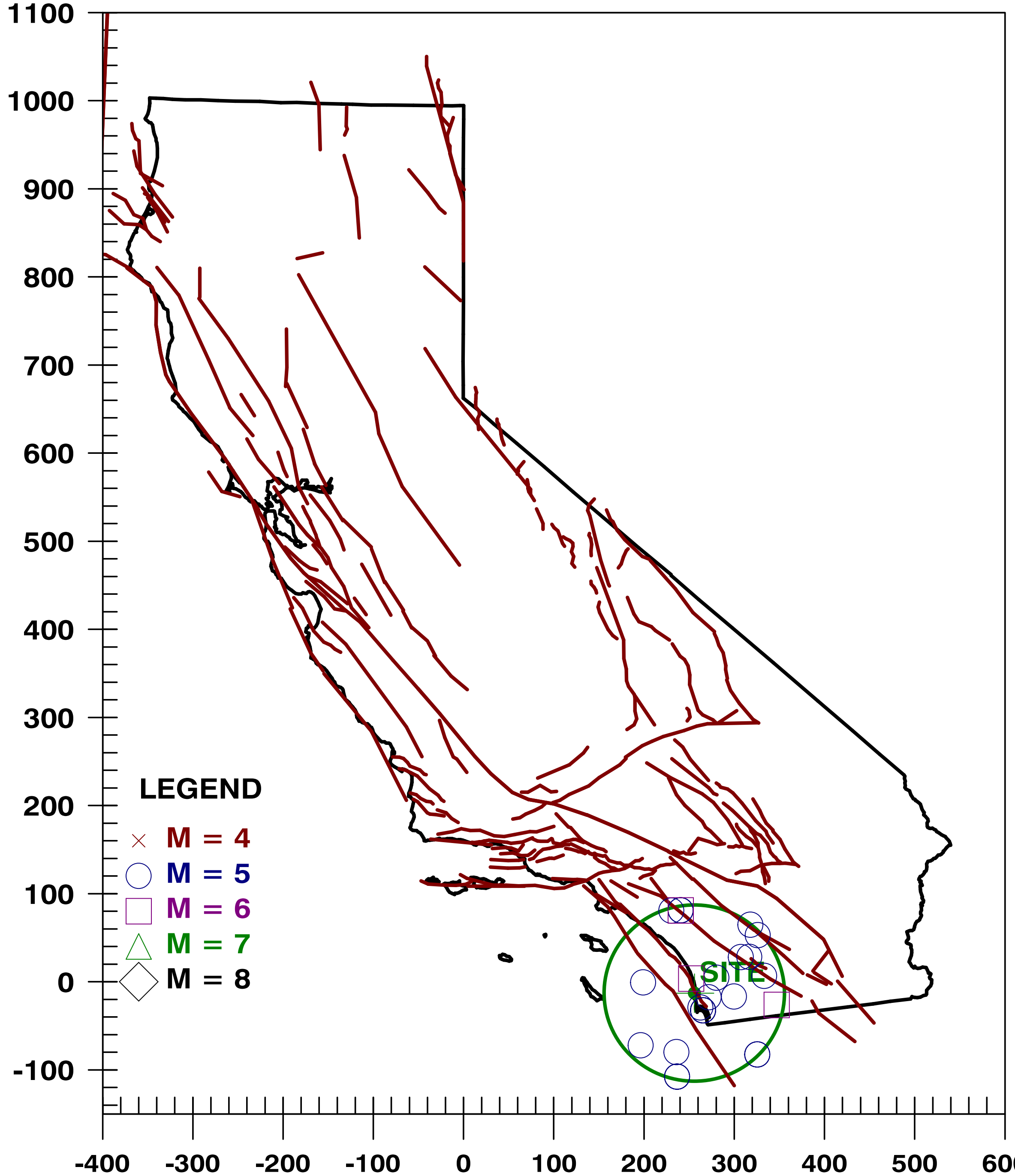
a-value= 1.105
b-value= 0.467
beta-value= 1.076

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	25	0.11261
4.5	25	0.11261
5.0	25	0.11261
5.5	8	0.03604
6.0	3	0.01351
6.5	2	0.00901

EARTHQUAKE EPICENTER MAP

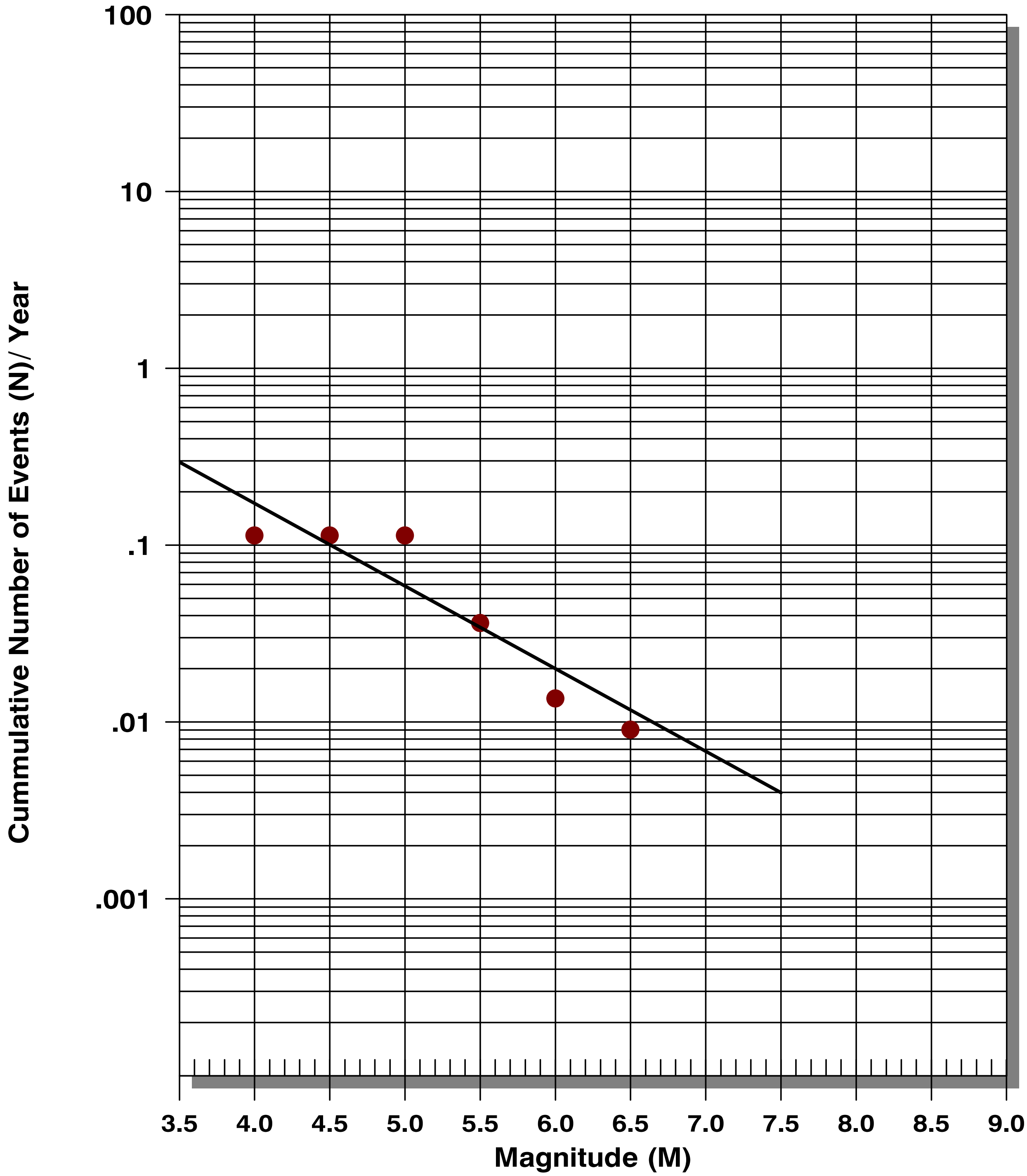
HERITAGE BRIDGE, LLC, FALCON COVE, LLC



W.O. 8358-A-SC
PLATE C-8

EARTHQUAKE RECURRENCE CURVE

HERITAGE BRIDGE, LLC, FALCON COVE, LLC

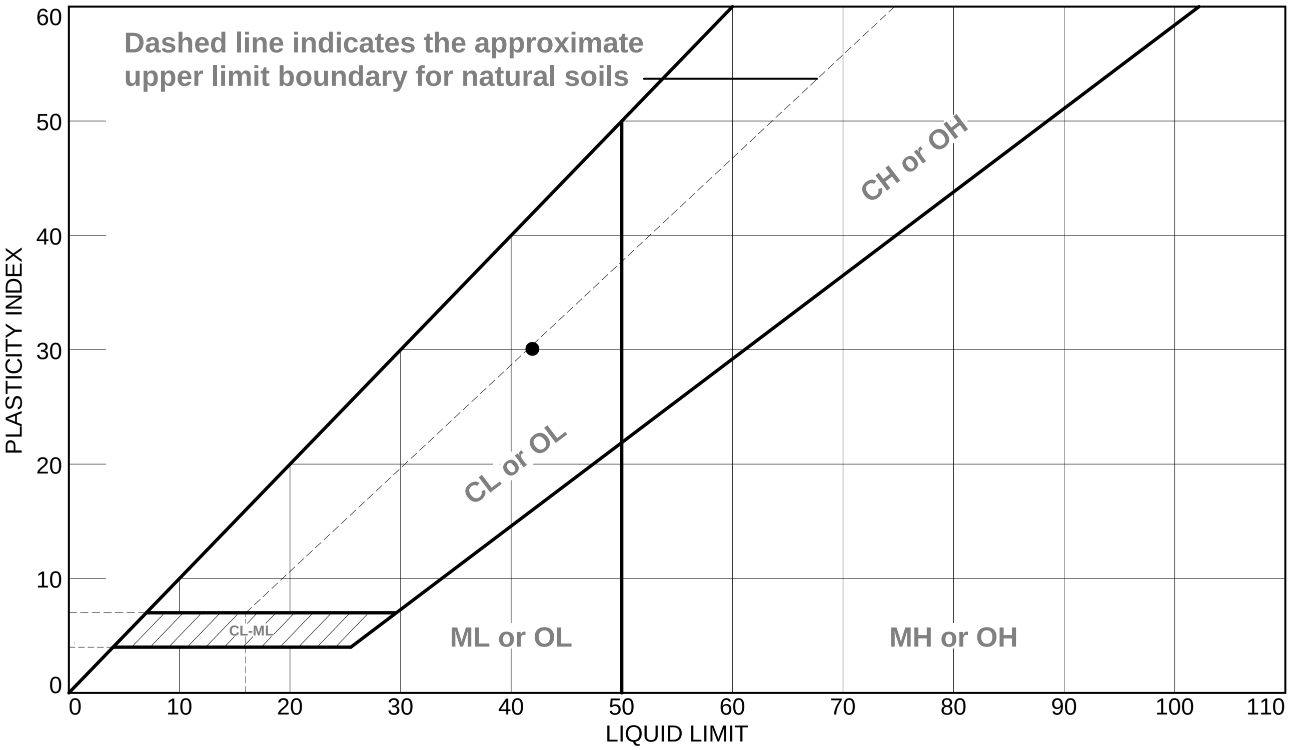


W.O. 8358-A-SC
PLATE C-9

APPENDIX D

LABORATORY DATA

LIQUID AND PLASTIC LIMITS TEST REPORT



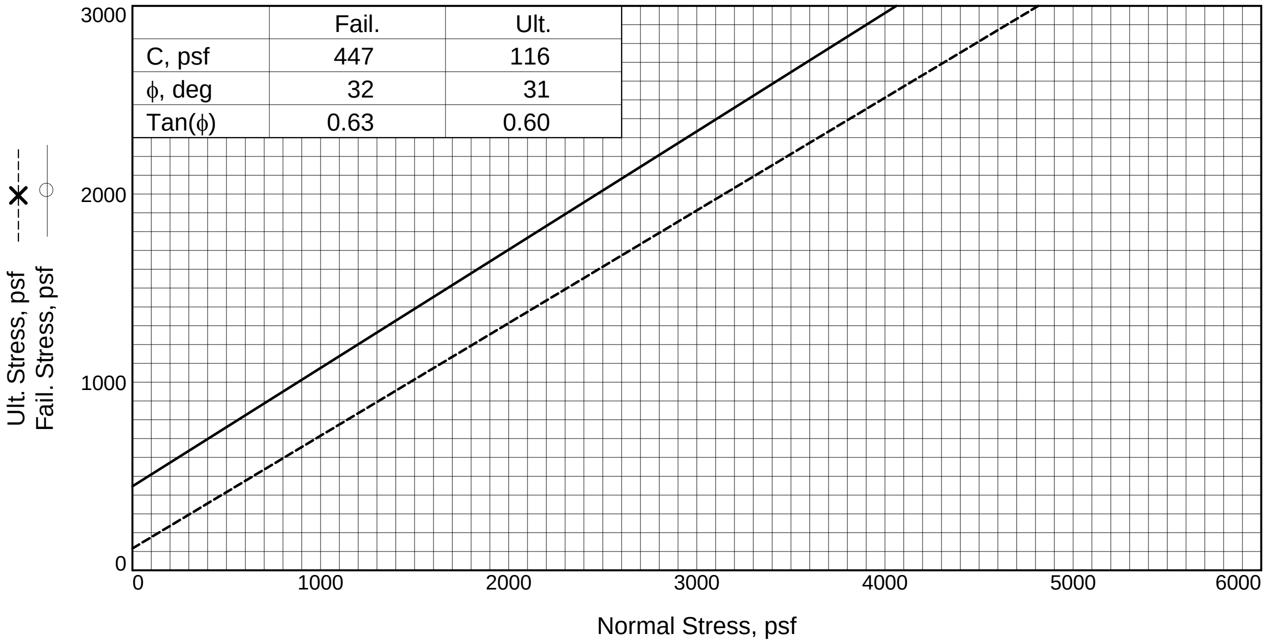
	SOIL DATA							
	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	B-1	B-1	10.0	-	12	42	30	CL

GeoSoils, Inc.

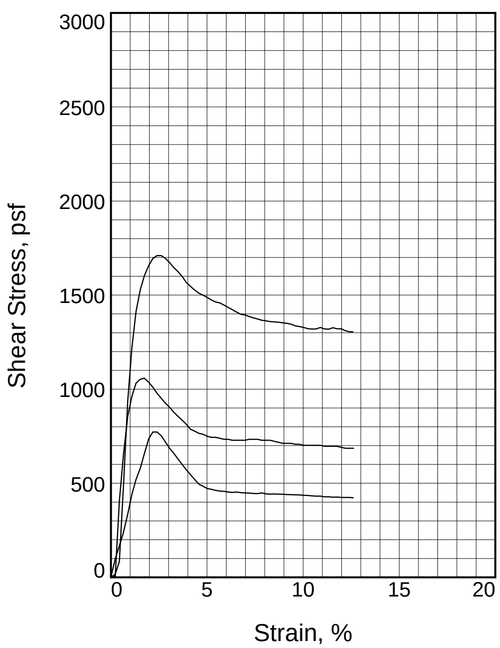
Client: Heritage Bridge, LLC, Falcon Cove, LLC
Project: APN: 350-131-02-00 & -29-00

Project No.: 8358-A-SC **Plate**

Tested By: TR **Checked By:** TR



	Fail.	Ult.
C, psf	447	116
ϕ, deg	32	31
Tan(ϕ)	0.63	0.60



	Sample No.	1	2	3
Initial	Water Content, %	11.4	11.4	11.4
	Dry Density, pcf	119.3	114.6	117.7
	Saturation, %	71.3	63.0	68.4
	Void Ratio	0.4395	0.4977	0.4584
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	15.0	15.8	15.7
	Dry Density, pcf	119.5	115.1	118.7
	Saturation, %	94.5	88.4	96.7
	Void Ratio	0.4366	0.4917	0.4467
	Diameter, in.	2.38	2.38	2.38
	Height, in.	1.00	1.00	0.99
	Normal Stress, psf	500	1000	2000
	Fail. Stress, psf	773	1058	1710
	Strain, %	2.2	1.7	2.4
	Ult. Stress, psf	428	697	1320
	Strain, %	11.3	11.1	11.1
	Strain rate, in./min.	0.001	0.001	0.001

Sample Type: Natural
Description: Olive Brown Sandy Clay

Specific Gravity= 2.75
Remarks:

Plate ________

Client: Heritage Bridge, LLC, Falcon Cove, LLC

Project: APN: 350-131-02-00 & -29-00

Source of Sample: B-1 **Depth:** 3.0
Sample Number: B-1
Proj. No.: 8358-A-SC **Date Sampled:**

GeoSoils, Inc.

Tested By: TR **Checked By:** TR

GeoSoils, Inc.
5741 Palmer Way, Carlsbad CA 92010
Phone (760) 438-3155

CORRESION REPORT SUMMARY

Project No: **8358-A-SC**
Project Name: **Heritage Bridge, LLC, Falcon Cove, LLC**
Report Date: **June 23, 2022**

SAMPLE ID	pH (H+)	Minimum Resistivity (ohm/cm)	Sulfate Content (wt%)	Chloride Content (mg/kg)
B-2, 0-5ft	7.0	1200	0.045	140

Samples testing in accordance with: pH - CTM 643, Resistivity - CTM 643
Sulfate - CTM 417, Chloride - CTM 422

Remarks: ______________________________
____

APPENDIX E

SLOPE STABILITY ANALYSES

APPENDIX E

SLOPE STABILITY ANALYSES

INTRODUCTION OF GEOSTASE v.4.30.31 COMPUTER PROGRAM

Introduction

GEOSTASE v.4.30.31 is a fully integrated two-dimensional limit equilibrium slope stability analysis program developed by Dr. Garry H. Gregory, Ph.D., P.E., D.GE, Principal Consultant with Gregory Geotechnical. The name GEOSTASE is an acronym for **G**eneral **E**quilibrium **O**ptions for **ST**ability **A**nalysis of **S**lopes and **E**mbankments. It permits the user to develop the slope geometry interactively and perform slope analysis from within a single program.

GEOSTASE v.4.30.31 is capable of performing popular limit equilibrium analysis methods, such as the Simplified Bishop Method, Simplified Janbu Method, Spencer Method, Morgenstern-Price Method, Simplified Janbu Corrected Method, United States Army Corps of Engineers (USACE) Modified Swedish Method, and the Lowe and Karafiath Method. Standard search options include circular, random, wedge, block, and composite surface options. The software also includes a non-circular refined search option, referred to as ZRSAUTO. "ZRS" is an acronym for **Z**one, **R**eduction, and **S**hifting. The program can be used to search for the most critical surface and the FOS may be determined for specific surfaces. GEOSTASE v.4.30.31 is programmed to handle:

1. Heterogenous soil systems
2. Mohr-Columb and anisotropic soil strength properties
3. Reinforcing and restraining elements (i.e., piers, tiebacks [anchors], soil nails, and applied forces)
4. Nonlinear Mohr-Coulomb strength envelope
5. Pore water pressures for effective stress analysis using:
   a. Phreatic and piezometric surfaces
   b. Pore-pressure ratios
   c. Artesian pressure
   d. Constant pore water pressure
6. Pseudo-static (seismic) earthquake loading
7. Distributed and line loads
8. Automatic generation and analysis of an unlimited number of circular, noncircular and block-shaped failure modes
9. Analysis of right- and left-facing slopes
10. Both SI and Imperial units

General Information

If the reviewer wishes to obtain more information concerning slope stability analysis, the following literature may be consulted initially:

1. The Stability of Slopes, by E.N. Bromhead, Surrey University Press, Chapman and Hall, N.Y., 411 pages, ISBN 412 01061 5, 1992.

2. Soil Strength and Slope Stability, by J.M. Duncan, S.G. Wright, and T.L. Brandon, John Wiley and Sons, Inc., Second Edition, 317 pages, ISBN 978-1-118-65165-0, 2014.

3. Rock Slope Engineering, by E. Hoek and J.W. Bray, Inst. of Mining and Metallurgy, London, England, Third Edition, 358 pages, ISNB 0 900488 573, 1981.

4. Landslides: Analysis and Control, by R.L. Schuster and R.J. Krizek (editors), Special Report 176, Transportation Research Board, National Academy of Sciences, 234 pages, ISBN 0 309 02804 3, 1978.

5. Landslides: Investigation and Mitigation, by A.K. Turner and R.J. Krizek (editors), Special Report 247, Transportation Research Board, National Research Board, 675 pages, ISBN 0 309 06208-X, 1996.

GEOSTASE v.4.30.31 Features

GEOSTASE v.4.30.31 contains the following features:

1. Allows user to calculate FOS for static stability and seismic stability evaluations.

2. Allows user to analyze stability situations with different failure modes.

3. Allows user to edit input for slope geometry and calculate corresponding FOS.

4. Allows user to readily review on-screen the input slope geometry.

5. Allows user to automatically generate and analyze defined numbers of circular, non-circular and block-shaped failure surfaces (i.e., bedding plane, slide plane, etc.).

Input Data

Input data includes the following items:

1. Unit weight, cohesion, and friction angle of earth materials and bedding/discontinuity planes.

2. Slope geometry and distributed (building) loads.

3. The apparent dip of bedding and discontinuities can be modeled in an anisotropic angular range (i.e., from 0 to 90 degrees in into-slope and out-of-slope directions). For the analyses, anisotropic strength properties were assigned to the old paralic

deposits (Qop) between an angular range of 5 degrees from the horizontal plane, oriented in both into-slope and out-of-slope directions. We also applied anisotropic strength properties for the Point Loma Formation (Kp) within an angular range of 1 to 19 degrees from the horizontal plane oriented in an into-slope direction.

4. For the pseudo-static (seismic) analyses, earthquake loading was modeled using a seismic coefficient of 0.15*i* and a peak horizontal ground acceleration adjusted for site effects (PGA_M) of 0.758 g.

5. Soil parameters used in the slope stability analyses are provided Table E-1:

TABLE E-1 - SOIL STRENGTH PARAMETERS

SOIL MATERIALS	SOIL UNIT WEIGHT (pcf)		STATIC SHEAR STRENGTH PARAMETERS			
	Moist	Saturated	C (psf)		Φ (degrees)	
			Bedding			
			Cross	Parallel	Cross	Parallel
Artificial Fill - Compacted (Afc)	120	N/A	350		27	
Artificial Fill - Undocumented (Afc)	115	N/A	200		27	
Quaternary Residual Soil (Qr)	105	N/A	100		27	
Quaternary Old Paralic Deposits (Qop)	130	135	400	300	31	29
Point Loma Formation (Kp)	125	130	1,500	1,000	39	35
N/A - Not applied						

Output Information

Output information includes:

1. All input data.

2. FOS for the 10 most critical surfaces.

3. High quality plots can be generated. The plots include the slope geometry, the critical surfaces and the FOS.

4. The analyses were configured to search for 4,999 trial surfaces.

<u>Results of Slope Stability Calculations</u>

Table E-2 provides a summary of the results of our stability analyses along Geologic Cross Section A-A'. Computer printouts from the GEOSTASE program are also included as Plates E-1 through E-3.

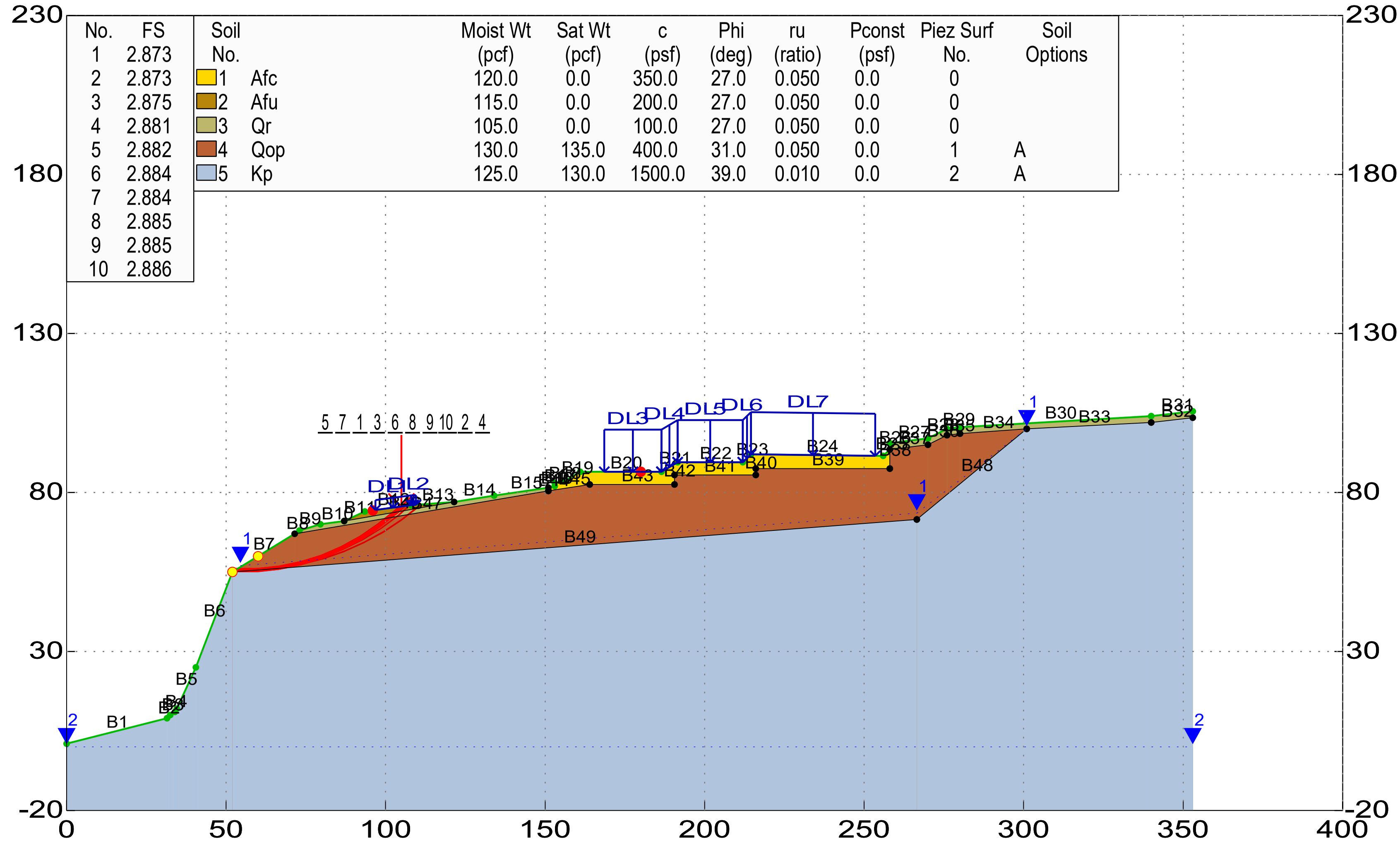
TABLE E-2 - SUMMARY OF SLOPE STABILITY ANALYSES

GEOLOGIC CROSS SECTION AND FAILURE TYPE	FACTOR-OF-SAFETY (FOS)		ANALYSIS METHOD	COMMENTS
	STATIC	SEISMIC		
A-A' Upper-Bluff Failure	2.8 (See Plate E-1)	2.0 (See Plate E-2)	Spencer	Static FOS = 2.8 at approximately 29 feet from the coastal bluff edge. Seismic FOS = 2.0 at approximately 35 feet from the coastal bluff edge.
A-A' Gross Bluff Failure	2.3 (See Plate E-3)	N/A	Spencer	Static FOS = 2.3 at about 43 feet from the coastal bluff edge.
N/A - Not analyzed				

HERITAGE BRIDGE, LLC, FALCON COVE, LLC / 8358-A-SC
UPPER-BLUFF FAILURE - STATIC

GEOSOILS, INC.

\A-A' Upper Bluff - Static.gsd



GEOSTASE FS = 2.873

Spencer Method

GEOSTASE Slope Stability Analysis

GREGORY GEOTECHNICAL

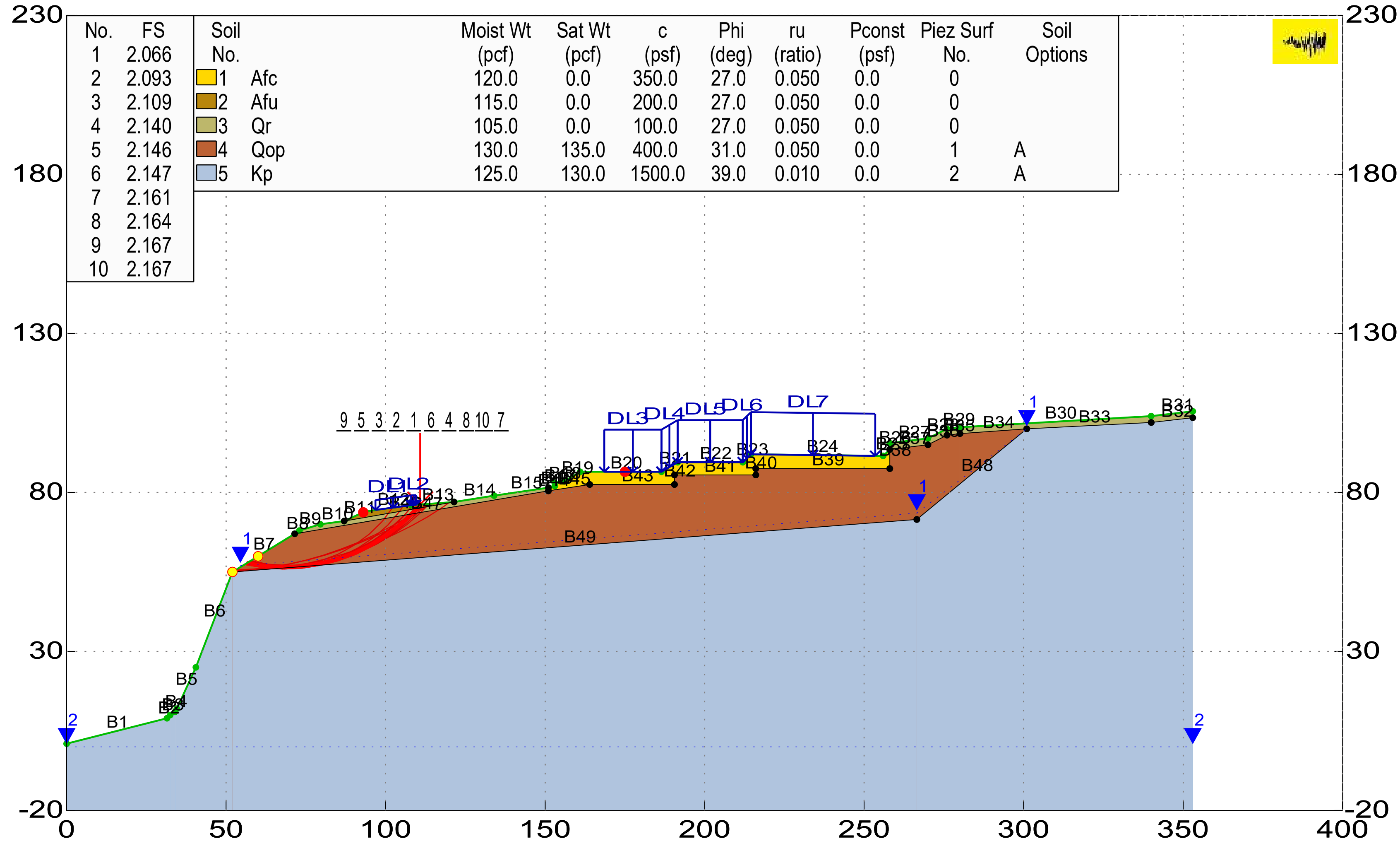
GEOSTASE® by GREGORY GEOTECHNICAL SOFTWARE

PLATE E-1

HERITAGE BRIDGE, LLC, FALCON COVE, LLC / 8358-A-SC
UPPER-BLUFF FAILURE - SEISMIC

GEOSOILS, INC.

\A-A' Upper Bluff - Seismic.gsd



GEOSTASE FS = 2.066

Spencer Method

kh = 0.15000

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GEOSTASE Slope Stability Analysis

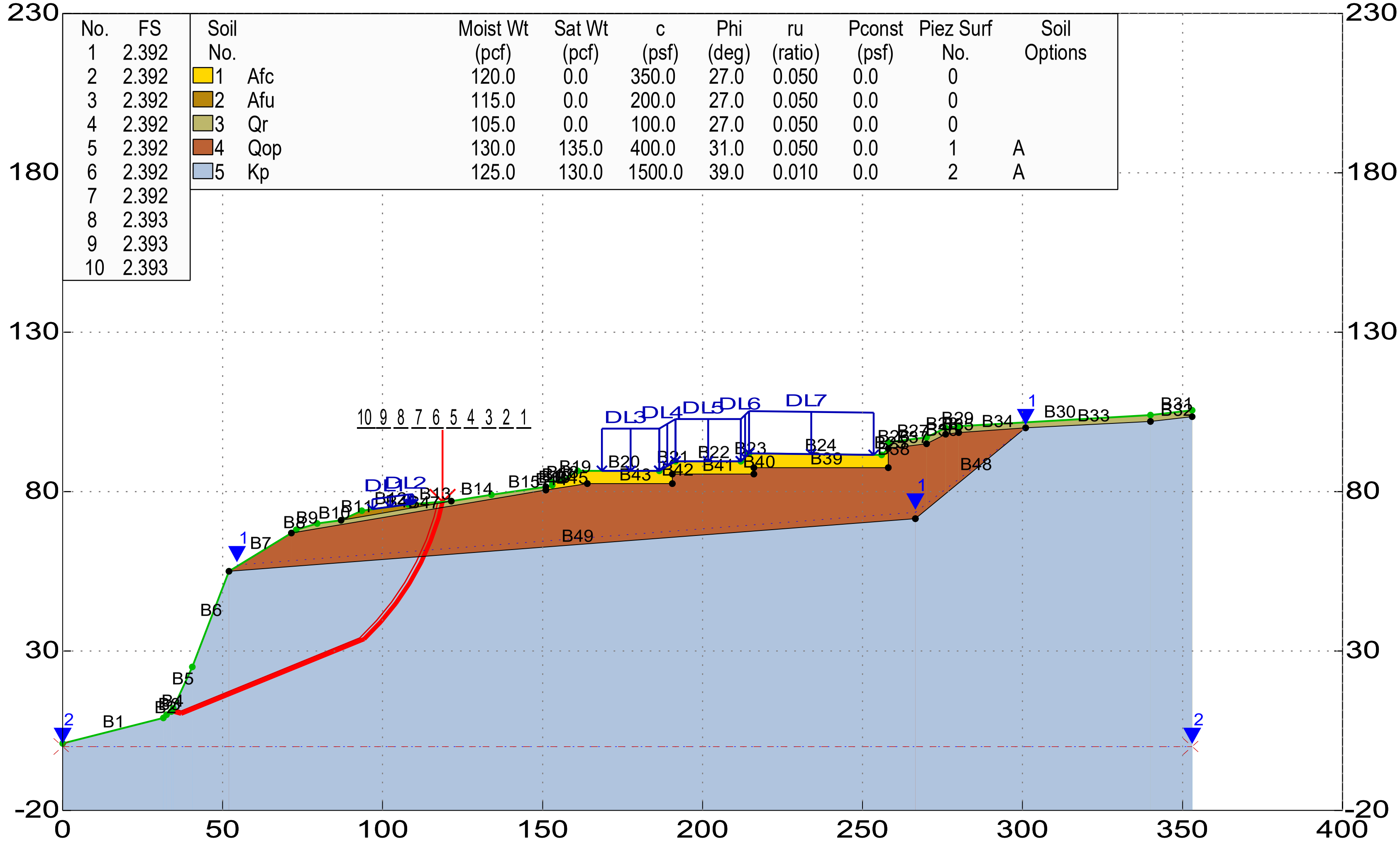
GREGORY GEOTECHNICAL

PLATE E-2

HERITAGE BRIDGE, LLC, FALCON COVE, LLC / 8358-A-SC
GROSS BLUFF FAILURE - STATIC

GEOSOILS, INC.

\A-A' Gross 6 - Static.gsd



No.	FS
1	2.392
2	2.392
3	2.392
4	2.392
5	2.392
6	2.392
7	2.392
8	2.393
9	2.393
10	2.393

Soil No.		Moist Wt (pcf)	Sat Wt (pcf)	c (psf)	Phi (deg)	ru (ratio)	Pconst (psf)	Piez Surf No.	Soil Options
1	Afc	120.0	0.0	350.0	27.0	0.050	0.0	0	
2	Afu	115.0	0.0	200.0	27.0	0.050	0.0	0	
3	Qr	105.0	0.0	100.0	27.0	0.050	0.0	0	
4	Qop	130.0	135.0	400.0	31.0	0.050	0.0	1	A
5	Kp	125.0	130.0	1500.0	39.0	0.010	0.0	2	A

GEOSTASE FS = 2.392

Spencer Method

GEOSTASE Slope Stability Analysis

GREGORY GEOTECHNICAL

GEOSTASE® by GREGORY GEOTECHNICAL SOFTWARE

PLATE E-3

APPENDIX F

GENERAL EARTHWORK AND GRADING GUIDELINES

GENERAL EARTHWORK AND GRADING GUIDELINES

General

These guidelines present general procedures and requirements for earthwork and grading as shown on the approved grading plans, including preparation of areas to be filled, placement of fill, installation of subdrains, excavations, and appurtenant structures or flatwork. The recommendations contained in the geotechnical report are part of these earthwork and grading guidelines and would supercede the provisions contained hereafter in the case of conflict. Evaluations performed by the consultant during the course of grading may result in new or revised recommendations which could supercede these guidelines or the recommendations contained in the geotechnical report. Generalized details follow this text.

The contractor is responsible for the satisfactory completion of all earthwork in accordance with provisions of the project plans and specifications and latest adopted Code. In the case of conflict, the most onerous provisions shall prevail. The project geotechnical engineer and engineering geologist (geotechnical consultant), or their representatives, should provide observation and testing services, and geotechnical consultation during the duration of the project.

EARTHWORK OBSERVATIONS AND TESTING

Geotechnical Consultant

Prior to the commencement of grading, a qualified geotechnical consultant (soil engineer and engineering geologist) should be employed for the purpose of observing earthwork procedures and testing the fills for general conformance with the recommendations of the geotechnical report(s), the approved grading plans, and applicable grading codes and ordinances.

The geotechnical consultant should provide testing and observation so that an evaluation may be made that the work is being accomplished as specified. It is the responsibility of the contractor to assist the consultants and keep them apprised of anticipated work schedules and changes, so that they may schedule their personnel accordingly.

All remedial removals, clean-outs, prepared ground to receive fill, key excavations, and subdrain installation should be observed and documented by the geotechnical consultant prior to placing any fill. It is the contractor's responsibility to notify the geotechnical consultant when such areas are ready for observation.

Laboratory and Field Tests

Maximum dry density tests to determine the degree of compaction should be performed in accordance with American Standard Testing Materials test method ASTM designation D 1557. Random or representative field compaction tests should be performed in accordance with test methods ASTM designation D 1556, D 2937 or D 2922, and D 3017, at intervals of approximately ±2 feet of fill height or approximately every 1,000 cubic yards placed. These criteria would vary depending on the soil conditions and the size of the project. The location and frequency of testing would be at the discretion of the geotechnical consultant.

Contractor's Responsibility

All clearing, site preparation, and earthwork performed on the project should be conducted by the contractor, with observation by a geotechnical consultant, and staged approval by the governing agencies, as applicable. It is the contractor's responsibility to prepare the ground surface to receive the fill, to the satisfaction of the geotechnical consultant, and to place, spread, moisture condition, mix, and compact the fill in accordance with the recommendations of the geotechnical consultant. The contractor should also remove all non-earth material considered unsatisfactory by the geotechnical consultant.

Notwithstanding the services provided by the geotechnical consultant, it is the sole responsibility of the contractor to provide adequate equipment and methods to accomplish the earthwork in strict accordance with applicable grading guidelines, latest adopted Codes or agency ordinances, geotechnical report(s), and approved grading plans. Sufficient watering apparatus and compaction equipment should be provided by the contractor with due consideration for the fill material, rate of placement, and climatic conditions. If, in the opinion of the geotechnical consultant, unsatisfactory conditions such as questionable weather, excessive oversized rock or deleterious material, insufficient support equipment, etc., are resulting in a quality of work that is not acceptable, the consultant will inform the contractor, and the contractor is expected to rectify the conditions, and if necessary, stop work until conditions are satisfactory.

During construction, the contractor shall properly grade all surfaces to maintain good drainage and prevent ponding of water. The contractor shall take remedial measures to control surface water and to prevent erosion of graded areas until such time as permanent drainage and erosion control measures have been installed.

SITE PREPARATION

All major vegetation, including brush, trees, thick grasses, organic debris, and other deleterious material, should be removed and disposed of off-site. These removals must be concluded prior to placing fill. In-place existing fill, soil, alluvium, colluvium, or rock materials, as evaluated by the geotechnical consultant as being unsuitable, should be removed prior to any fill placement. Depending upon the soil conditions, these materials may be reused as compacted fills. Any materials incorporated as part of the compacted fills should be approved by the geotechnical consultant.

Any underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipelines, or other structures not located prior to grading, are to be removed or treated in a manner recommended by the geotechnical consultant. Soft, dry, spongy, highly fractured, or otherwise unsuitable ground, extending to such a depth that surface processing cannot adequately improve the condition, should be overexcavated down to firm ground and approved by the geotechnical consultant before compaction and filling operations continue. Overexcavated and processed soils, which have been properly mixed and moisture conditioned, should be re-compacted to the minimum relative compaction as specified in these guidelines.

Existing ground, which is determined to be satisfactory for support of the fills, should be scarified (ripped) to a minimum depth of 6 to 8 inches, or as directed by the geotechnical consultant. After the scarified ground is brought to optimum moisture content, or greater and mixed, the materials should be compacted as specified herein. If the scarified zone is greater than 6 to 8 inches in depth, it may be necessary to remove the excess and place the material in lifts restricted to about 6 to 8 inches in compacted thickness.

Existing ground which is not satisfactory to support compacted fill should be overexcavated as required in the geotechnical report, or by the on-site geotechnical consultant. Scarification, disc harrowing, or other acceptable forms of mixing should continue until the soils are broken down and free of large lumps or clods, until the working surface is reasonably uniform and free from ruts, hollows, hummocks, mounds, or other uneven features, which would inhibit compaction as described previously.

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical [h:v]), the ground should be stepped or benched. The lowest bench, which will act as a key, should be a minimum of 15 feet wide and should be at least 2 feet deep into firm material, and approved by the geotechnical consultant. In fill-over-cut slope conditions, the recommended minimum width of the lowest bench or key is also 15 feet, with the key founded on firm material, as designated by the geotechnical consultant. As a general rule, unless specifically recommended otherwise by the geotechnical consultant, the minimum width of fill keys should be equal to ½ the height of the slope.

Standard benching is generally 4 feet (minimum) vertically, exposing firm, acceptable material. Benching may be used to remove unsuitable materials, although it is understood that the vertical height of the bench may exceed 4 feet. Pre-stripping may be considered for unsuitable materials in excess of 4 feet in thickness.

All areas to receive fill, including processed areas, removal areas, and the toes of fill benches, should be observed and approved by the geotechnical consultant prior to placement of fill. Fills may then be properly placed and compacted until design grades (elevations) are attained.

COMPACTED FILLS

Any earth materials imported or excavated on the property may be used in the fill provided that each material has been evaluated to be suitable by the geotechnical consultant.

These materials should be free of roots, tree branches, other organic matter, or other deleterious materials. All unsuitable materials should be removed from the fill as directed by the geotechnical consultant. Soils of poor gradation, undesirable expansion potential, or substandard strength characteristics may be designated by the consultant as unsuitable and may require blending with other soils to serve as a satisfactory fill material.

Fill materials derived from benching operations should be dispersed throughout the fill area and blended with other approved material. Benching operations should not result in the benched material being placed only within a single equipment width away from the fill/bedrock contact.

Oversized materials defined as rock, or other irreducible materials, with a maximum dimension greater than 12 inches, should not be buried or placed in fills unless the location of materials and disposal methods are specifically approved by the geotechnical consultant. Oversized material should be taken offsite, or placed in accordance with recommendations of the geotechnical consultant in areas designated as suitable for rock disposal. GSI anticipates that soils to be used as fill material for the subject project may contain some rock. Appropriately, the need for rock disposal may be necessary during grading operations on the site. From a geotechnical standpoint, the depth of any rocks, rock fills, or rock blankets, should be a sufficient distance from finish grade. This depth is generally the same as any overexcavation due to cut-fill transitions in hard rock areas, and generally facilitates the excavation of structural footings and substructures. Should deeper excavations be proposed (i.e., deepened footings, utility trenching, swimming pools, spas, etc.), the developer may consider increasing the hold-down depth of any rocky fills to be placed, as appropriate. In addition, some agencies/jurisdictions mandate a specific hold-down depth for oversize materials placed in fills. The hold-down depth, and potential to encounter oversize rock, both within fills, and occurring in cut or natural areas, would need to be disclosed to all interested/affected parties. Once approved by the governing agency, the hold-down depth for oversized rock (i.e., greater than 12 inches) in fills on this project is provided as 10 feet, unless specified differently in the text of this report. The governing agency may require that these materials need to be deeper, crushed, or reduced to less than 12 inches in maximum dimension, at their discretion.

To facilitate future trenching, rock (or oversized material), should not be placed within the hold-down depth feet from finish grade, the range of foundation excavations, future utilities, or underground construction unless specifically approved by the governing agency, the geotechnical consultant, and the developer's representative.

If import material is required for grading, representative samples of the materials to be used as compacted fill should be analyzed in the laboratory by the geotechnical consultant to evaluate it's physical properties and suitability for use onsite. Such testing should be performed three (3) days prior to importation. If any material other than that previously tested is encountered during grading, an appropriate analysis of this material should be conducted by the geotechnical consultant as soon as possible.

Approved fill material should be placed in areas prepared to receive fill in near horizontal layers, that when compacted, should not exceed about 6 to 8 inches in thickness. The geotechnical consultant may approve thick lifts if testing indicates the grading procedures are such that adequate compaction is being achieved with lifts of greater thickness. Each layer should be spread evenly and blended to attain uniformity of material and moisture suitable for compaction.

Fill layers at a moisture content less than optimum should be watered and mixed, and wet fill layers should be aerated by scarification, or should be blended with drier material. Moisture conditioning, blending, and mixing of the fill layer should continue until the fill materials have a uniform moisture content at, or above, optimum moisture.

After each layer has been evenly spread, moisture conditioned, and mixed, it should be uniformly compacted to a minimum of 90 percent of the maximum density as evaluated by ASTM test designation D 1557, or as otherwise recommended by the geotechnical consultant. Compaction equipment should be adequately sized and should be specifically designed for soil compaction, or of proven reliability to efficiently achieve the specified degree of compaction.

Where tests indicate that the density of any layer of fill, or portion thereof, is below the required relative compaction, or improper moisture is in evidence, the particular layer or portion shall be re-worked until the required density and moisture content has been attained. No additional fill shall be placed in an area until the last placed lift of fill has been tested and found to meet the density and moisture requirements, and is approved by the geotechnical consultant.

In general, per the latest adopted Code, fill slopes should be designed and constructed at a gradient of 2:1 (h:v), or flatter. Compaction of slopes should be accomplished by over-building a minimum of 3 feet horizontally, and subsequently trimming back to the design slope configuration. Testing shall be performed as the fill is elevated to evaluate compaction as the fill core is being developed. Special efforts may be necessary to attain the specified compaction in the fill slope zone. Final slope shaping should be performed by trimming and removing loose materials with appropriate equipment. A final evaluation of fill slope compaction should be based on observation and testing of the finished slope face. Where compacted fill slopes are designed steeper than 2:1 (h:v), prior approval from the governing agency, specific material types, a higher minimum relative compaction, special reinforcement, and special grading procedures will be recommended.

If an alternative to over-building and cutting back the compacted fill slopes is selected, then special effort should be made to achieve the required compaction in the outer 10 feet of each lift of fill by undertaking the following:

1. An extra piece of equipment consisting of a heavy, short-shanked sheepsfoot should be used to roll (horizontal) parallel to the slopes continuously as fill is placed. The sheepsfoot roller should also be used to roll perpendicular to the slopes, and extend out over the slope to provide adequate compaction to the face of the slope.

2. Loose fill should not be spilled out over the face of the slope as each lift is compacted. Any loose fill spilled over a previously completed slope face should be trimmed off or be subject to re-rolling.

3. Field compaction tests will be made in the outer (horizontal) ±2 to ±8 feet of the slope at appropriate vertical intervals, subsequent to compaction operations.

4. After completion of the slope, the slope face should be shaped with a small tractor and then re-rolled with a sheepsfoot to achieve compaction to near the slope face. Subsequent to testing to evaluate compaction, the slopes should be grid-rolled to achieve compaction to the slope face. Final testing should be used to evaluate compaction after grid rolling.

5. Where testing indicates less than adequate compaction, the contractor will be responsible to rip, water, mix, and recompact the slope material as necessary to achieve compaction. Additional testing should be performed to evaluate compaction.

SUBDRAIN INSTALLATION

Subdrains should be installed in approved ground in accordance with the approximate alignment and details indicated by the geotechnical consultant. Subdrain locations or materials should not be changed or modified without approval of the geotechnical consultant. The geotechnical consultant may recommend and direct changes in subdrain line, grade, and drain material in the field, pending exposed conditions. The location of constructed subdrains, especially the outlets, should be recorded/surveyed by the project civil engineer. Drainage at the subdrain outlets should be provided by the project civil engineer.

EXCAVATIONS

Excavations and cut slopes should be examined during grading by the geotechnical consultant. If directed by the geotechnical consultant, further excavations or overexcavation and refilling of cut areas should be performed, or remedial grading of cut slopes should be performed. When fill-over-cut slopes are to be graded, unless otherwise approved, the cut portion of the slope should be observed by the geotechnical consultant prior to placement of materials for construction of the fill portion of the slope. The geotechnical consultant should observe all cut slopes, and should be notified by the contractor when excavation of cut slopes commence.

If, during the course of grading, unforeseen adverse or potentially adverse geologic conditions are encountered, the geotechnical consultant should investigate, evaluate, and make appropriate recommendations for mitigation of these conditions. The need for cut slope buttressing or stabilizing should be based on in-grading evaluation by the geotechnical consultant, whether anticipated or not.

Unless otherwise specified in geotechnical and geological report(s), no cut slopes should be excavated higher or steeper than that allowed by the ordinances of controlling governmental agencies. Additionally, short-term stability of temporary cut slopes is the contractor's responsibility.

Erosion control and drainage devices should be designed by the project civil engineer and should be constructed in compliance with the ordinances of the controlling governmental agencies, and in accordance with the recommendations of the geotechnical consultant.

COMPLETION

Observation, testing, and consultation by the geotechnical consultant should be conducted during the grading operations in order to state an opinion that all cut and fill areas are graded in accordance with the approved project specifications. After completion of grading, and after the geotechnical consultant has finished observations of the work, final reports should be submitted, and may be subject to review by the controlling governmental agencies. No further excavation or filling should be undertaken without prior notification of the geotechnical consultant or approved plans.

All finished cut and fill slopes should be protected from erosion and be planted in accordance with the project specifications and as recommended by a landscape architect. Such protection and planning should be undertaken as soon as practical after completion of grading.

JOB SAFETY

General

At GSI, getting the job done safely is of primary concern. The following is the company's safety considerations for use by all employees on multi-employer construction sites. On-ground personnel are at highest risk of injury, and possible fatality, on grading and construction projects. GSI recognizes that construction activities will vary on each site, and that site safety is the prime responsibility of the contractor; however, everyone must be safety conscious and responsible at all times. To achieve our goal of avoiding accidents, cooperation between the client, the contractor, and GSI personnel must be maintained.

In an effort to minimize risks associated with geotechnical testing and observation, the following precautions are to be implemented for the safety of field personnel on grading and construction projects:

Safety Meetings: GSI field personnel are directed to attend contractor's regularly scheduled and documented safety meetings.

Safety Vests: Safety vests are provided for, and are to be worn by GSI personnel, at all times, when they are working in the field.

Safety Flags: Two safety flags are provided to GSI field technicians; one is to be affixed to the vehicle when on site, the other is to be placed atop the spoil pile on all test pits.

Flashing Lights: All vehicles stationary in the grading area shall use rotating or flashing amber beacons, or strobe lights, on the vehicle during all field testing. While operating a vehicle in the grading area, the emergency flasher on the vehicle shall be activated.

In the event that the contractor's representative observes any of our personnel not following the above, we request that it be brought to the attention of our office.

Test Pits Location, Orientation, and Clearance

The technician is responsible for selecting test pit locations. A primary concern should be the technician's safety. Efforts will be made to coordinate locations with the grading contractor's authorized representative, and to select locations following or behind the established traffic pattern, preferably outside of current traffic. The contractor's authorized representative (supervisor, grade checker, dump man, operator, etc.) should direct excavation of the pit and safety during the test period. Of paramount concern should be the soil technician's safety, and obtaining enough tests to represent the fill.

Test pits should be excavated so that the spoil pile is placed away from oncoming traffic, whenever possible. The technician's vehicle is to be placed next to the test pit, opposite the spoil pile. This necessitates the fill be maintained in a driveable condition. Alternatively, the contractor may wish to park a piece of equipment in front of the test holes, particularly in small fill areas or those with limited access.

A zone of non-encroachment should be established for all test pits. No grading equipment should enter this zone during the testing procedure. The zone should extend approximately 50 feet outward from the center of the test pit. This zone is established for safety and to avoid excessive ground vibration, which typically decreases test results.

When taking slope tests, the technician should park the vehicle directly above or below the test location. If this is not possible, a prominent flag should be placed at the top of the slope. The contractor's representative should effectively keep all equipment at a safe operational distance (e.g., 50 feet) away from the slope during this testing.

The technician is directed to withdraw from the active portion of the fill as soon as possible following testing. The technician's vehicle should be parked at the perimeter of the fill in a highly visible location, well away from the equipment traffic pattern. The contractor should inform our personnel of all changes to haul roads, cut and fill areas or other factors that may affect site access and site safety.

In the event that the technician's safety is jeopardized or compromised as a result of the contractor's failure to comply with any of the above, the technician is required, by company policy, to immediately withdraw and notify his/her supervisor. The grading contractor's representative will be contacted in an effort to affect a solution. However, in the interim, no further testing will be performed until the situation is rectified. Any fill placed can be considered unacceptable and subject to reprocessing, recompaction, or removal.

In the event that the soil technician does not comply with the above or other established safety guidelines, we request that the contractor bring this to the technician's attention and notify this office. Effective communication and coordination between the contractor's representative and the soil technician is strongly encouraged in order to implement the above safety plan.

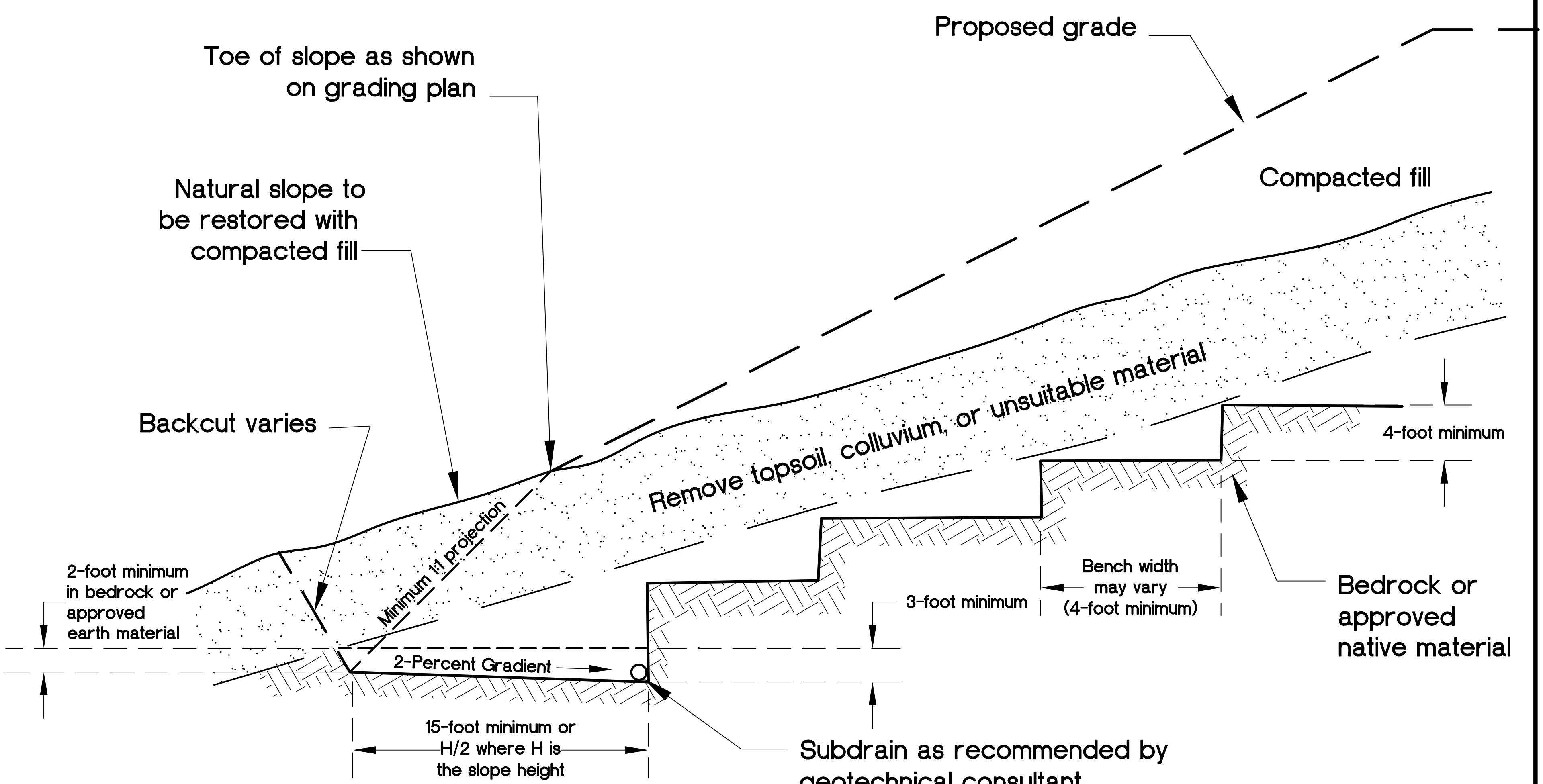
Trench and Vertical Excavation

It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or other debris which could fall into the trench; or 3) displays any other evidence of any unsafe conditions regardless of depth.

All trench excavations or vertical cuts in excess of 5 feet deep, which any person enters, should be shored or laid back. Trench access should be provided in accordance with Cal/OSHA and state, and local standards. Our personnel are directed not to enter any trench by being lowered or "riding down" on the equipment.

If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor's representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns or other reasons could be subject to reprocessing or removal.

If GSI personnel become aware of anyone working beneath an unsafe trench wall or vertical excavation, we have a legal obligation to put the contractor and owner/developer on notice to immediately correct the situation. If corrective steps are not taken, GSI then has an obligation to notify Cal/OSHA and the proper controlling authorities.

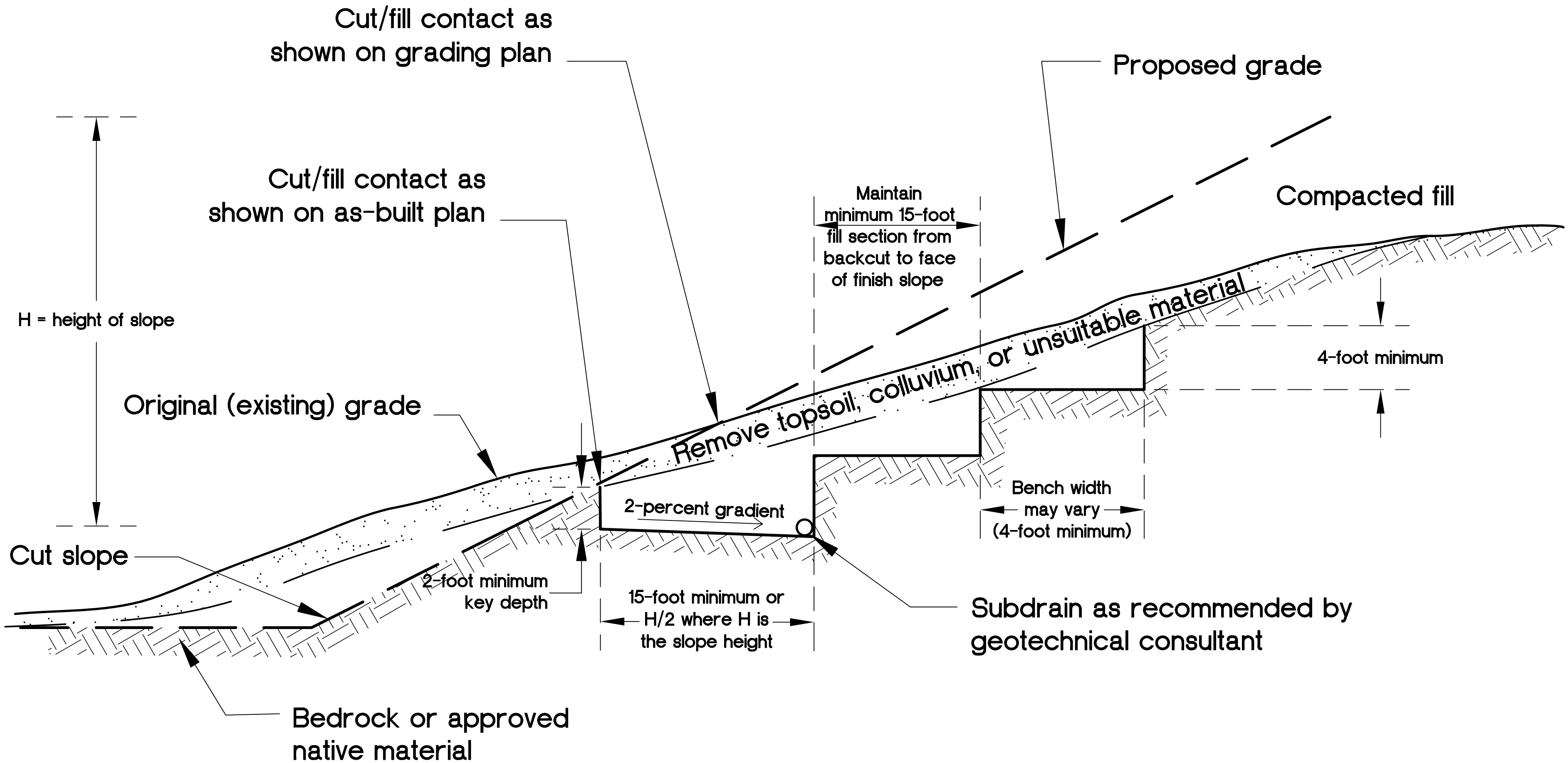


NOTES:

1. Where the natural slope approaches or exceeds the design slope ratio, special recommendations would be provided by the geotechnical consultant.

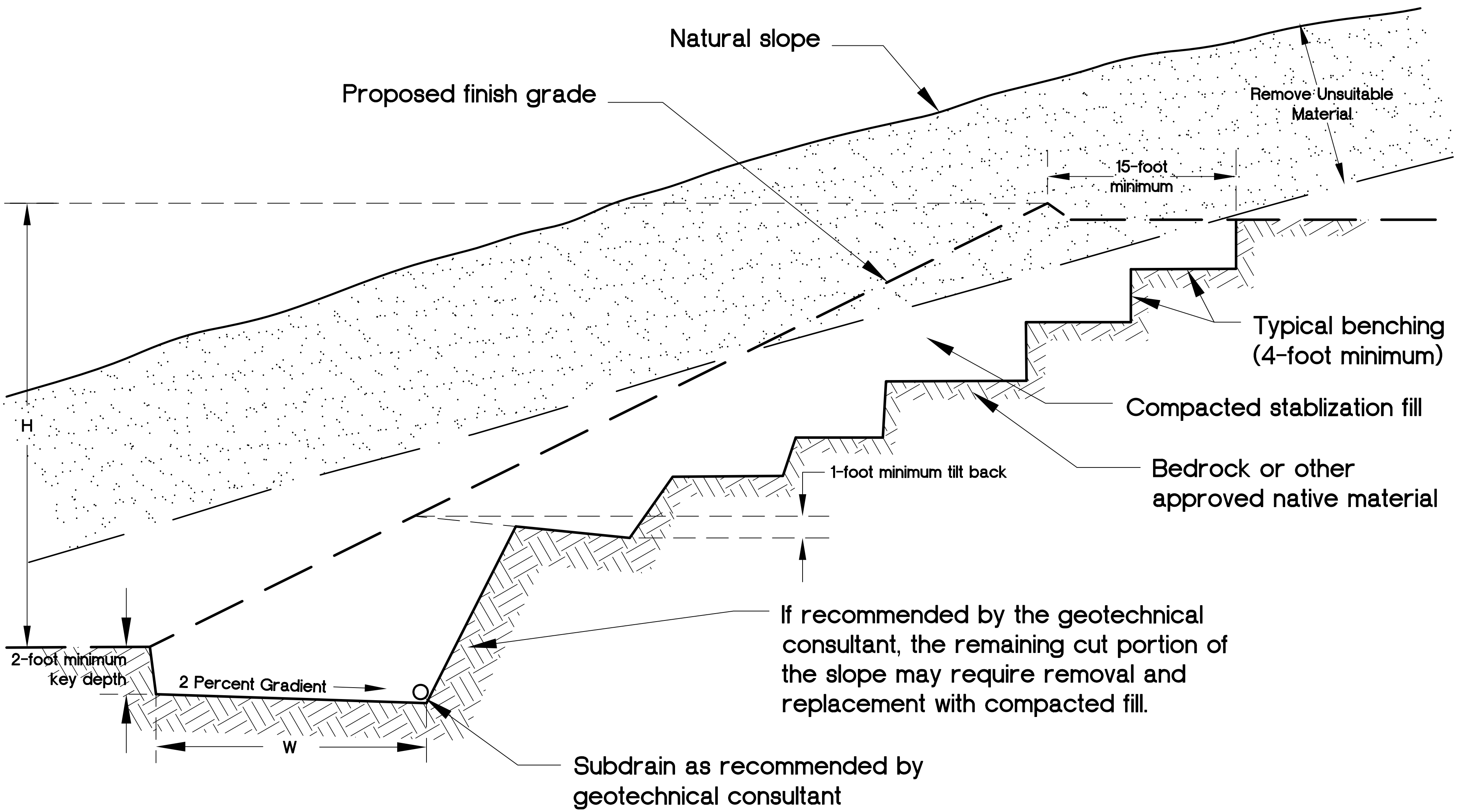
2. The need for and disposition of drains should be evaluated by the geotechnical consultant, based upon exposed conditions.

GeoSoils, Inc.	FILL OVER NATURAL (SIDEHILL FILL) DETAIL	Plate F-7



NOTE: The cut portion of the slope should be excavated and evaluated by the geotechnical consultant prior to construction of the fill portion.

GeoSoils, Inc.	FILL OVER CUT DETAIL	Plate F-8

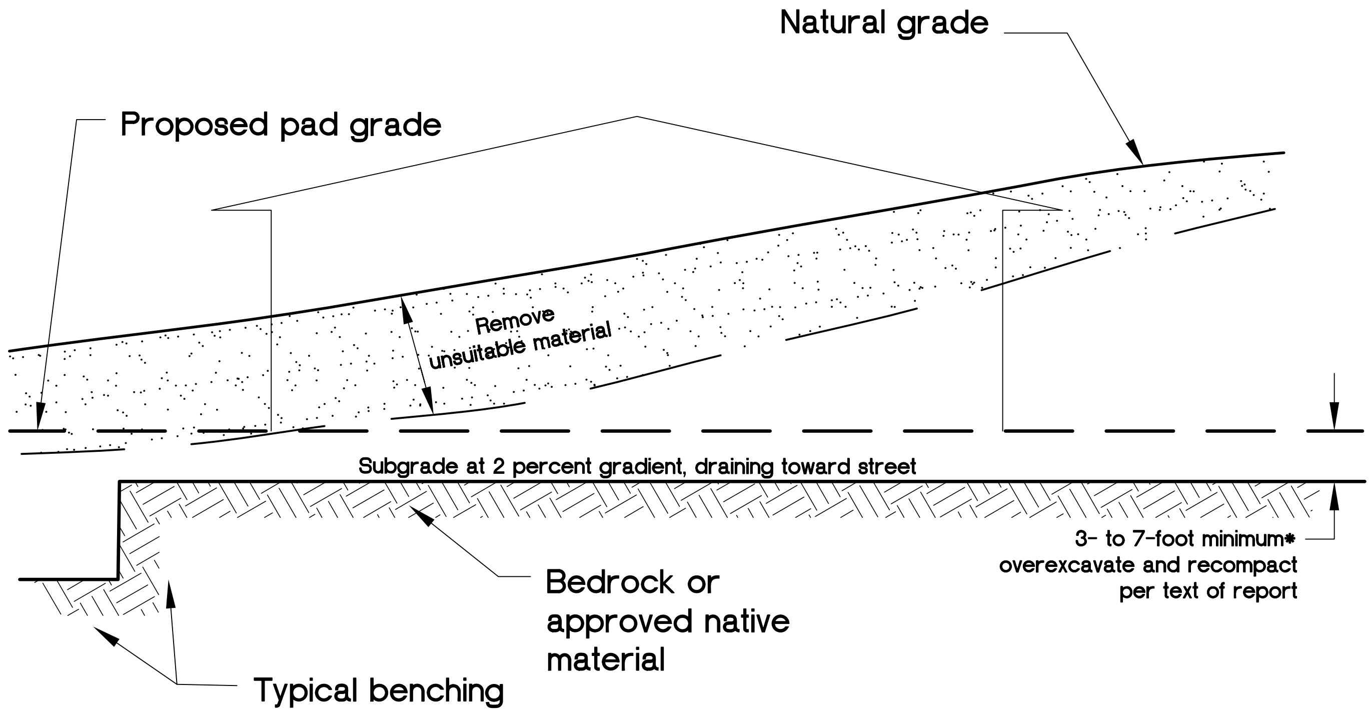


NOTES:

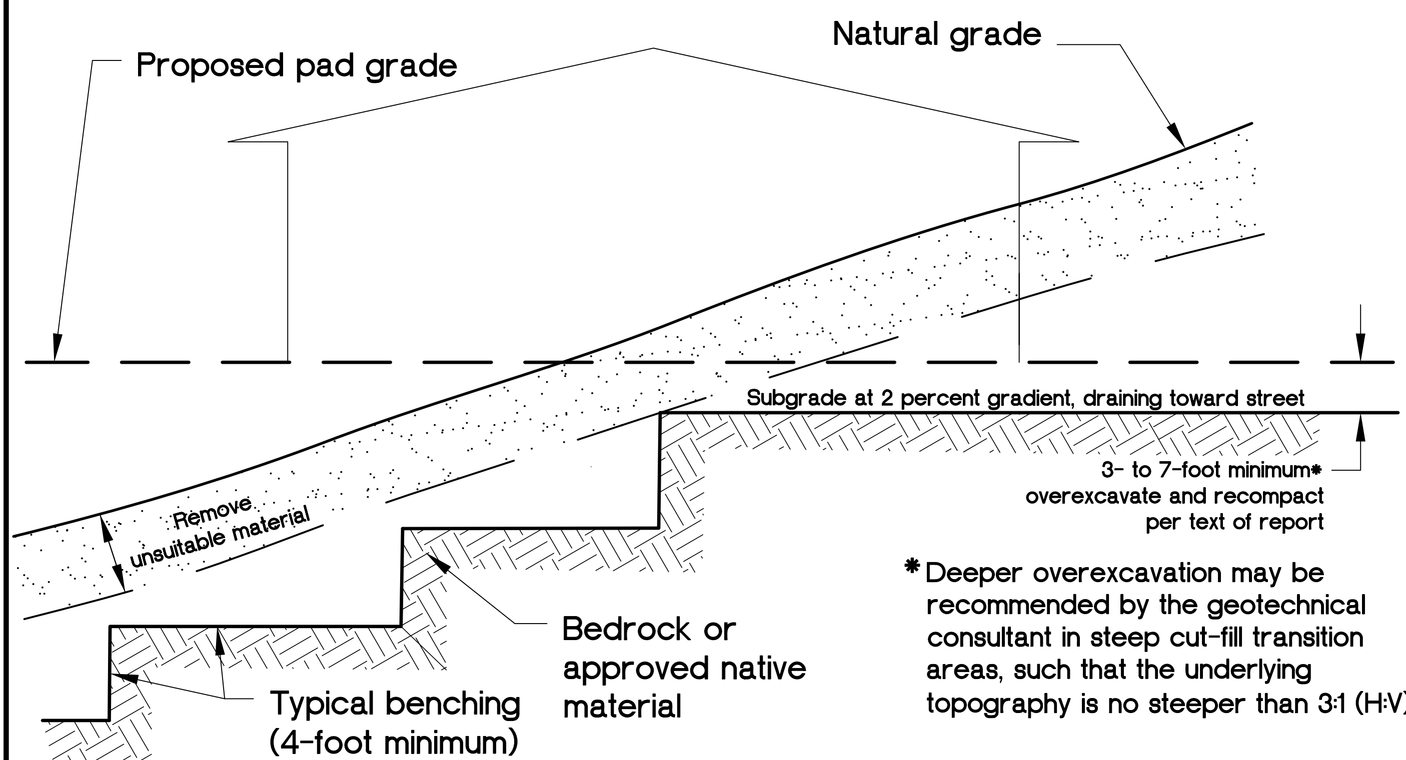
1. Subdrains may be required as specified by the geotechnical consultant.

2 W shall be equipment width (15 feet) for slope heights less than 25 feet. For slopes greater than 25 feet, W shall be evaluated by the geotechnical consultant. At no time, shall W be less than H/2, where H is the height of the slope.

GeoSoils, Inc.	STABLIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN CUT SLOPE DETAIL	Plate F-9



CUT LOT OR MATERIAL-TYPE TRANSITION

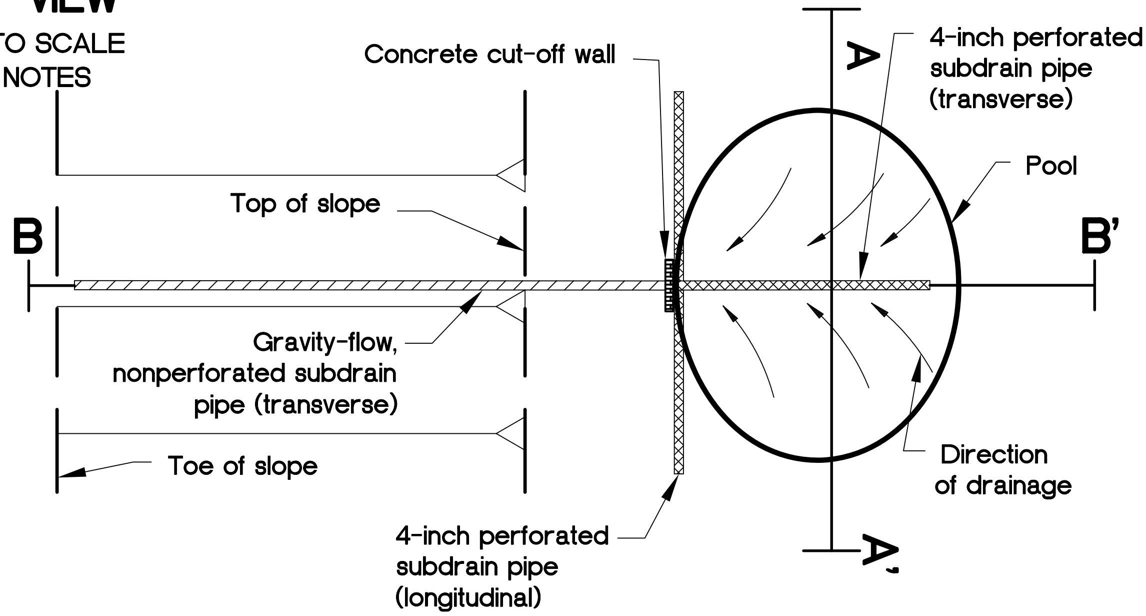


CUT-FILL LOT (DAYLIGHT TRANSITION)

GeoSoils, Inc. | TRANSITION LOT DETAILS | Plate F-12

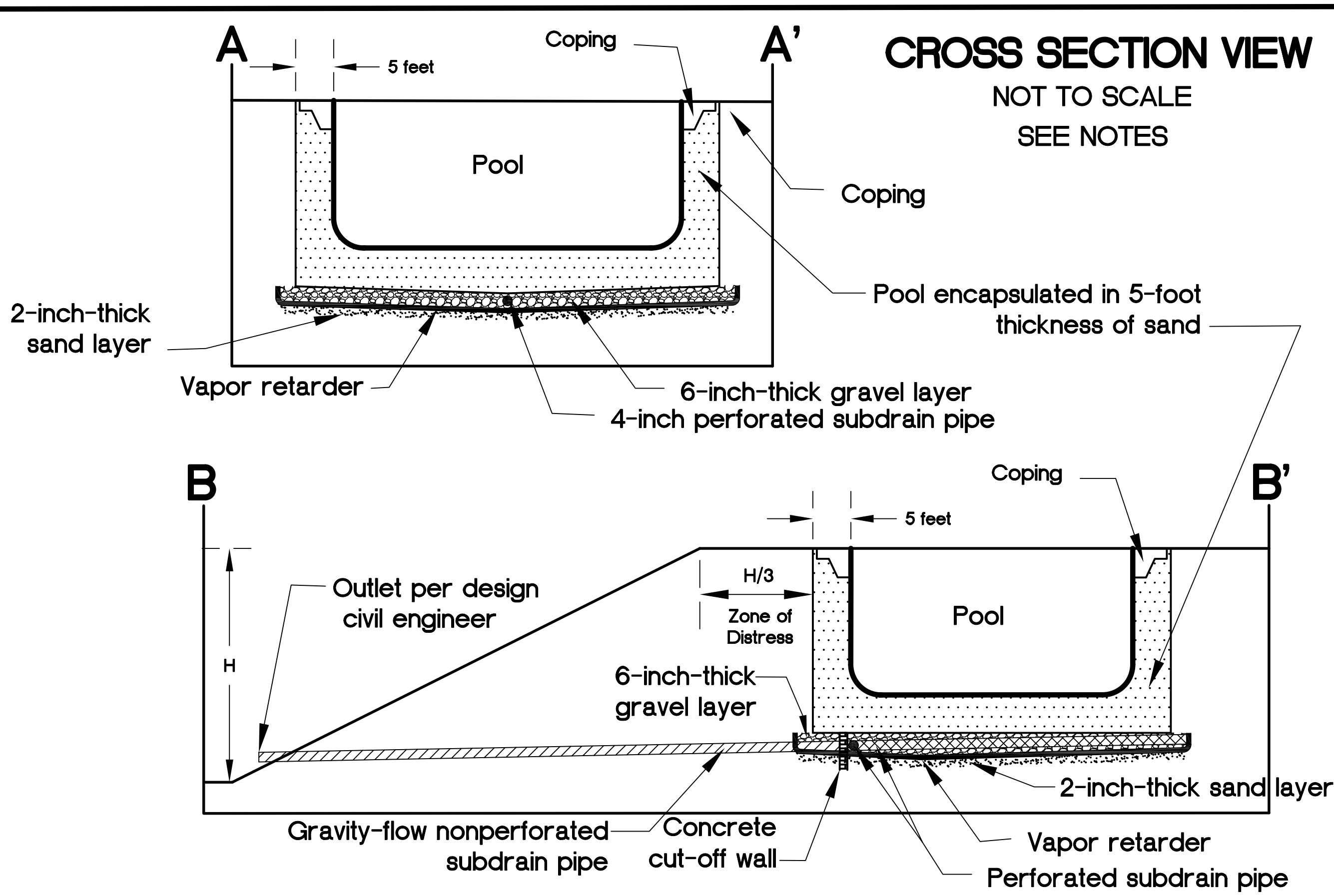
MAP VIEW

NOT TO SCALE
SEE NOTES



CROSS SECTION VIEW

NOT TO SCALE
SEE NOTES

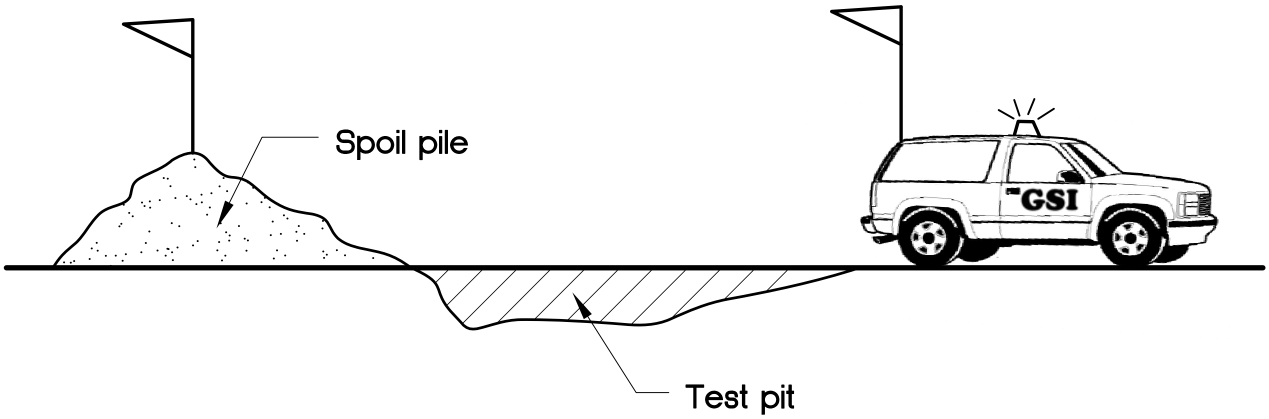


NOTES:

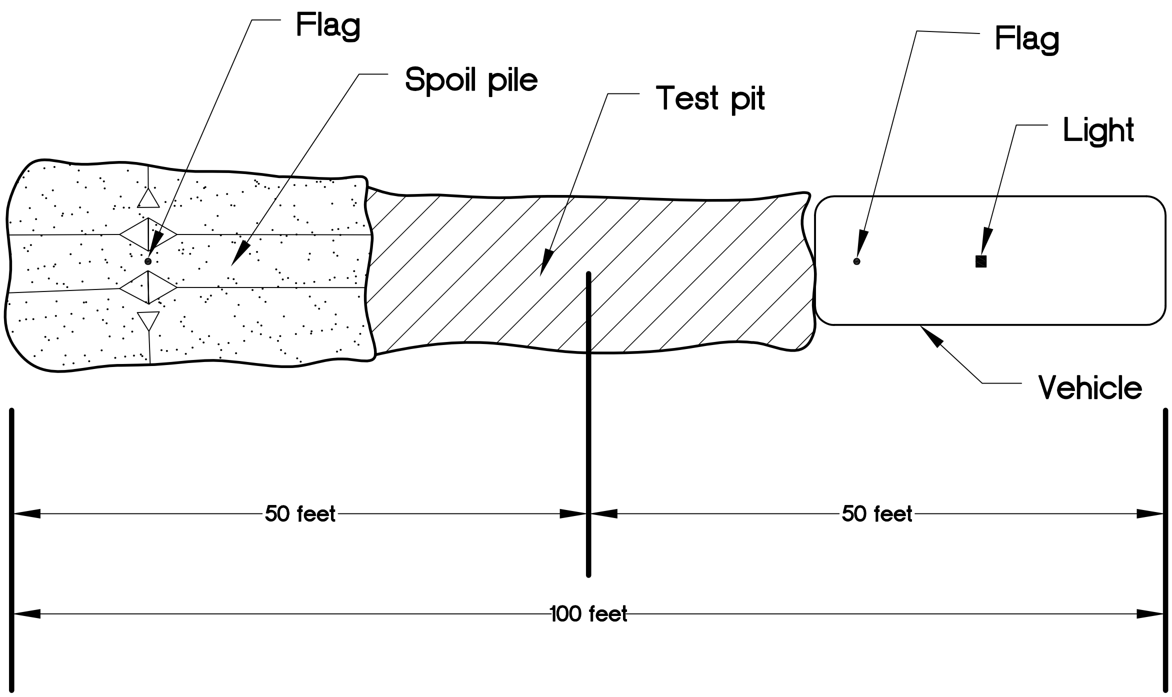
1. 6-inch-thick, clean gravel (3/4 to 1½ inch) sub-base encapsulated in Mirafi 140N or equivalent, underlain by a 15-mil vapor retarder, with 4-inch-diameter perforated pipe longitudinal connected to 4-inch-diameter perforated pipe transverse. Connect transverse pipe to 4-inch-diameter nonperforated pipe at low point and outlet or to sump pump area.
2. Pools on fills thicker than 20 feet should be constructed on deep foundations; otherwise, distress (tilting, cracking, etc.) should be expected.
3. Design does not apply to infinity-edge pools/spas.

GeoSoils, Inc.	TYPICAL POOL/SPA DETAIL	Plate F-17

SIDE VIEW



TOP VIEW



GSI LEGEND

Afu — ARTIFICIAL FILL - UNDOCUMENTED

Qop — QUATERNARY OLD PARALIC DEPOSITS, CIRCLED WHERE BURIED

Kp — CRETACEOUS POINT LOMA FORMATION, CIRCLED WHERE BURIED

? — APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN

30° — REGIONAL BEDDING ATTITUDE WITH DIP IN DEGREES, (KENNEDY & TAN, 2008)

40° — REGIONAL BEDDING ATTITUDE WITH DIP IN DEGREES, (KENNEDY, 1975)

B-2 TD=19' — APPROXIMATE LOCATION OF EXPLORATORY BORING WITH TOTAL DEPTH IN FEET

C C' — LOCATION OF GEOLOGIC CROSS SECTION

N.A.P. — NOT A PART OF THIS STUDY

N

GRAPHIC SCALE

20 0 10 20 40

1" = 20'

ISLAND ARCHITECTS

TONY CRISAFI, LISA KRIEDEMAN, R.A

NOT FOR CONSTRUCTION

COAST WALK - HOUSE 2

1555 Coast Blvd., La Jolla CA 92037

A1.1

SITE PLAN

01/05/22

ALL LOCATIONS ARE APPROXIMATE

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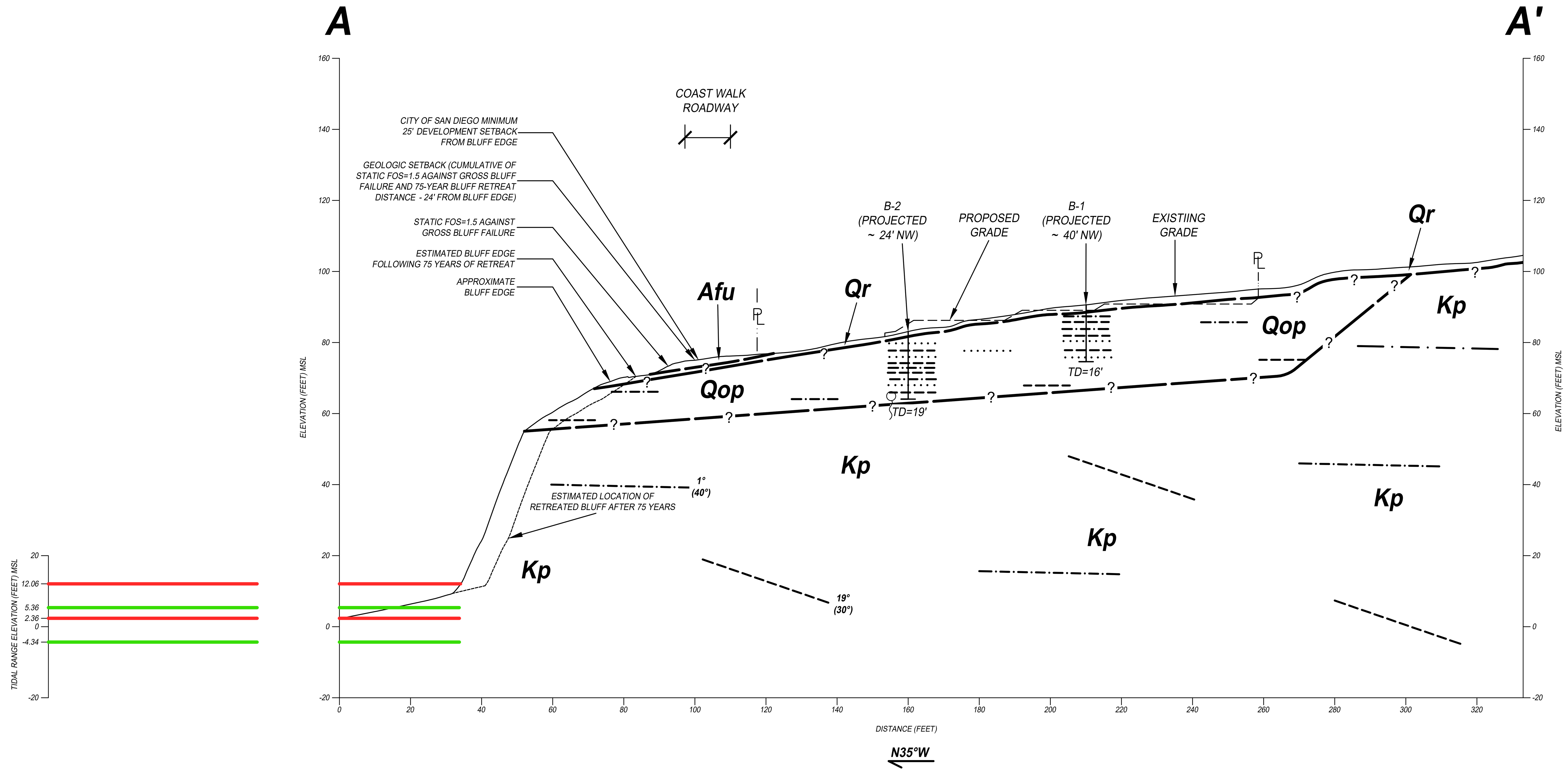
GEOTECHNICAL MAP

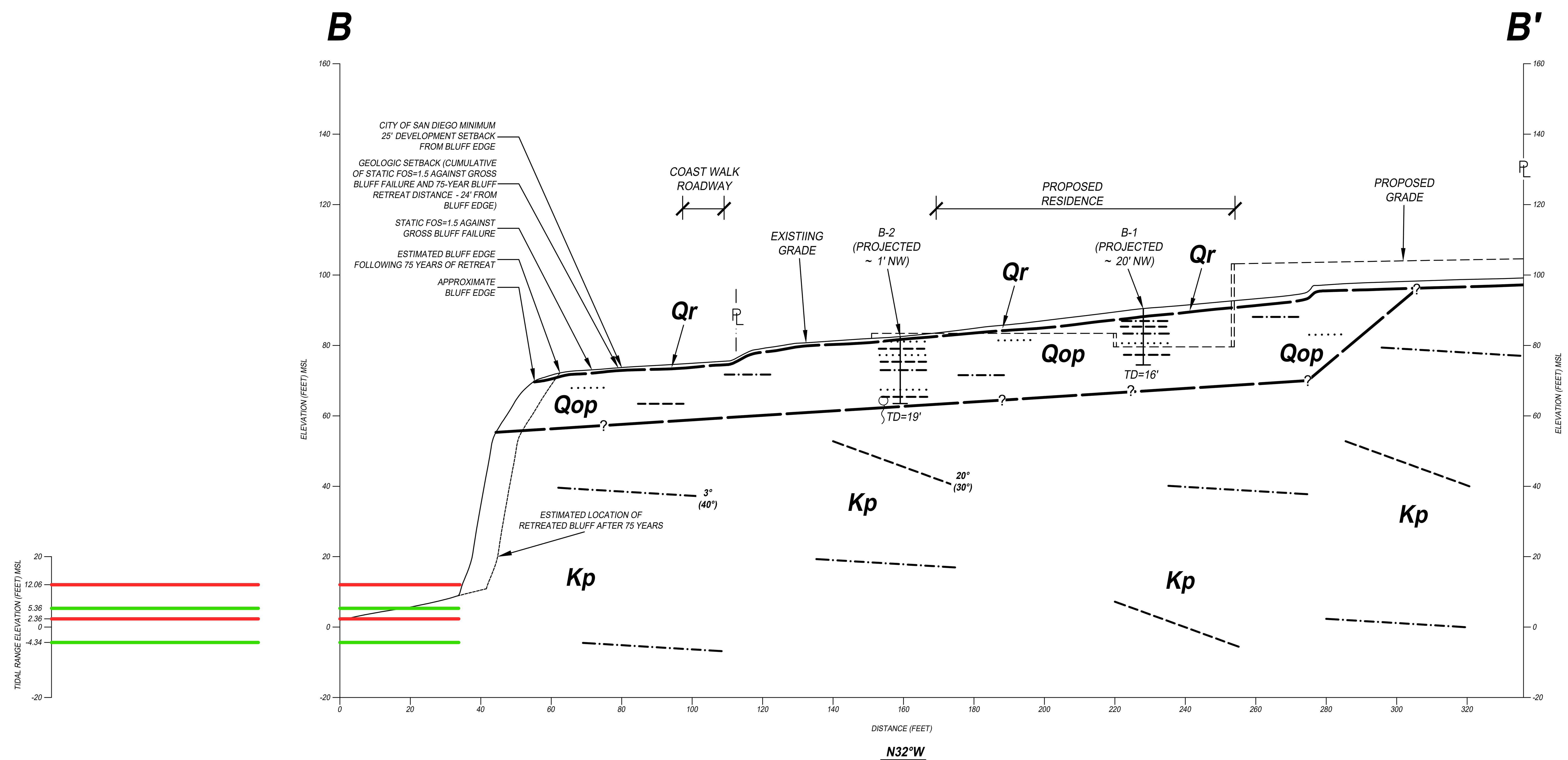
Plate 1

W.O. 8358-A-SC	DATE: 08/22	SCALE: 1" = 20'

GSI LEGEND

- **Afu** — ARTIFICIAL FILL - UNDOCUMENTED
- **Qr** — QUATERNARY RESIDUAL SOIL
- **Qop** — QUATERNARY OLD PARALIC DEPOSITS
- **Kp** — CRETACEOUS POINT LOMA FORMATION
- — APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN
- — APPROXIMATE LOCATION OF PERCHED GROUNDWATER SEEPAGE
- 20° (30°) — REGIONAL BEDDING WITH APPARENT DIP IN DEGREES, AND TRUE DIP IN PARENTHESES (KENNEDY & TAN, 2008)
- 3° (40°) — REGIONAL BEDDING WITH APPARENT DIP IN DEGREES, AND TRUE DIP IN PARENTHESES (KENNEDY, 1975)
- PL — APPROXIMATE LOCATION OF PROPERTY LINE
- — CURRENT TIDAL RANGE 2022
- — TIDAL RANGE 2097 W/ 0.5% PROBILITY OF EXCEEDANCE W/ 6.7 FT SLR (75 YEARS)





GRAPHIC SCALE 1" = 20'

ALL LOCATIONS ARE APPROXIMATE

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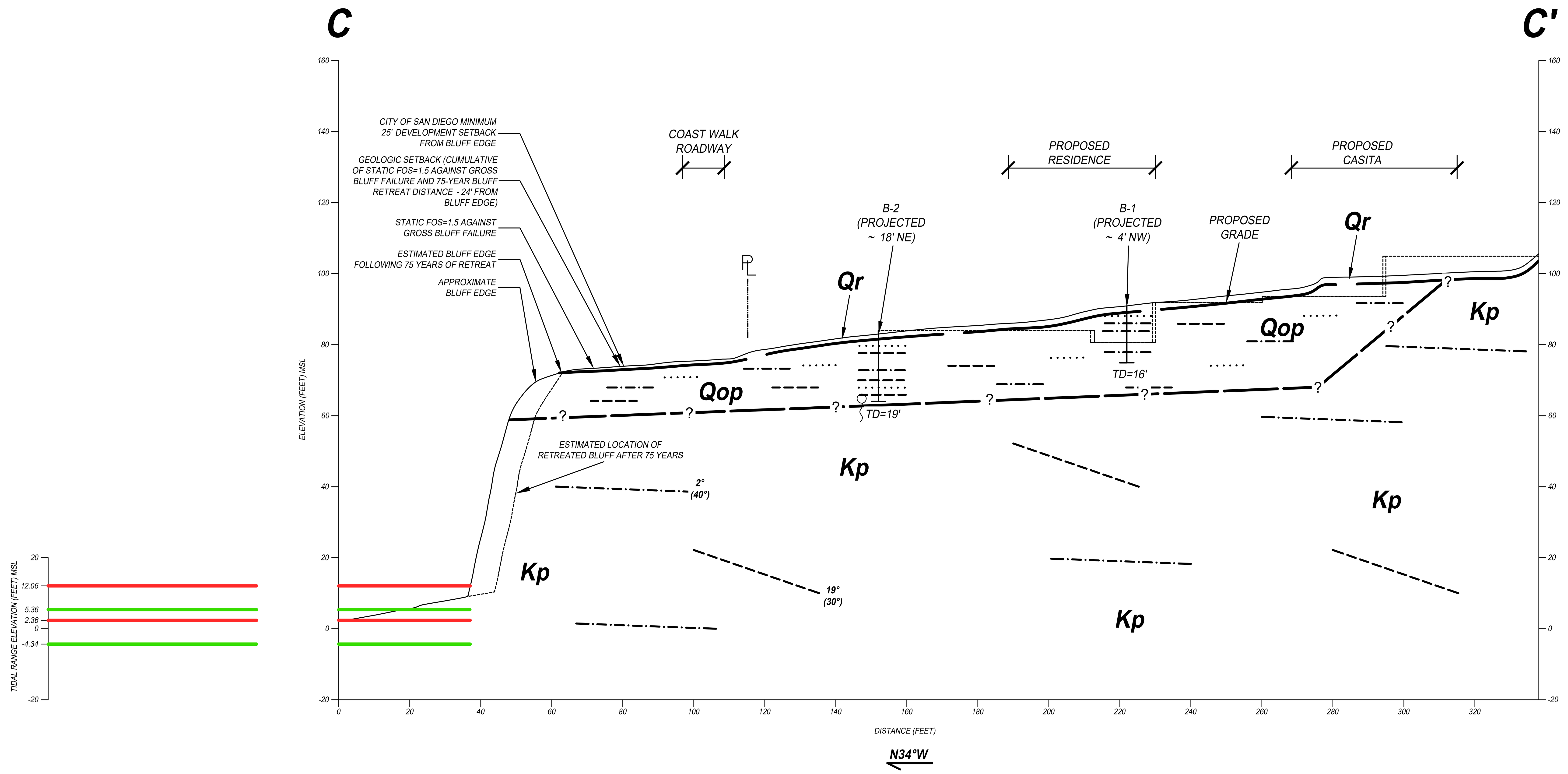
GEOLOGIC CROSS SECTIONS A-A' & B-B'

Plate 2

W.O. 8358-A-SC	DATE: 08/22	SCALE: 1" = 20'

SEE PLATE 2 FOR LEGEND



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GEOLOGIC CROSS SECTION C-C'

Plate 3

W.O. 8358-A-SC	DATE: 08/22	SCALE: 1" = 20'