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A-6-ENC-24-0046 (Bevand Residence)

December 12, 2024

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Coastal Property Rights, Land Use & Litigation

December 6, 2024

VIA EMAIL

California Coastal Commission
c/o Camila Pauda, Coastal Planner
7575 Metropolitan Drive, Suite 103
San Diego, California 92108
camila.pauda@coastal.ca.gov

RE: Appeal No. A-6-ENC-24-0046 (Bevand, Encinitas)

Dear Chair Hart, Vice Chair Escalante, and Honorable Commissioners:

Our firm represents Marc Bevand, the applicant in this matter. For the reasons outlined below, the Commission should find that this appeal raises no substantial issue.

There is no substantial issue with regard to the applicant's geotechnical analysis

The staff report does not dispute that the applicant submitted a geotechnical report that addresses all of the eleven issues required by the Encinitas LCP (Encinitas Mun. Code, § 30.34.020.D). The staff report contends, however, that (1) "the shear strength values used for the Torrey Sandstone in the slope stability analysis are particularly high and not supported by laboratory testing data"; (2) the estimated future erosion rate of 0.22 per year is inadequate; and (3) "[e]ven if this retreat rate was deemed adequate, using the additive approach required by the LCP and this chosen retreat rate of 0.22 ft/yr would result in a setback of 46.5 feet, thus making the principal structure's proposed setback of 42 feet inadequate." (Staff Rpt., pp. 12–13.)

As to the shear strength, the staff report does not provide *any* evidence to contradict the conclusions of the applicant's geotechnical engineers and does not provide *any* alternative analysis or even suggest what an appropriate range of values should be for shear strength. The staff report suggests vaguely that the values "are particularly high" (Staff Rpt., p. 12) but does not cite any support for this proposition. The staff report thus utterly fails to demonstrate a substantial issue.

The staff report also faults the applicant's geotechnical report because the shear value is "not supported by laboratory testing data." (*Ibid.*) The staff report does not cite any requirement in the LCP for laboratory testing data. Regardless, the staff report is incorrect. The applicant's geotechnical report states: "Laboratory tests were performed on representative samples of the onsite earth materials in order to evaluate their physical characteristics." (Ex. A, p. 37.) In addition, the geotechnical report relies on "[s]hear testing [that] was previously performed on undisturbed samples of the old paralic deposits and Torrey Sandstone collected during our field work at nearby 330 Neptune Avenue in preparation of GSI (2000)." (*Ibid.*) The testing data are reported in appendices B and D to the geotechnical report. The applicant's geotechnical engineers analyzed the data to determine slope stability using the accepted, industry-standard Modified Bishop and GLE (Spencer's) methods. (*Id.* at 39.) The results are summarized at pages 38–43 of the geotechnical report and further detailed at Appendix D. The staff report's claim that the shear value is "not supported by laboratory testing data" is a sheer fabrication.

As to erosion rates, the historical rate was determined through a sophisticated photogrammatic analysis performed by a licensed land surveyor, based on high-resolution aerial and satellite photographs and a ground survey performed on March 9, 2020. (Ex. B.) This analysis shows that the bluff edge retreated only 2.3 feet between the years 1932 and 2020—a time period of 88 years. (*Ibid.*) Conservatively applying the outside margin of error, "this yields 5.7 and 8.0 feet, respectively, in 88 years. This is equivalent to a site specific historic rate of 0.065 (low) to 0.091(high) feet/year." (Ex. A, Executive Summary, p. 2.) The only contrary evidence vaguely referred to in the staff report is a regional (not site-specific) study of estimated (not measured) erosion rates. This is insufficient to establish a substantial issue.

Using the historic erosion rate as a starting point, the applicant's geotechnical engineer estimated future erosion rates for the expected 75-year life of the structure, using USGS's CoSMoS computer application and appropriately increasing the erosion rate over time to account for predicted sea-level rise until reaching a rate of 0.15 feet annually by the year 2096 under a low-risk scenario and 0.211 under a high-risk scenario. (Ex. A, pp. 15–26.) Conservatively assuming the high-risk scenario, the estimated future retreat of the bluff adds up to 9.39 feet over a 75-year period. (*Id.* at 23.) The City's geotechnical engineer independently reviewed the applicant's report and concurred with its analysis of erosion rates. (Ex. C.) The staff report's statement that there is "no data supporting this chosen rate" is utterly false. (Staff Rpt., p. 13.)

Finally, staff demonstrates a misunderstanding of the geotechnical report when staff states that "a retreat rate of 0.22 ft/yr would result in a setback of 46.5 feet …." (Staff Rpt., p. 13.) The geotechnical report's analysis does not show a retreat rate of 0.22 for the entire 75-year future period.[1] Rather, 0.22 is the asymptotic erosion rate reached at the end of 75 years. Adding up the erosion at lower annual rates in the early years and at the higher rates in later years gives a total retreat of 9.39 feet, as noted above. (Ex. A, p. 23.) Thirty feet (representing the 1.5-factor-of-safety line) plus 9.39 feet (representing total bluff retreat over 75 years) equals 39.39 feet, well within the 42-foot setback for this project and providing a comfortable margin of error.

There is no substantial issue with regard to the basement

The staff report's concerns over the project's basement are misplaced. Staff cite nothing in the Encinitas LCP prohibiting basements. Over the years, the City of Encinitas has permitted many

[1] An erosion rate of 0.211 can be conservatively rounded up to 0.22 feet per year, which is presumably why the latter number is sometimes cited.

basements under its LCP, and until recently the Coastal Commission has never taken issue with them.

Even if there were some general policy against basements, the staff report contradicts itself. As staff acknowledge, there is an *existing* basement, and the applicant proposes to utilize this existing basement in the project. (Staff Rpt., p. 14.) If removal of the basement were not feasible, or if removal would somehow cause "destructive erosion or collapse," as suggested by staff, then staff should not be demanding that the applicant remove the basement as part of this project! Staff's argument is nonsensical. The least impactful course of action is to leave the basement in place.

Staff's assertion that "the basement could act as shoreline protection in the future if erosion occurs on the site" (Staff Rpt., p. 14) is absurd. The floor of the basement is 76.8 feet above sea level. (Ex. D.) The basement would have to sink some 70 feet before it could ever come into contact with the shoreline, even at the highest of high tides.

There is no substantial issue with regard to shoreline protection

It is undisputed that the existing seawall at the base of the bluff is legal and permitted. Consistent with the Encinitas LCP, the applicant has demonstrated that the proposed new structure "is expected to be reasonably safe from failure and erosion over its lifetime without having to propose any shore or bluff stabilization to protect the structure in the future," as explained above.[2] (Encinitas Mun. Code, § 30.34.020.D.) The staff report does not cite any provision of the Encinitas LCP requiring an existing, legal, permitted seawall to be removed when an existing structure on the blufftop is rebuilt. Nor does the staff report cite any provision of the LCP requiring the City to "address the feasibility of removal of the seawall at this time …." (Staff Rpt., p. 15.) That question can be addressed when the seawall reaches the end of its useful life.

The staff report states that "[t]he City also did not require the property owners to assume the current and future risks in the form of a deed restriction and waiver of rights to any future shoreline armoring, which represents another inconsistency with the City's LCP." (Staff Rpt., p. 15.) But the staff report does not cite any provision of the LCP requiring the applicant to execute such a deed restriction and waiver. Further, in stating that section 30.34.020.D of the LCP "prohibits new development from requiring future shoreline protection" (Staff Rpt., p. 15), the staff report undermines its argument. Assuming this statement is correct, if the applicant at some future time applies for a permit to construct shore protection to protect his new development, such application will be denied, and a waiver would be superfluous. Staff's desire to obtain a waiver from the applicant is not a reason to delay his project and drag him through a de novo hearing.

There is no substantial issue with regard to assumption of risk

Finally, the staff report contends that the project is inconsistent with the LCP because "the City did not condition an assumption of risk" for "potential hazards." (Staff Rpt., p. 16.) This statement is simply false. Condition BD 01 of the City's approval, attached to the staff report as Exhibit No.

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[2] All estimates of future erosion in the applicant's geotechnical report assume an unprotected bluff, as the staff report acknowledges. (Staff Rpt., p. 15.)

4, requires that "Owner(s) shall enter into and record a covenant satisfactory to the City Attorney waiving any claims of liability against the City and agreeing to indemnify and hold harmless the City and City's employees relative to the approved project. This covenant is applicable to any bluff failure and erosion resulting from the development project." In any event, staff does not cite any provision of the LCP requiring such a condition. Hence, there can be no inconsistency with the LCP.

Conclusion

For the foregoing reasons, the appeal does not raise a substantial issue as to consistency with the Encinitas LCP. We respectfully request that the Commission enter a finding of no substantial issue.

Sincerely,

AANNESTAD ANDELIN & CORN LLP

Lee M. Andelin

cc: Chandra Slaven

Enclosures

Exhibit A

UPDATE PRELIMINARY GEOTECHNICAL EVALUATION
PROPOSED RESIDENTIAL CONSTRUCTION, 312 NEPTUNE AVENUE
ENCINITAS, SAN DIEGO COUNTY, CALIFORNIA 92024
ASSESSOR'S PARCEL NUMBER (APN) 256-352-04-00

FOR

MR. MARC BEVAND
312 NEPTUNE AVENUE
ENCINITAS, CALIFORNIA 92024

W.O. 7638-A2-SC DECEMBER 17, 2019
UPDATED MAY 11, 2021

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

5741 Palmer Way • Carlsbad, California 92010 • (760) 438-3155 • FAX (760) 931-0915 • www.geosoilsinc.com

December 17, 2019
Updated May 11, 2021

W.O. 7638-A2-SC

Mr. Marc Bevand
312 Neptune Avenue
Encinitas, California 92024

Subject: Update Preliminary Geotechnical Evaluation, Proposed Residential Construction, 312 Neptune Avenue, Encinitas, San Diego County, California 92024, Assessor's Parcel Number (APN) 256-352-04-00

Dear Mr. Bevand:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) has performed a preliminary geotechnical evaluation of the subject site. The purpose of our study was to evaluate the onsite geologic, geomorphic, and geotechnical conditions relative to the proposed residential construction in order to provide preliminary geotechnical recommendations for site earthwork and the design of foundations, slab-on-grade floors, retaining walls, flatwork, and other improvements applicable to the project.

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 property appears to be feasible from a geotechnical perspective, provided the recommendations presented in the text of this report are properly incorporated into the design and construction of the project. The most significant elements of this study are summarized below:

- Based on a review of BGI Architecture ([BGI] 2021), GSI understands that proposed development at the site consists of razing the existing residential structure, and constructing a two-and three-story/split level residence, with typical footings, and slabs on grade, utilizing wood-frame construction, with a stucco exterior. The proposed garage is attached, and a balcony is also proposed on the second floor overlooking the ocean. Building loads are assumed to by typical for this type of relatively light structure. Sewage disposal is proposed to be accommodated by tying into the regional municipal system.

- The distance from the top of the bluff/bluff top to the calculated factor-of-safety (FOS) of $\geq$ 1.5 is $\pm$ 24 feet. The amount of erosion over 75 years is calculated to range from about 7 to 9.4 feet, even with Sea Level Rise (SLR). Our slope stability analyses and bluff retreat evaluation indicate that the minimum 40-foot development setback from the coastal bluff edge, prescribed by the City of Encinitas, is appropriate for the proposed development.

- Coastal Land Solutions (CLSI), performed a photogrammetric analysis to evaluate the site specific historical erosion rate (CLSI, 2020), over the period 1932 to 2020 (88 years). A bluff edge survey completed the end point for the year 2020. Their "Top of Bluff Exhibit" shows that the bluff edge is essentially unchanged from 1932, as the various ages bluff top lines generally coincide within a few feet. In fact, the 1932 bluff top is inboard of all other calculated years (artifically prograding, not retreating). If we use the minimum and maximum prograding distances of 2 and 4.3 feet, respectively, in the southwest corner, and for conservatism, add the maximum error discussed (CLSI, 2020) of 3.7 feet, this yields 5.7 and 8.0 feet, respectively, in 88 years. This is equivalent to a site specific historic rate of 0.065 (low) to 0.091 (high) feet/year, and corresponds to a conservative buff retreat rate ranging from about 4.9 to 6.8 feet over 75 years (not coincidently similar to future SLR). This site specific data is considered the best available science in this regard, and demonstrates that there has been little, and insignificant bluff edge retreat in 88 years.

- To account for the possible added effects from Sea Level Rise (SLR) over the design life of the project (75 yrs), GSI has reasonably assumed that the rate of Bluff Retreat over the next 37 years (2021-2057), should be similar to the past, for several reasons: 1) as sea level rises, the cemented bedrock portion of the bluff is occasionally impacted by waves, as it is now, and should have very little effect on Bluff Retreat (see Plate 1); and 2) the plots of SLR approach asymptotic near the end of the 75-year design life/year 2100, and are much more linear toward the beginning of the design life. Not withstanding, for conservatism, GSI has reasonably assumed SLR will follow today's trends for the period of 2021-2057, and will increase the bluff retreat rate by ⅓ of the 2096 bluff retreat during the period 2058-2082, although the premises discussed above will still largely allow the retreat rate to remain unaffected in reality. During the postulated asymptotic SLR end of the 75-year design life (2083-2096), GSI has assumed that the bluff retreat rate will be that of the year 2096, even though only the cemented bedrock would be impacted by SLR (see Plate 2), as it is now. These are equivalent to bluff retreat rates ranging from 0.065-0.091 ft/yr from 2021-2057, 0.094-0.131 ft/yr for 2058-2082, and 0.151-0.211 ft/yr for 2083-2096, derived from site specific historical retreat rates (considered the best available science), being influenced by postulated SLR. The rates are discussed further herein. Furthermore, an analysis of SLR over the last 11.33 years shows that the CCC mandated guidance document for SLR in the year 2100 predicts a value that is an order of magnitude higher than current SLR tracking validates. Regardless of the range of historic site specific bluff retreat rates utilized in calculations for future retreat rates, when calculated future bluff retreat rates are added to the static FOS = 1.5 distance from the bluff edge, these distances are less than the prescribed 40-feet City of Encinitas bluff setback.

- In general, much of the site is mantled by undocumented artificial fill associated with the existing development. The undocumented fill includes backfill materials for the existing basement and site retaining/planter walls. Quaternary-age colluvium (topsoil) may be present at the surface in less disturbed areas of the property, near its westerly margin. These surficial earth units are underlain by Quaternary-age old paralic deposits (formerly termed “Terrace Deposits”), which in turn, are underlain by sedimentary bedrock belonging to the Tertiary Torrey Sandstone Formation. Transient beach deposits occur offsite, along the shoreline. These deposits are also underlain by the Torrey Sandstone Formation.

- Earth materials considered unsuitable for the support of proposed settlement-sensitive improvements (i.e., any new footings, new slab-on-grade floors, underground utilities, hardscape, etc.) and/or planned engineered fills include all undocumented artificial fill (including existing wall backfill), colluvium (if present within the area of proposed development), and weathered portions of the Quaternary-age old paralic deposits. Unweathered old paralic deposits are considered suitable for the support of settlement-sensitive improvements and/or planned fills in their existing state. New planned fills are not anticipated, based on our understanding of the proposed development. The thickness of unsuitable earth materials, encountered during our subsurface exploration, ranged from approximately ¾ foot to more than 6⅓ feet. Based on our understanding of the existing site development, undocumented fills may be 10 feet or greater in thickness along the northerly, southerly, and westerly sides of the existing basement. In order to reduce the potential for damaging settlements, all unsuitable earth materials located within the influence of the proposed improvements should be removed to expose suitable, unweathered old paralic deposits and then be reused for compacted fills, or replaced with a 2-sack sand-cement slurry, as recommended herein. A 2-sack slurry may also be used to backfill remedial excavations. <u>If slurry backfill is used, the Project Structural Engineer should evaluate the effects of uncured, fluid slurry on the existing basement walls, prior to placement. In addition, the Project Landscape Architect should specify the hold-down depth for slurry backfill relative to finish grade to allow for planting</u>. Shoring and/or alternating slot excavations may be necessary to support the existing offsite property during remedial grading (i.e., during the removal and recompaction of unsuitable soils). Where new footings will be located within the influence of unsuitable soils, deepened footings and/or cast-in-drilled-hole (CIDH) piles may be used. Recommendations for the design and construction of CIDH piles can be provided upon request. Due to limited working space within the site, remedial grading will be challenging, possibly requiring shoring.

- Laboratory testing, performed on a representative sample of the near-surface onsite soils, indicates an expansion index (E.I.) less than 5. This corresponds to very low expansion potential. Thus, the tested onsite soils do not meet the criteria for expansive soils, as indicated in Section 1803.5.3 of the 2019 California Building Code ([2019 CBC], California Building Standards Commission [CBSC], 2019). On a preliminary basis, specific structural design or earthwork mitigation to reduce damaging shrink/swell effects of expansive soil is not warranted.

- Soil pH, saturated resistivity, and soluble sulfate, and chloride testing, performed on a representative sample of the near-surface onsite soils, indicates that the sample is mildly alkaline with respect to soil acidity/alkalinity; is moderately corrosive to exposed, buried metals when saturated; presents negligible sulfate exposure to concrete (Exposure Classes S0, C1, and W0 per Table 19.3.1.1 of American Concrete Institute [ACI] 318-14); and contains relatively low concentrations of soluble chlorides. GSI does not consult in corrosion engineering. Therefore, additional comments and recommendations may be obtained from a qualified corrosion engineer based on the level of corrosion protection desired or required for the project, as determined by the Project Architect, Civil Engineer, and/or Structural Engineer. Owing to the site's proximity to the Pacific Ocean, the corrosive effects of sea spray should be considered in the design and construction of the proposed development.

- On a preliminary basis, temporary excavations greater than 4 feet, but less than 20 feet in overall height, completed into existing undocumented fills, colluvium, and weathered and unweathered old paralic deposits should conform to CAL-OSHA and/or OSHA requirements for Type "C" soil conditions (i.e., a 1.5:1 [horizontal:vertical {h:v}] slope gradient) provided that perched groundwater, running sands, and/or other adverse conditions are <u>not</u> present. All temporary excavations should be observed by a licensed engineering geologist or geotechnical engineer prior to entry by an unprotected worker. If the recommended temporary slope construction conflicts with property boundaries or existing improvements that need to remain in service, shoring or alternating slot excavations may be necessary. The need for shoring or alternating slot excavations should be further evaluated during the preparation of project construction documents.

- The regional groundwater table is considered nearly coincident with sea level or approximately 73 feet below the lowest existing site elevation. GSI did not encounter groundwater nor evidence of perched groundwater in our subsurface explorations nor did we observe groundwater exiting the bluff face during our site-specific field evaluation. A review of oblique aerial photographs available on the California Coastal Records website (https://www.californiacoastline.org), did reveal infrequent evidence of groundwater exiting the bluff face in the past. In addition, during field work GSI performed at nearby 330 Neptune Avenue, approximately 21 years ago (GSI, 2000), we observed perched groundwater seeping from the bluff face, along the geologic contact between the old paralic deposits and the Torrey Sandstone. We also encountered perched groundwater seepage at an approximate depth of 60 feet below the existing grade within a boring advanced at this nearby property. This roughly equates to an elevation of 15 feet above sea level. Based on its depth, groundwater is not anticipated to significantly affect the proposed site development. However, perched groundwater conditions may manifest either prior to, or during site development, and would typically occur along boundaries of contrasting permeability and density (i.e., fill/old paralic deposits contacts, old paralic deposits/Torrey Sandstone contacts, fill lifts, etc.), and along geologic discontinuities (i.e., fractures, joints, etc.). The potential for perched groundwater to manifest in the future should be anticipated and disclosed to all interested/affected parties. The possible origins of perched groundwater may include, but not limited to: site rainfall severity and frequency, onsite and offsite irrigation practices, broken or damaged wet underground utilities, etc. Owing to its past occurrence on a nearby property, perched groundwater was considered in the slope stability analyses conducted as part of this study.

- Our evaluation indicates there are no known Holocene-active faults crossing the site. Thus, the potential for surface fault rupture to affect the existing and proposed site development is considered very low. However, due to its location within a seismically active region, the site could experience moderate to strong ground shaking over the life of the development.

- Owing to the depth to regional groundwater below the existing site elevations and the dense nature of the old paralic deposits, and the Torrey Sandstone, the potential for the site to be adversely affected by liquefaction/lateral spreading is considered very low.

- The seismic acceleration values and design parameters provided herein should be considered during the design of the proposed development. The adverse effects of seismic shaking on the proposed residence will likely be wall cracks, some foundation/slab distress, and some seismic settlement.

- Due to its elevated position, the proposed development is at low risk for tsunami inundation. However, the coastal bluff descending from the site is located within a tsunami inundation zone, and could experience some erosion from a tsunami impact. The effects from a tsunami would be generally similar to those created by storm waves.

- Infiltration for permanent stormwater treatment and disposal is not recommended, as it could negatively affect bluff stability.

- Surface drainage should be directed away from the bluff edge and toward Neptune Avenue, and should be properly maintained over the life of the development. Allowing surface runoff to flow over the bluff edge could result in erosion and surficial bluff failures, leading to increased rates of upper bluff retreat.

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

- Provided that the recommendations included in this report are properly incorporated during project planning, design, and construction, the proposed development will not directly or indirectly cause, promote, or encourage bluff erosion or failure, either on the site or the adjacent properties. Encinitas Municipal Code (EMC) 30.34.020C.b(iii). In addition, the proposed project will not restrict or reduce public access or beach use. EMC 30.34.020C.b(v).

- Provided our recommendations are properly implemented, based on the estimated long-term erosion rates reported herein, the proposed development will be safe from bluff failure and erosion over its lifetime, without having to propose any additional bluff stabilization to protect the structure in the future (EMC 30.34.020D), even with a rise in sea level. This assumes regular and periodic maintenance of the property, and prudent control of surface runoff water. Regular and periodic maintenance of the existing seawall will also help in this regard.

- The recommendations presented in this report should be incorporated into the design and construction considerations of the project.

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

David W. Skelly
Civil Engineer, RCE 47857

JPF/DWS/sh

Distribution: (3) Addressee (2 wet signed)

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ATTACHMENTS:

PRELIMINARY GEOTECHNICAL EVALUATION
PROPOSED RESIDENTIAL CONSTRUCTION, 312 NEPTUNE AVENUE
ENCINITAS, SAN DIEGO COUNTY, CALIFORNIA 92024
ASSESSOR'S PARCEL NUMBER (APN) 256-352-04-00

SCOPE OF SERVICES

The scope of our services has included the following:

1. Review of in-house geologic literature, regional geologic maps, aerial photographs of the site and near vicinity, and the existing preliminary geotechnical evaluation report for nearby 330 Neptune Avenue property (see Appendix A).

2. Delineating the coastal bluff edge in the field.

3. Geologic site reconnaissance, mapping, and subsurface exploration with one (1) exploratory test pit, and two (2) hand-auger borings to evaluate the near-surface soil and geologic conditions, and to sample the onsite earth materials (see Appendix B). The GSI (2000) boring log for nearby 330 Neptune Avenue is also provided in Appendix B.

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

5. Appropriate laboratory testing of representative soil samples and a review of the laboratory testing, performed in preparation GSI (2000), for nearby 330 Neptune. The laboratory test results are provided in Appendix D.

6. Engineering and geologic analysis of data collected, including analyses of the stability of the coastal bluff (Appendix E).

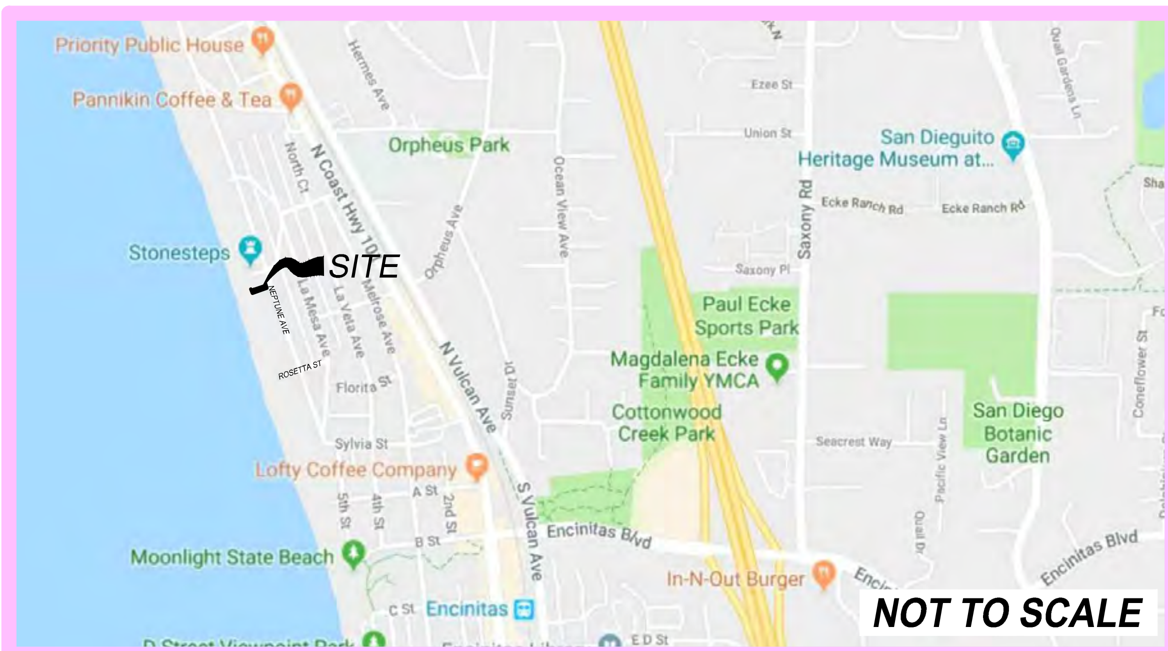
7. Preparation of this summary report and accompaniments.

SITE DESCRIPTION AND PROPOSED DEVELOPMENT

Site Description

The site consists of a bluff top, quadrilateral-shaped, developed residential property located at 312 Neptune Avenue in the City of Encinitas, San Diego, California 92024 (see Figure 1, Site Location Map). The latitude and longitude of the approximate centroid of the subject site are 33.0545, -117.3005. The property is situated above an approximately 75- to 76-foot high coastal bluff, descending toward the Pacific Ocean. The property is bounded by Neptune Avenue to the east, by the aforementioned coastal bluff and Pacific Ocean to the west, and by existing residential development to the remaining quadrants. According to the 5-scale topographic survey provided by Rancho Coastal Engineering and Surveying ([RCE&S],2019b), the upper site is relatively flat-lying to gently

Base Map: TOPO!® ©2003 National Geographic, U.S.G.S Encinitas Quadrangle, California -- San Diego Co., 7.5 Minute, dated 1997, current, 1999.



Base Map: Google Maps, Copyright 2019 Google, Map Data Copyright 2019 Google

GeoSoils, Inc.

W.O. **7638-A-SC**

SITE LOCATION MAP

Figure 1

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sloping in a southeasterly direction at gradients on the order of 2:1 (h:v) or flatter. RCE&S (2019b) shows that elevations vary between approximately 73 and 86 feet (North American Vertical Datum of 1988 [NAVD88]), for an overall relief of about 13 feet. The sea cliff descends to the Pacific Ocean at inclinations ranging from about 2:1 (H:V) to near vertical. Surface drainage is generally accommodated by sheet flow runoff, which is locally directed toward drop inlets connected into area drains. Ultimately, site surface drainage is directed toward Neptune Avenue.

Existing site improvements include a split-level single-family residence with a partial subterranean floor level and associated brick and concrete flatwork, and site/planter walls. The maximum height of the existing site/planter walls is approximately 5½ feet. Vegetation generally consists of sparse, drought-tolerant vegetation with localized palm trees, and shrubbery in planters, located near the easterly property margin.

Proposed Development

Based on a review of BGI Architecture ([BGI] 2021), GSI understands that proposed development at the site consists of razing the existing residential structure, and constructing a two-and three-story/split level residence, with typical footings, and slabs on grade, utilizing wood-frame construction, with a stucco exterior. The proposed garage is attached, and a balcony is also proposed on the second floor overlooking the ocean. Building loads are assumed to by typical for this type of relatively light structure. Sewage disposal is proposed to be accommodated by tying into the regional municipal system.

NEARBY GEOTECHNICAL STUDY

As previously mentioned, GSI performed a preliminary geotechnical evaluation of 330 Neptune Avenue in 2000 (second property to the north of the subject site). That evaluation included geologic mapping of the exposed conditions of the coastal bluff, including bedding and joint/fracture attitudes; subsurface exploration by advancing one (1) exploratory hollow-stem auger boring in a former driveway, near the northerly property margin, to an approximate depth of 81½ feet below the existing grade (b.e.g.) and one (1) hand-auger boring in the former rear-yard to an approximate depth of 4 feet b.e.g. to determine the soil/bedrock profiles, obtain relatively undisturbed and bulk samples of representative materials, and delineate earth material parameters that may affect the stability of the existing bluff and the proposed development; and laboratory testing of representative soil samples collected during our subsurface exploration program. The GSI (2000) hollow-stem auger boring log and laboratory shear strengths were used for this current study to model the geologic conditions within the coastal bluff for our slope stability analyses. The GSI (2000) hollow-stem auger boring log is provided in Appendix B. The GSI (2000) shear strengths are repeated in the “Laboratory Testing” section and in Appendix E.

SITE EXPLORATION

On June 13, 2019, a GSI representative met with Mr. Jim Knowlton (City of Encinitas Geotechnical Reviewer). The purpose of this meeting was to show Mr. Knowlton our demarcated position of the coastal bluff adjacent to the subject property. At the conclusion of this meeting both GSI and Mr. Knowlton were in mutual agreement regarding the bluff edge location. RCE&S then surveyed our delineated bluff edge location and included it on their topographic survey maps (RCE&S; 2019a and 2019b).

In early October 2019, GSI returned to the subject site to conduct geologic mapping of the coastal bluff and to perform shallow subsurface explorations within the property. Near-surface soil and geologic conditions were explored with one (1) exploratory test pit, excavated with hand equipment and two (2) hand-auger borings. The approximate locations of the subsurface explorations are shown on the Geotechnical Map (see Plate 1), which has been adopted from RCE&S (2019a; 2019b). A geologic cross-section (X-X’) depicting the subsurface conditions in profile is provided as Plate 2. Logs of the test pit and borings, including the GSI (2000) hollow-stem auger boring advanced at nearby 330 Neptune Avenue, are presented in Appendix B.

COASTAL BLUFF GEOMORPHOLOGY

The typical coastal-bluff profile may be divided into three zones: the shore platform; a lower near-vertical cliff surface, termed the seacliff; and an upper-bluff slope generally ranging in inclination between about 20 and 80 degrees (measured from the horizontal plane). The bluff top or bluff edge is the boundary between the upper bluff and the relatively flat lying to gently sloping coastal terrace.

The coastal bluff, directly adjacent to the subject site is generally consistent with the previously described typical bluff profile. However, portions of the seacliff, directly below the subject site, are concealed by an existing seawall. Based on our review of the as-built plans prepared by Soil Engineering Construction, Inc. ([SEC], 1995), the seawall was completed in December 1995, in conjunction with Coastal Development Permit No. 6-93-85. According to SEC (1995), the seawall was installed between elevations ±-5 feet and ±+13 feet Mean Sea Level (MSL), although RCE&S (2019a) shows the elevation of the top of the seawall at approximately 15 feet (NAVD88). This difference between the top of seawall elevations shown on SEC (1995) and RCE&S (2019a) is attributed to the use of different survey datums. RCE&S (2019a) shows that upper-bluff inclinations vary between approximately 40 and 60 degrees from the horizontal plane.

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). In the

Encinitas and Solana Beach areas, the shore platform extends 500 to 900 feet offshore at a 1 to 2 percent grade (United States Army Corps of Engineers [USACE], 2012). 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. In this case, a shoreline with a bluff, the foreshore zone extends from low water to the lower face of the bluff.

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 Encinitas coastline as “active” and “Type C(c).” The letter “C” designates coastal bluffs having a resistant geologic formation along the base of the bluff and more erodible earth materials in the upper portions of the bluff. The relative effectiveness of marine erosion compared to subaerial erosion of the bluff produces a characteristic profile of a near-vertical sea cliff with a more gently sloping, albeit steep, upper-bluff slope. The letter “(c)” indicates that the long-term rate of marine erosion is approximately equal to that of subaerial erosion.

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 began 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, during 13 ice ages, there were glaciations lasting typically 90,000 years and 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 causes 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 on Figure 2. As can be seen on Figure 2, each of the last inter-glacial warming periods (as we are in today), was

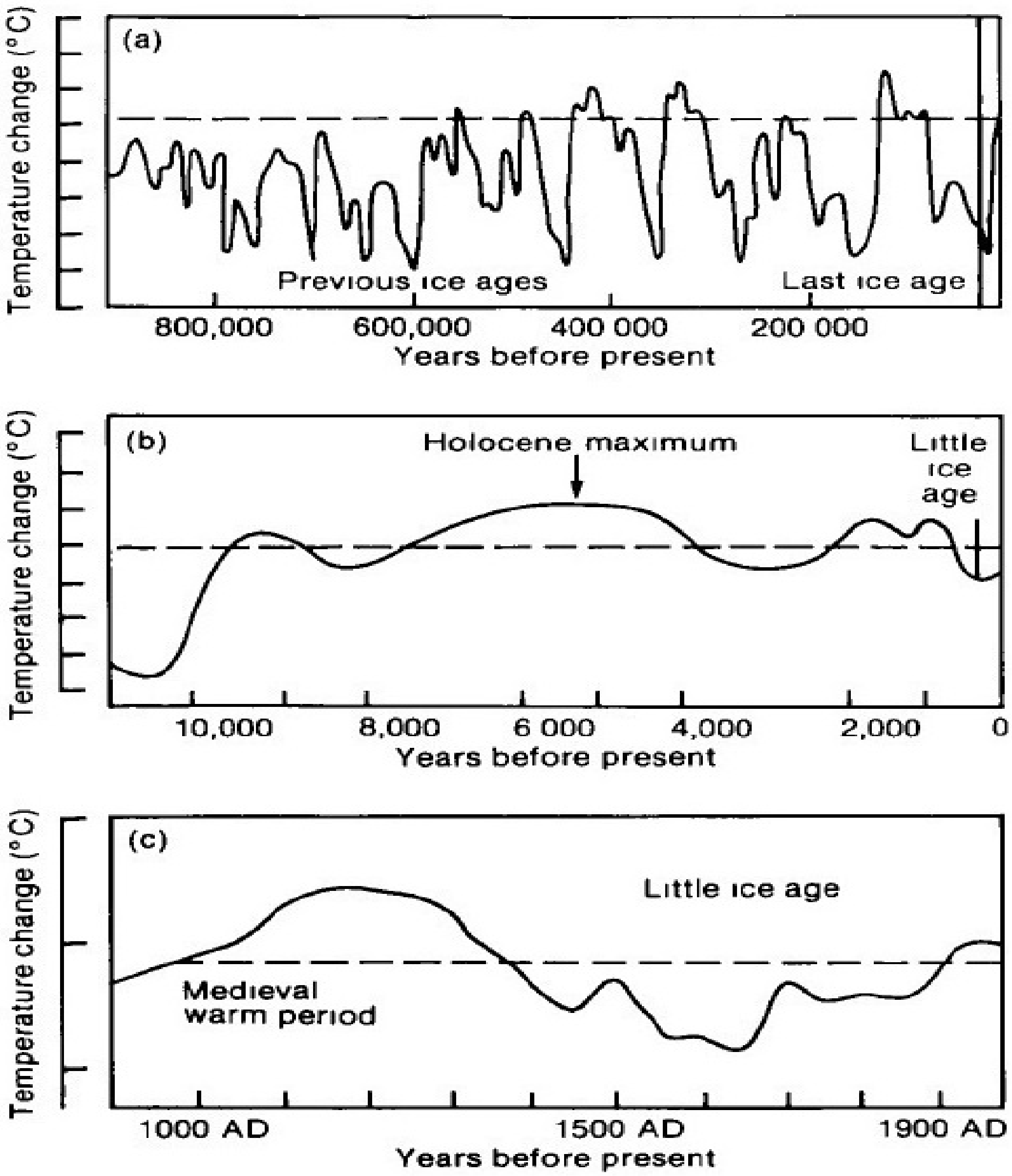


Figure 2 (from Figure 7.1 [IPCC, 1990]): Schematic diagrams of global temperature variations since the Pleistocene on three time scales (a) the last million years (b) the last ten thousand years and (c) the last thousand years. The dotted line nominally represents conditions near the beginning of the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

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 ±350 feet below the present to possibly as high as about ±31 feet above.

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).

Sea level changes during the last ±20,000 years 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). Following the peak of the Last Glacial Maximum (LGM) about 18,000 to 20,000 years ago, as indicated on Figure 2, Earth entered the present inter-glacial 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 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 (Alley, 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 mm/yr (1.97 in/yr) and 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 about 1 to 7.2 meters (±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 since that time, but has never remained static for long periods. During the past 3,000 to 2,000 years, the rate appears to have fluctuated and haltingly slowed 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 over the past 20 years, SLR has increased to about 3.1 mm/yr (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 along California rose approximately ±0.6 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 inter-glacial 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 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

There is currently a wide range of predicted rates in sea level rise (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 2100-2150 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. 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 then 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 USACE Coastal Engineering Manual (CEM) provides an extensive discussion of water levels used for design. A summary of the CEM conclusions with regard to 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 east coast, and 11 cm/century on the west coast, and a range along the 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 west US coast sea level for 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 (for example, 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, 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:

<u>Sea-Level Change Curve Calculator</u>

The sea-level change curve calculator (see Figure 3, below) provides a way to visualize the USACE and other authoritative sea level rise scenarios for any tide gauge that is part of the NOAA National Water Level Observation Network (NWLON). Scenarios include those of the The West Coast National Research Council 2012 study, the New York State Department of Environmental Conservation 6 NYCRR Part 490 projections for New York City and Long Island (available when the NOAA gauge, "The Battery" or "Montauk Point" is selected), the New York City Panel on Climate Change 2013 and 2015 projections for The Battery (8518750), the Maryland Climate Change Commission 2013 Projections (available when selecting a gauge in Maryland), the CARSWG scenarios for developed for the 2016 CARSWG report, and the 2017 US Global Change Research Program scenarios.

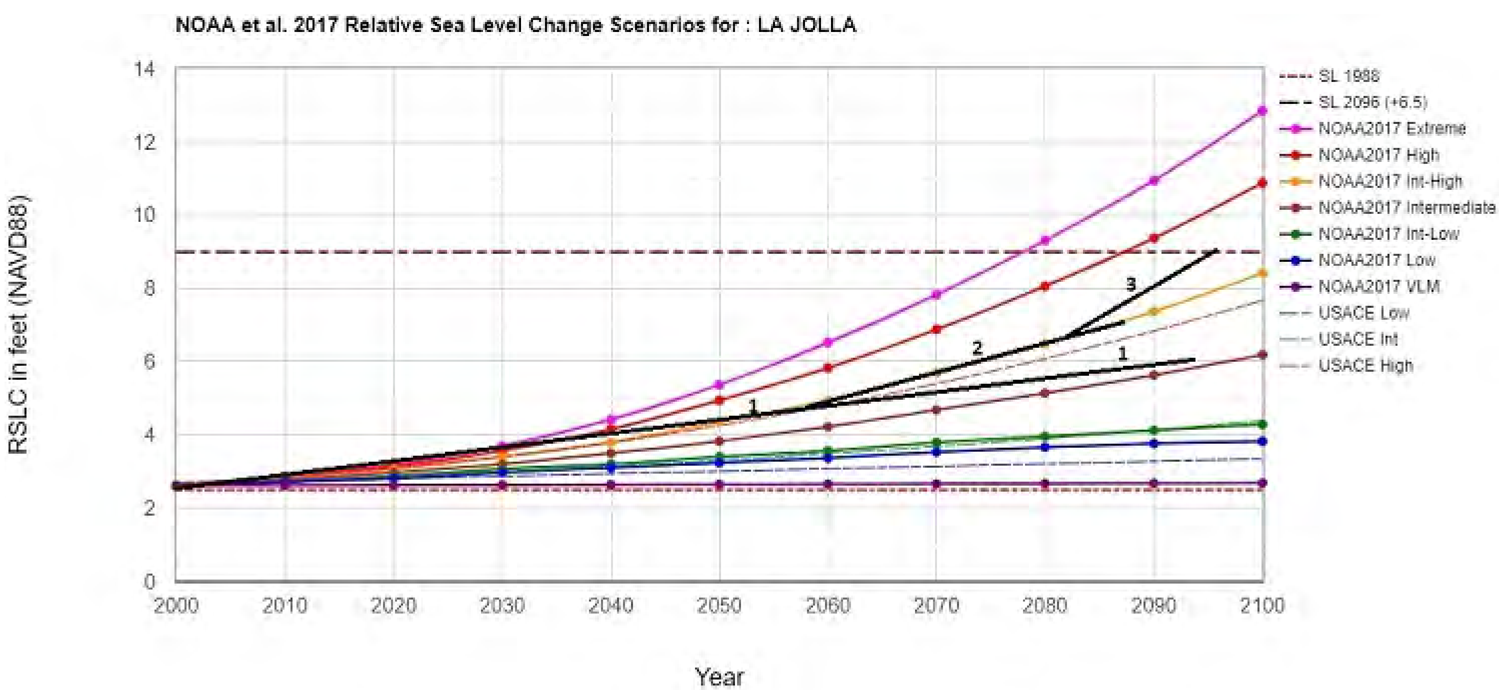


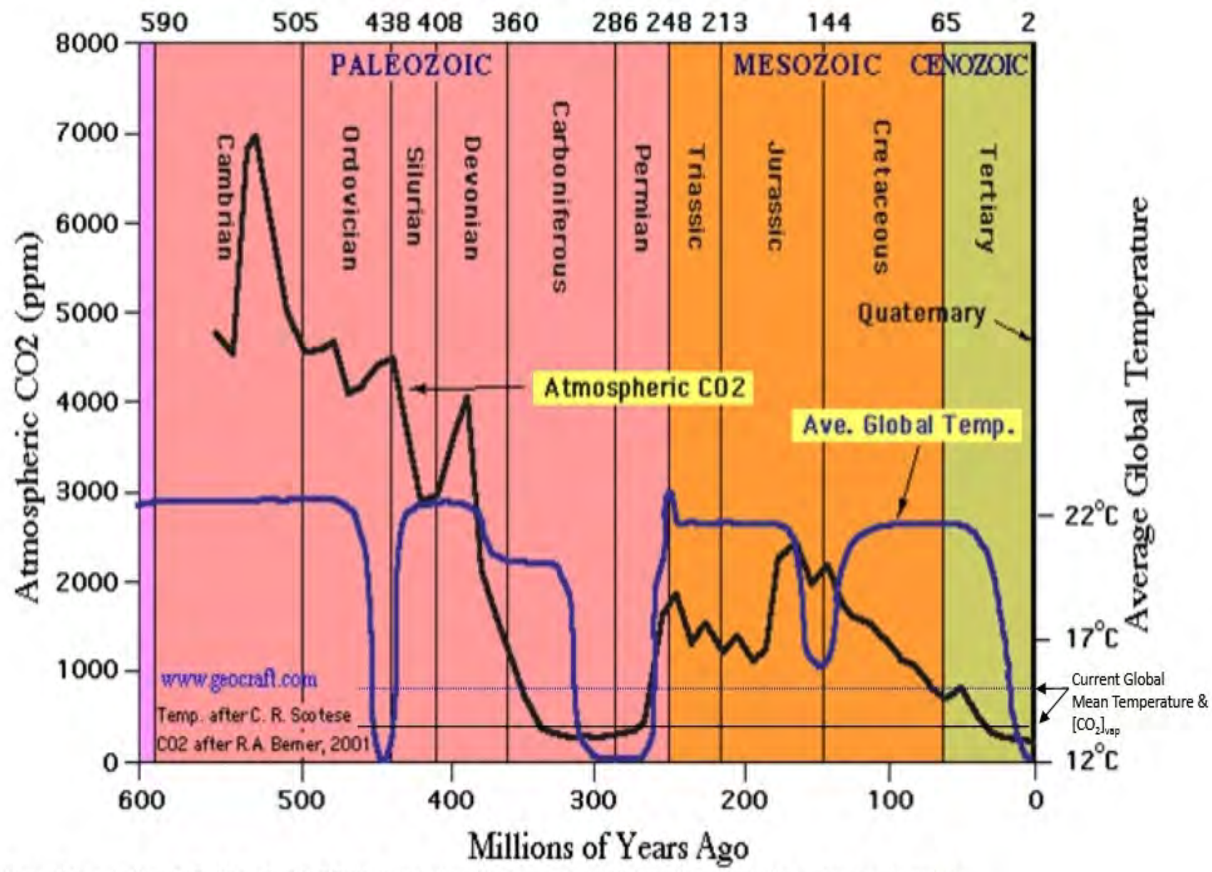
Figure 3 - Sea-Level Change Curve

The SLR curve developed above was generated from data derived from Gauge:9410230, La Jolla, California, using the Sea-level Change Curve Calculator. 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 2021 to the year 2057; from 2058 to 2082, and from 2083 to 2096 (the end of the design life). These three linear portions are discussed further, later in the text.

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. The most current EPA global sea level rise prediction is available on their website. The EPA approximate range for global sea level rise in 2100 is 0.2 meters (0.6 feet) to 2.0 meters (6.56 feet) above present sea level (NOAA, 2017).

Recently adopted guidelines by the California Coastal Commission (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. In contrast, Scripps Institution of Oceanography indicates current SLR as tracking along the lower intermediate to low curve. 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 (±285-405 ppm), as indicated on Figure 4, below. The predicted large rise in sea level comes from computer climate models predicated on greenhouse gas emissions (primarily <u>CO_2</u>, which comprises a <u>mere</u> <u>±6 percent of all greenhouse gases</u>) causing global temperature to rise (rather than the other way around), regardless of the dubious correlation of that relationship during geologic time (see Figure 4). 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 in 2100 of 7.1 feet above current sea level. Extrapolating for a 75-year design life, this is equivalent to about 6.5 feet above current sea level at La Jolla (closest available projection in CCC [2018]).

Due to the relatively high elevation of the proposed development (approximately ±76 to ±86 feet NAVD88), it is considered reasonably safe from coastal hazards including shoreline erosion, wave attack, wave overtopping and coastal flooding, even with a conservative sea level rise of 6.5 feet over the 75-year life of the development (0.5% probability that SLR exceeds this amount, per CCC[2018]), from 2021 to 2096 (year 2096 extrapolated from 2090 to 2100 data). The existing seawall below the subject site would provide additional protection from coastal hazards.



Berner RA and Kothavala Z. 2001. GEOCARB III: A revised model of atmospheric CO2 over Phanerozoic time. *American Journal of Science.* 301: 182-204

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

HISTORIC COASTAL-BLUFF RETREAT

Most of San Diego County's coastline has experienced a measurable amount of erosion in the last 75 years, with more rapid erosion occurring during periods of heavy storm surf (Kuhn and Shepard, 1984). The entire base of the seacliff portion of coastal bluffs is based upon the available information, the use of a 6.4-foot rise in sea level over the design life of the proposed improvements to the property is fairly conservative (0.5% probability that SLR exceeds this amount [CCC, 2018]). Due to the relatively high elevation of the existing and proposed development (approximately ±76 to ±86 feet NAVD88), it is considered reasonably safe from coastal hazards including shoreline erosion, wave attack, wave overtopping and coastal flooding, even with a conservative sea level rise of 6.4 feet over the 75-year life of the development (0.5% probability that SLR exceeds this amount, per CCC[2018]), from 2019 to 2094 (year 2094 extrapolated from 2090 to 2100 data). The existing seawall below the subject site would provide additional protection from coastal hazards. exposed to direct wave attack along most of the coast. The waves erode the seacliff by impact on small joints/fractures and fissures in the otherwise essentially massive 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 of the friable sands. Where these processes are active, unraveling of cohesionless sands has occurred along portions of the upper bluffs. Finally, improvements sited near the bluff edge can concentrate surface runoff onto the bluff slope, and can contribute to subaerial erosion and bluff instability.

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 at the site 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. Below the subject site, there is an existing seawall that affords some protection of the seacliff from the mechanical and biological processes contributing to seacliff erosion. Immediately to the north and south of the seawall, there is evidence of backwearing near the toe of the bluff (i.e., basal notching).

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, which 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 upon the formational materials at the site is spring sapping, or the mechanical erosion of sand grains by water exiting the bluff face. Chemical solution; however, is also a significant contributor (especially of carbonate matrix material). As indicated previously, as groundwater approaches the bluff, it infiltrates near-surface, stress-relief, bluff-parallel joints/fractures, which form naturally behind and parallel to the bluff face. Hydrostatic loading of bluff parallel (and sub-parallel) joints/fractures is an important cause of block-toppling on steep-cliffed lower bluffs, especially where the basal notching is present (Kuhn and Shepard, 1980). During our field work, GSI did not observe evidence of groundwater seepage near the toe of the bluff, nor at the overlying old paralic deposits/Torrey Sandstone contact. However, oblique aerial photographs taken in the early to middle part of last decade (www.californiacoastline.org) indicates possible evidence of groundwater seepage exiting the bluff face north and south of the subject site at approximate elevation 15 feet. In addition, GSI encountered perched groundwater along the geologic contract between the old paralic deposits and the Torrey Sandstone and approximately 15 feet above sea level in a boring advanced during field work performed at nearby 330 Neptune Avenue in 2000 (GSI, 2000). These groundwater conditions were modeled in our slope stability analyses.

Slope Decline

The process of slope decline 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 niches and/or sea caves, and bluff-parallel tension joint/fractures. 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. In the region, upper bluffs with slope angles in the range of 35 to 40 degrees may indicate ages of 65- to 100-years (Wallace, 1977). Steeper slopes indicate a younger age.

According to RCE&S (2019), upper-bluff slope angles below the site range between approximately 40 and 60 degrees (upper bluff [old paralic deposits]), suggesting an age of 65 years or less. Whereas, the seacliff portion of the adjacent coastal bluff (lower bluff [Torrey Sandstone]) consist of a near- vertical slope, indicating a relatively young age at the base of the bluff (i.e., 5 to 40 years), which is generally typical of active erosion.

Surface Drainage

Uncontrolled concentrated surface drainage can result in significant upper-bluff erosion. Improvements, such as patios and other hardscape, located at, or adjacent to, the bluff top can create water paths that concentrate surface water runoff toward the bluff edge. In addition, drains, gutters, and downspouts can often become clogged with vegetation during torrential rains which results in concentrated uncontrolled surface water runoff over the bluff. These “top down” type bluff failures are characterized by small “V” shaped erosion gullies, a few feet across, that extend down the bluff face but terminate above the wave runup line. These features are present along the upper-bluff slope adjacent to the subject site.

Historic Coastal Bluff Retreat Summary

Numerous studies have been undertaken to analyze coastal bluff retreat along the Encinitas 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 joint spacing and width; earth material weathering and fatigue; groundwater seepage; and wave impact at the seacliff. These data were compared to the bluff edge locations observed in soft-copy photogrammetric images of the coast for the years 1932, 1949, 1952, 1956, and 1994 as well as more recent bluff edge locations surveyed with global positioning instruments.

For the Encinitas coast section, which reportedly was located between 507 “A” Street and 410 Neptune Avenue and includes the subject site, Benumof and Griggs (1999) arrived at a mean recession rate of 6 cm/yr (2.36 in/yr), or approximately 0.20 ft/yr, with a standard deviation of 2.31 cm/yr (0.91 in/yr or 0.076 ft/yr). Benumof and Griggs (1999) also provided alongshore plots quantifying the amount of coastal erosion within their Encinitas study area. Their findings indicated an approximate retreat rate of 10 cm/yr (3.94 inches/yr) or approximately 0.328 ft/yr for this reach , which is an order of magnitude higher than the site specific historical retreat rate calculated (see below), and in contrast to the upper bound of bluff retreat rate for the entire Encinitas reach of 14 cm/ (5.5 in/yr), or 0.46 ft/yr, provided by Benumof and Griggs (1999). We note that the aerial photographs reviewed (see below, Appendix A, and Coastal Land Solutions, Inc. [2020]), did <u>not</u> support the aggressive Benumoff and Griggs retreat rates.

Coastal Land Solutions (CLSI), performed a photogrammetric analysis to evaluate the site specific historical erosion rate (CLSI, 2020), over the period 1932 to 2020 (88 years). A bluff edge survey completed the end point for the year 2020. Their "Top of Bluff Exhibit" shows that the bluff edge is essentially unchanged from 1932, as the various ages bluff top lines generally coincide within a few feet. In fact, the 1932 bluff top is inboard of all other calculated years (artifically prograding, not retreating). If we use the minimum and maximum prograding distances of 2 and 4.3 feet, respectively, in the southwest corner, and for conservatism, add the maximum error discussed (CLSI, 2020) of 3.7 feet, this yields 5.7 and 8.0 feet, respectively, in 88 years. This is equivalent to a site specific historic rate of 0.065 (low) to 0.091(high) feet/year, and corresponds to a conservative buff retreat rate ranging from about 4.9 to 6.8 feet over 75 years. This site specific data is considered the best available science in this regard, and demonstrates that there has been little, and insignificant bluff edge retreat in 88 years.

FUTURE LONG-TERM BLUFF RETREAT RATE

CoSMoS 3.0 Computer Application

Until recently, the CCC was utilizing the online application CoSMos 3.0 computer application to bolster their claims of increased bluff erosion under SLR. The United States Geological Survey (USGS) developed the CoSMoS computer application (Barnard, et. al., 2014) to predict coastal flooding, and was modeled assuming soft cliffs with unconsolidated sediments (unlike the subject site). The USGS then expanded upon the computer models therein to include shoreline evolution using data from the Hapke and Reid (2007) and Hapke, et al. (2006) studies, which assess historical retreat of cliffed and sandy shorelines along the California coast, respectively. GSI points out that the CoSMoS website contains a disclaimer stating,

> ***This interactive mapping tool, including its data and other information ('tool and data') are provided for informational purposes. The tool and data are not for the purpose of providing advice or guidance on issues or activities related to its content including, but not limited to, navigation, investment, development or permitting.***

The broad spatiality and the inherent uncertainties of the Hapke and Reid (2007) and Hapke, et al (2006) studies do not afford high resolution interpretations of shoreline retreat at a particular site. Thus, neither of these reports should be used to override comprehensive, detailed site-specific analysis of historical shoreline retreat as methods to predict future retreat. For instance, the alongshore plots of bluff retreat in Hapke and Reid (2007) only provide generalized reference locations for measuring distances along regional stretches of the coastline. Owing to the ambiguities with locating a particular coastal site on these plots, the user is forced to arrive at a range of historical bluff retreat rates for the site, and the end points of this range may either under- or over-predict the historical rate for the coastal bluff adjacent to the subject site. As indicated in Figure 37 of Hapke and Reid (2007), historical bluff retreat near the subject site may range between approximately 0.034 m/yr (0.11 ft/yr) and 0.10 m/yr (0.33 ft/yr).

In addition, the Hapke and Reid (2007) study explicitly reports a retreat rate uncertainty of 0.2 m/yr (0.656 ft/yr). To put this into perspective, this uncertainty is on the same order of magnitude for the upper end of the range of historical retreat Hapke and Reid (2007) concluded for the coastal bluff at the subject site to as much as double the order of magnitude for the lower end of the range.

CoSMoS 3.0 provides detailed (meter-scale) predictions of coastal flooding due to both future sea level rise and storms integrated with long-term coastal evolution (i.e., beach changes and cliff/bluff retreat) for the southern California region, from Point Conception (in Santa Barbara County) to Imperial Beach, California. However, since all of the coastal evolution models rely on the historical rate to predict the future rate, if the historical rate is incorrect, then the future rate intuitively would be erroneous, regardless of the accuracy of the erosion model. It is not clear as to what model the USGS used to predict cliff retreat for a given SLR amount by the year 2100.

Importantly, the use of the CoSMoS model is limited with the following disclaimer:

> ***Inundated areas shown should not be used for navigation, regulatory, permitting, or other legal purposes. The U.S. Geological Survey provides these data "as is" for a quick reference, emergency planning tool but assumes no legal liability or responsibility resulting from the use of this information.***

Any modeling supporting future site-specific bluff retreat rate utilizing CoSMoS is expressly discouraged by the USGS. In addition, we have reviewed third-party electronic communication between Dr. Benjamin Benumof (co-author of the aforementioned study), and Dr. Patrick Limber of the USGS (who co-developed the CoSMoS 3.0 computer simulation). In their correspondence, Dr. Limber states,

> ***The Cosmos cliff projections are large-scale, long-term estimates of cliff behavior -- they project the long-term rate that results from multiple cliff failures accumulating through time, rather than the exact timing of individual cliff failure events. If you're looking at 1) short-term site-specific behavior, as in "how soon is this cliff likely to fail?", or 2) how a site-specific cross-shore cliff profile might evolve through time,*** **Cosmos-cliffs is probably not the right tool and should be supplemented by local, more geotechnically-detailed, investigations.**

Clearly, CoSMos is not the appropriate tool for assigning site-specific rates of future coastal bluff retreat.

While we disagree with its use for site-specific future bluff erosion, for the purpose of illustrating its inappropriateness, we utilized CoSMoS to evaluate future coastal bluff retreat adjacent to the subject site using the CoSMoS cliff retreat modeling. For this assessment, we input 25 cm (or approximately 0.82 ft.) of SLR selected/predicted for the year 2100 in CoSMoS, which nearly models current sea level conditions. The output of the CoSMoS modeling is provided as Figure 5. As shown in Figure 5, the CoSMoS output shows a retreated bluff edge at the subject site. However, the projected bluff edge position at the subject site and the properties to the north and south roughly mimics the trend of the coastline and does not appear to consider the protection afforded by the existing seawall at the subject site nor at the Stone Steps public access staircase. This is because the program is not accurate and explicitly is not intended to be used for site-specific analysis. In addition, one would expect that the use of a very low SLR estimate in the CoSMoS application should produce a future bluff edge position that is nearly identical to its current location without the applied SLR (i.e., the bluff edge location obtained through historical analyses). In fact, careful measurement shows that in the year 2100 CoSMoS predicts about 5½ feet of bluff retreat with 25 cm SLR. This suggests a retreat rate of approximately 0.07 ft/yr between present day and the year 2100 (similar to CLSI, 2020). As previously stated, our site-specific evaluation of long-term bluff retreat by photogrammetry and physical surveys (CLSI, 2020), was estimated to range from 0.065 (low) to 0.091(high) feet/year from 1932 to 2020, similar to the CoSMos modeling of 0.07 ft/yr over the next 81years, with 25 cm (0.82 ft) of SLR. This future retreat rate estimate produced by CoSMoS 3.0 is not reconcilable with the hypothesis that SLR will greatly affect bluff retreat, given that it is essentially the same as the historic rate. This does, however, gives credence to the reasonable conclusion that future SLR should have little, if any, impact on the site in the next 75 years.

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 cemented seacliff portion of the bluff is occasionally impacted by waves, as it is now, and should have very little effect on bluff retreat (see Plate 2); 2) the plots of SLR approach asymptotic near the end of the 75-year design life; and 3) see CoSMos discussion: in contrast, the curves are much more linear toward the beginning of the design life.

Additionally, we point out that 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 hitting the bluff. Under high tides/storm conditions, incoming waves will impact similar bluff materials as they do at present, only at a slightly higher elevation within the bluff profile (see Plate 2).

Figure 5. Our Coast Our Future (OCOF) CoSMoS output showing the extent of regional coastal bluff retreat in the year 2100, assuming 25 cm (0.82 ft) of SLR.

Simplified Numerical Model of Shoreline Evolution

GSI understands that the CCC now observes the simplified numerical models developed by Ashton, et al. (2011) and Young, et al. (2013) as 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.

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.

Sediment Contributions from the Onsite and Nearby Coastal Bluffs

Visual classification as well as our review of sieve analysis tests performed on samples of typical coastal bluff earth materials from nearby sites indicates that the adjacent bluff is mostly comprised of sand with little fines (i.e., silt and clay). This is important in that scientific consensus suggests a direct relationship between SLR and bluff erosion. That being said, it should be expected that more frequent bluff erosion caused by accelerating SLR would contribute more sand, originating from the coastal bluff, onto the adjacent beach, thus, enhancing this protective berm and slowing bluff erosion over time.

Coastal Bluff Lithology

The lithology of the onsite coastal bluff likely provides the greatest dampening effect on marine erosion. As shown on Plate 2, wave attack will still be focused on the more resistant Torrey Sandstone formation rather than the more erodible old paralic deposits over the 75-year design life of the proposed reesidence, even during astronomical high tides. A review of Figures 6(a) and 6(b) in Benumof and Griggs (1999) indicates that the Torrey Sandstone formation within the Encinitas section (which includes the subject site) exhibited the third highest mean Schmidt Hammer rebound values of their studied San Diego County coastal bluffs. Only coastal bluffs comprised of Cretaceous-age sedimentary bedrock in La Jolla and Sunset Cliffs displayed higher rebound values.

Presence of a Protective Beach

The coastal bluff at the subject site is fronted by a shingle beach, which will equilibrate in step with SLR over the 75-year design life of the proposed remodel/addition (Dean,1990). The beach profile will attenuate in-coming wave energy prior to impacting the coastal bluff. Most of the time, the beach is much wider than 20 feet, similar to a conditionally decoupled profile model (CDPM) curve BB:0 (see GSI's Figure 6, below, which is Figure 12 of Young, et al., 2013). 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 6.5 feet of SLR will have occurred). Given the closeness to the BB:0 line, m = 0.333 (or ⅓) appears appropriate for the coastal bluff adjacent to the subject site.

Sediment Contributions from the Onsite and Nearby Coastal Bluffs

Visual classification as well as our review of sieve analysis tests performed on samples of typical coastal bluff earth materials from nearby sites indicates that the adjacent bluff is mostly comprised of sand with little fines (i.e., silt and clay). This is important in that scientific consensus suggests a direct relationship between SLR and bluff erosion. That being said, it should be expected that more frequent bluff erosion caused by accelerating SLR would contribute more sand, originating from the coastal bluff, onto the adjacent beach, thus, enhancing this protective berm and slowing bluff erosion over time.

GSI has evaluated the long-term erosion rate of the coastal bluff at the subject site in light of sea level rise using the simplified numerical model equation described above. The values assigned to the site-specific model equation are summarized below.

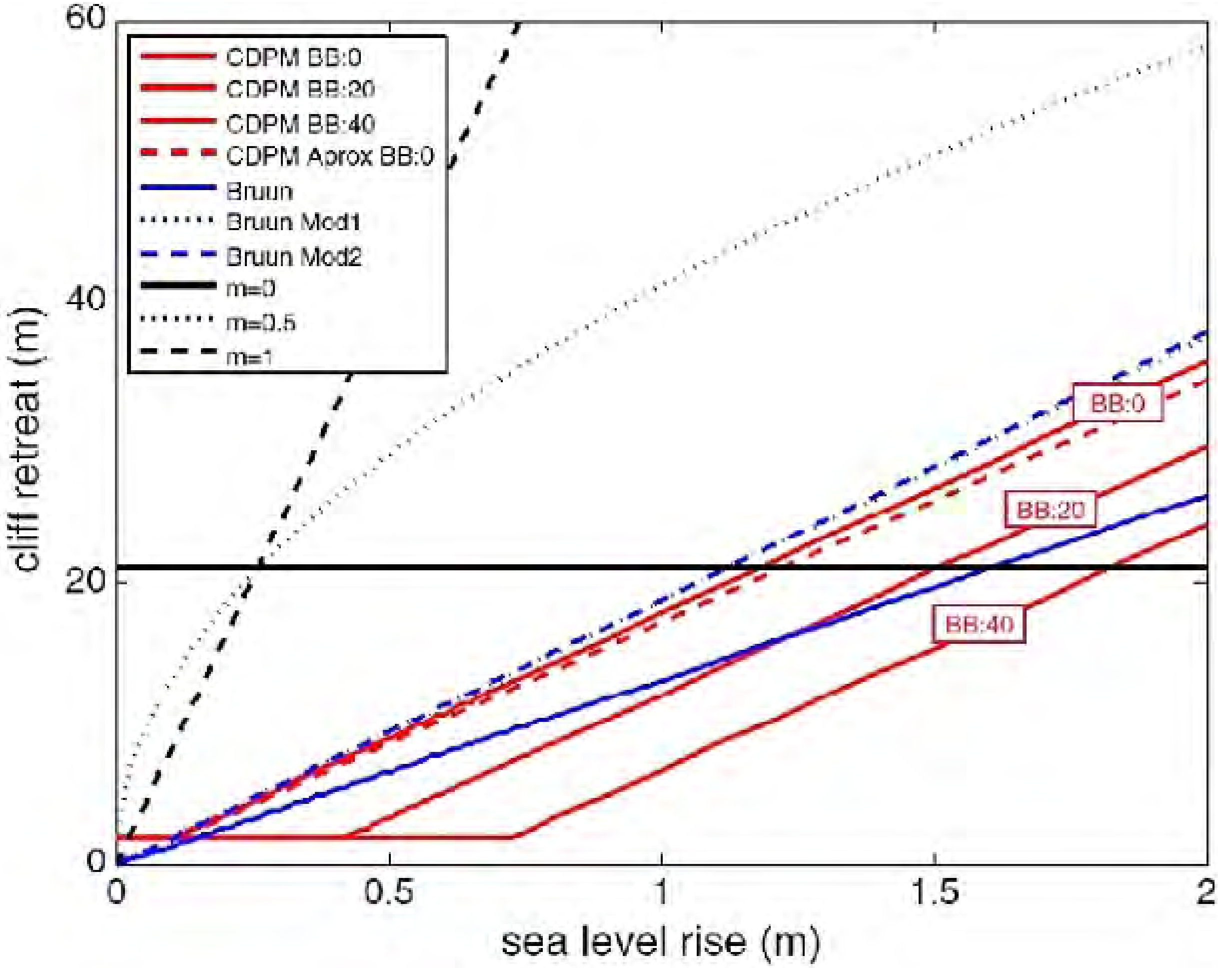


Fig. 12. Comparison of the conditionally decoupled profile model (CDPM) with 0, 20, and 40 m beach buffers (BB) and original Bruun, modified Bruun (Bruun Mod1 and Mod2), no feedback ($m = 0$), approximate SCAPE ($m = 0.5$), and linear extrapolation ($m = 1$). Exponent m models are based on historical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

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

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 rate based on the site specific photogrammetry sudy (CLSI, 2020), is between 0.065 (low) to 0.091(high) feet/year = R_1

2. Avg SLR rate over 91 years (1924 to 2015), based on NOAA (Scripps Pier, La Jolla) is 2.13 mm/yr = 0.084 inch/yr x 1 ft/12 in = 0.007 ft/yr = S1

3. Future SLR rate (2096), under *medium-high risk aversion scenario* = 6.5 ft/75 yrs = 0.087 ft/yr=S_2

4. m=⅓

GSI's assignment of the value for the exponent "m" is reasonable based on the response of the onsite coastal bluff to increased rates of SLR would lie somewhere between the instant response (m =1) and no feedback (m=0) systems discussed in Ashton, et al. (2011), and is likely closer to zero.

The three premises discussed previously (see CoSMoS discussion regarding SLR plots) 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 utilized the endpoints of the range of 0.065 ft/yr and 0.091 ft/yr for this time interval. The erosion rate should marginally increase for the following 25 years (2058-2083), and we have reasonably added ⅓ of the change in erosion rate in 2096, to the initial erosion rate (Δ, see below). During the more asymptotic SLR end of the 75-year design life (2083-2096), the bluff retreat rate should be closer to the site specific upper bound bluff retreat rate for this time interval, even though only the cemented bedrock would be impacted by SLR.

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

Low Site Specific Rate

At year 2096, under *medium-high risk aversion scenario (0.5% Probability)*,

$$R_2 = R_1\ (S_2/S_1)^m$$
$$R_2 = (0.065\ \text{ft/yr})\ (0.087\ \text{ft/yr}/[\ 0.007\ \text{ft/yr}\ ])^{1/3}$$
$$R_2 = (0.065)\ (12.43)^{1/3}$$
$$R_2 = (0.065)(2.32) = 0.151\ \text{ft/yr in the year 2096.}$$

Based on the above, the retreat rate will change from 0.065 to 0.151ft/yr, and the difference between the 75-year commencement and end of the design life, Δ= 0.086 ft/yr, from 2021 to 2096.

FUTURE BLUFF RETREAT BASED ON SLR CURVE INCREMENTS - Low			
APPLICABLE DATES	BLUFF RETREAT RATE (FT/YR)	DURATION (YEARS)	BLUFF RETREAT (FEET)
2021-2057 (0.065) current SLR rate	0.065	37	2.41
2058-2082 (0.065 + ⅓[Δ] = (0.065 +⅓[0.086]= 0.094) increase in SLR rate	0.094	25	2.35
2083-2096 (Calculated SLR rate in 2096 = 0.15)	0.15	13	1.95
	Totals	**75**	**6.71**

As shown above, the onsite coastal bluff may experience approximately 7 feet of retreat over the 75-year design life of the proposed residential structure, with a future SLR of 6.5 feet. Plate 2 shows 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 2096, based on the ±7 feet of bluff retreat, with an assumed SLR of ±6.5 feet over that design life interval.

High Site Specific Rate

At year 2096, assuming the high site specific rate applied under medium-high risk aversion scenario (0.5% Probability),

$$R_2 = R_1\ (S_2/S_1)^m$$
$$R_2 = (0.091\text{ ft/yr})\ (0.085\text{ ft/yr}/[\ 0.007\text{ ft/yr }])^{1/3}$$
$$R_2 = (0.091)\ (12.43)^{1/3}$$
$$R_2 = (0.091)(2.32) = 0.211\text{ ft/yr in the year 2096.}$$

Based on the above, the retreat rate will change from 0.091 to 0.211 ft/yr, and the difference between the 75-year commencement and end of the design life, $\Delta = 0.12$ ft/yr, from 2021 to 2096.

FUTURE BLUFF RETREAT BASED ON SLR CURVE INCREMENTS - High			
APPLICABLE DATES	**BLUFF RETREAT RATE (FT/YR)**	**DURATION (YEARS)**	**BLUFF RETREAT (FEET)**
2021-2057 (0.091) current SLR rate	0.091	37	3.37
2058-2082 (0.091 + ⅓[Δ]) = (0.091 +⅓[0.12])= 0.127 increase in SLR rate	0.131	25	3.28
2083-2096 (Calculated SLR rate in 2096 = 0.21)	0.211	13	2.74
	Totals	**75**	**9.39**

As shown above, using the high rate, the onsite coastal bluff may experience approximately ±9 feet of retreat over the 75-year design life of the proposed residential structure with a future SLR of 6.5 feet. Plate 2 shows 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 2096, based on the ±9 feet of bluff retreat, with an assumed SLR of ±6.5 feet over that interval. We note that <u>regardless</u> of the site specific erosion rate utilized, when added to the static FOS line from the bluff edge (± 24 feet), these rates plot well inside of the City of Encinitas 40-foot bluff-edge setback zone.

CALIFORNIA COASTAL COMMISSION:REALITY CHECK - "Best Available Science"

The CCC SLR Guidance (CCCSLRG) is based upon the California Ocean Protection Council (COPC) update to the State's Sea-Level Rise Guidance in March 2018. These COPC estimates are based upon a 2014 report entitled "Probabilistic 21st and 22nd century sea-level projections at a global network of tide-gauge sites" by Kopp, et al., 2014. The Kopp et al. paper used 2009 to 2012 SLR modeling by climate scientists for the probability analysis, which means the "best available science" used by the CCC is almost 10 years old. The SLR models used as the basis for the COPC and CCCSLRG have been in place for a decade. The accuracy of any model can be determined by comparing the measured SLR (real data) to the model predicted SLR (model prediction). If the model cannot predict, with any accuracy, what will happen in over the approximate last 11 years, it is very unlikely that the model will increase in accuracy when predicting SLR over the next 100 years. Simply put, if the model is not accurate now, it will be even less accurate in the future.

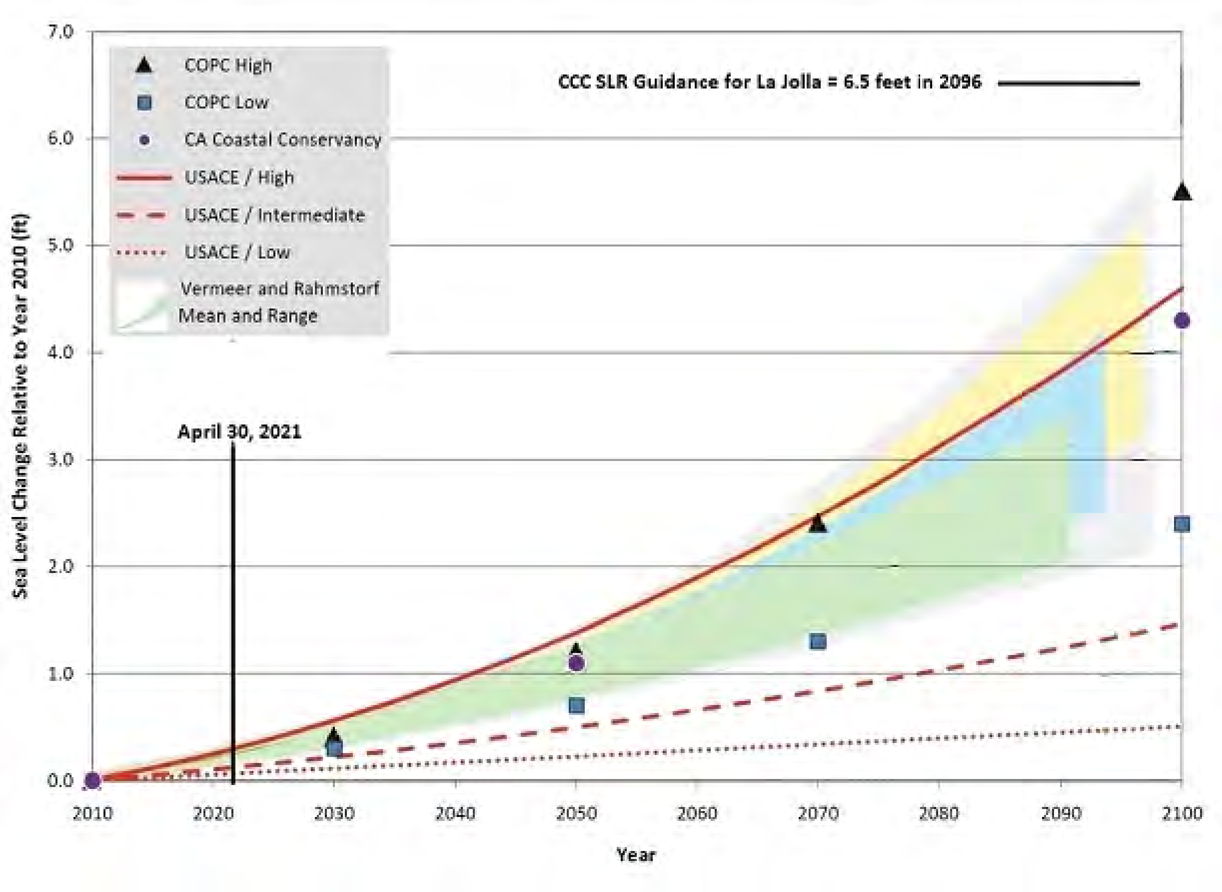


Figure 7 - SLR model projections after COPC

The National Oceanic and Atmospheric Administration (NOAA) has been measuring SLR globally and in La Jolla. The NOAA global SLR shows a current SLR rate of 3 mm/yr. The rate can be used to calculate a sea level rise of 33.99 mm (0.11 ft) over the last 11.33 years (2010 through April 2021). The NOAA La Jolla SLR rate is 2.13 mm/yr. The rate can be used to calculate a sea level rise of 24.13mm (0.079 ft) over the last 11 years and 4 months (2010 through April 2021). What is clear is that SLR in La Jolla is currently about 72% of the global SLR. What is also interesting is that the NOAA website shows that SLR is not occurring in several locations and the ocean is actually getting lower in elevation in these areas. Clearly "one size fits all" is not the best available science.

The COPC provided plots of the various SLR model projections over time starting in the year 2010. Figure 7, above, is the model projections taken from the COPC. To see which model is more accurately predicting SLR, the global data and the data for La Jolla can be plotted onto the curves. Figure 8, below, is an enlargement of a portion of Figure 7 to show the results. As stated before, the SLR in La Jolla is 72% of the global SLR over the same time period. The CCC Guidance SLR is almost 4 times the measured SLR in La Jolla. More importantly the current global SLR trend matches the USACE "Intermediate" SLR, which predicts about 1.5 feet of SLR by the year 2100. The SLR for La Jolla matches the USACE "Low" SLR, which predicts about 0.5 feet of SLR in the year 2100, an order of magnitude lower than the CCCSLRG (0.5 feet versus 6.5 feet).

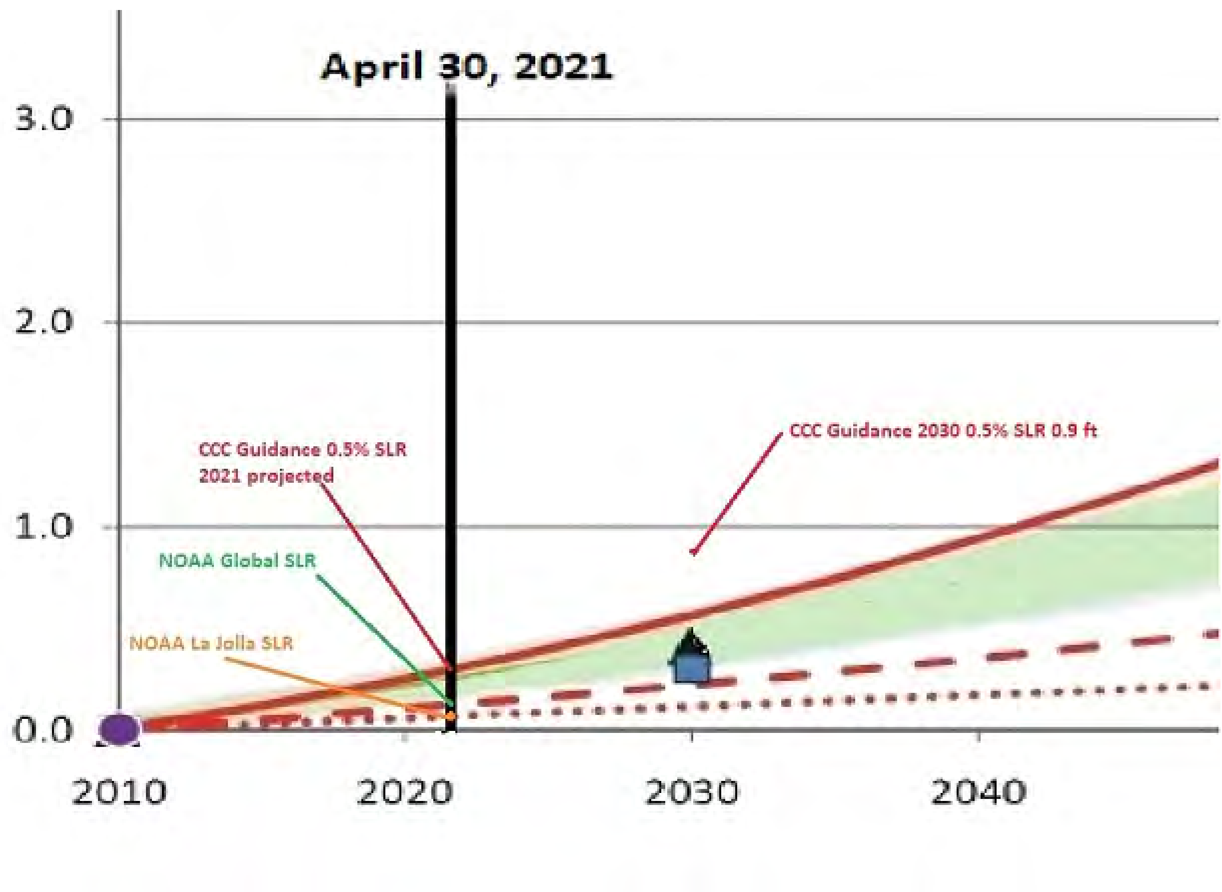


Figure 8 - Enlargement of Current SLR

The CCCSLRG document recommends that a project designer determine the range of SLR using the "best available science". The information provided above is more current than the CCC SLR Guidance. The checking of the models is the "best available science" for SLR prediction and is required to be used. Currently, the SLR model that the CCC is "requiring" to be used for development is <u>incorrect by almost a factor of 4</u> as to the amount of the SLR in La Jolla, and has been demonstrated to result in an <u>error of magnitude</u> in the year 2100. Clearly, the CCC required model does not predict the currently measured SLR and cannot be considered the "best available science".

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 northwesterly (Norris and Webb, 1990). This geomorphic 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.

The San Diego County region was originally a broad area composed of pre-batholithic rocks that were subsequently subjected to tectonism and metamorphism. In the late Cretaceous Period, the southern California Batholith was emplaced causing the aforementioned metamorphism of pre-batholithic rocks. Many separate magmatic injections originating from this body occurred along zones of structural weakness.

Following batholith emplacement, uplift occurred, resulting in the removal of the overlying rocks by erosion. Erosion continued until the area was that of low relief and highly weathered. The eroded materials were deposited along the sea margins. Sedimentation also occurred during the late Cretaceous Period. However, subsequent erosion has removed much of this evidence. In the early Tertiary Period, terrestrial sediments were deposited on a low-relief land surface. In Eocene time, previously fluctuating sea levels stabilized and marine deposition occurred. In the late Eocene, regional uplift produced erosion and thick deposition of terrestrial sediments. In the middle Miocene, the submergence of the Los Angeles Basin resulted in the deposition of thick marine beds in the northwestern portion of San Diego County. During the Pliocene, marine sedimentation was more discontinuous and generally occurred within shallow marine embayments. The Pleistocene saw regressive and transgressive sea levels that fluctuated with prograding and recessive glaciation. The changes in sea level had a significant effect on coastal topography and resultant wave erosion and deposition formed many terraces along the coastal plain. In the mid-Pleistocene, regional faulting separated highland erosional surfaces into major blocks lying at varying elevations. A later rise in sea level during the late Pleistocene, caused the deposition of thick alluvial deposits within the coastal river channels. In recent geologic time, crystalline rocks have weathered to form soil residuum, highland areas have eroded, and deposition of river, lake, lagoonal, and beach sediments has occurred.

Regional geologic mapping by Kennedy and Tan (2007, 2005) indicates that the site is immediately underlain by old paralic deposits (Subunits 6-7). This unit was formerly termed “terrace deposits” on older geologic maps. The old paralic deposits consist of marine and non-marine sediments deposited on wave cut platforms that emerged from the sea approximately 80,000 to 120,000 years before present. Kennedy and Tan (2007, 2005) indicate that the old paralic deposits are underlain by sedimentary bedrock belonging to the Tertiary Santiago Formation at this site. However, GSI disagrees with the mapped occurrence of the Santiago Formation below the subject site. Rather, it is our opinion that the Tertiary Torrey Sandstone underlies the old paralic deposits owing to the pervasive cross bedding exposed in the seacliff; a depositional characteristic more commonly associated with the Torrey Sandstone. The presence of the Torrey Sandstone in the seacliff portion of the coastal bluff corroborates the published findings of Eisenberg and Abbott (1985). The difference in nomenclature does not change the fundamental conclusions reported herein.

SITE GEOLOGIC UNITS

The onsite geologic units, encountered within the subject property during our subsurface investigation and site reconnaissance, included undocumented artificial fill and Quaternary-age old paralic deposits. Although not encountered in our subsurface explorations, Quaternary-age colluvium may mantle the surface of the subject property, westerly of the existing site improvements. Based on our observations of geologic exposures within the adjacent coastal bluff. The aforementioned earth materials are underlain at depth by Tertiary-age sedimentary bedrock belonging to the Torrey Sandstone Formation. Transient beach deposits occur along the base (toe) of the coastal bluff and seaward of the existing seawall. The beach deposits are also underlain by the Torrey Sandstone. The earth materials are generally described below from the youngest to the oldest. The distribution of these materials across the site and adjacent coastal bluff is shown in plan view on Plate 1.

Quaternary-age Beach Deposits (Map Symbol - Qb)

A transient shingle beach composed of light and dark gray fine- to medium-grained sand with cobbles exists at the base of the bluff and seaward of the existing seawall. The thickness of the beach deposits, near the toe of the bluff, at the time of our October 2019 field investigation is estimated at approximately 10 to 11 feet. The beach deposits will not be encountered during the proposed site development. The thickness of the beach deposits varies on a seasonal basis.

Artificial Fill - Undocumented (Map Symbol - Afu)

Undocumented artificial fill was encountered at the surface in the test pit and both hand-auger borings. As observed therein, the undocumented fill extended to depths ranging between approximately ½ foot b.e.g. in Test Pit TP-1 and Hand-Auger Boring HA-1 to greater than 6⅓ feet b.e.g. in Hand-Auger Boring HA-2. The fill primarily consisted of brownish gray, brown, dark gray, and light gray fine- to medium-grained sand with varying amounts of silt; and variegated reddish brown and light brownish gray fine- to medium-grained silty sand. The fill was generally dry to moist and very loose to medium dense. The undocumented fill, encountered within Hand-Auger Boring HA-2, is likely associated with the backfill of the existing, northerly basement wall, which may explain its relative thickness, as compared to its vertical extent within Test Pit TP-1 and Hand-Auger Boring HA-1. The fill encountered in Test Pit TP-1 and Hand-Auger Boring HA-1 may have been used for planting soil.

Documentation regarding the engineering suitability of the undocumented fill was unavailable for GSI review. Therefore, it should not be used for the support of the proposed settlement-sensitive improvements and/or new planned fills in its existing state.

Quaternary-age Colluvium (Not Mapped)

A surficial mantle of Quaternary-age colluvium (topsoil) may occur in undisturbed areas of the subject property, westerly of the existing site improvements. If present, the colluvium likely consists of brown and dark grayish brown silty sand and fine-to medium-grained sand. Colluvium is considered unsuitable for the support of the proposed settlement-sensitive improvements and/or new planned fills in its existing state. However, based on its location within the property and our understanding of the proposed site development, encountering colluvium during construction is considered unlikely.

Quaternary-age Old Paralic Deposits (Map Symbol - Qop)

Quaternary-age old paralic deposits were encountered beneath the undocumented artificial fill in test pit TP-1 and in hand-auger boring HA-1 at an approximate depth of ½ foot b.e.g. However, the upper approximately ¼ foot to 1 foot of these deposits, within the aforementioned subsurface explorations, exhibited some degree of weathering. These deposits were also observed in the coastal bluff exposure above approximate elevation 29 feet. Where weathered, the old paralic deposits generally consisted of dark yellowish brown fine- to medium-grained sand and light brown, fine- to medium-grained silty sand that were typically dry to moist and medium dense. The unweathered old paralic deposits, encountered in test pit TP-1 and in hand-auger boring HA-1, occurred at depths ranging between approximately ¾ foot and 1¼ feet b.e.g. The unweathered old paralic deposits generally consisted of dark yellowish brown and reddish yellow fine- to medium-grained sand and reddish yellow silty sand. The unweathered old paralic deposits were typically dry to damp and dense to very dense. Weathered old paralic deposits are considered unsuitable for the support of settlement-sensitive improvements and/or new planned fills in their existing state. Unweathered old paralic deposits are considered suitable bearing materials.

Tertiary Torrey Sandstone Formation (Map Symbol - Tt)

As reported in Eisenberg and Abbott (1985), sedimentary bedrock belonging to the Tertiary Torrey Sandstone Formation unconformably underlies the old paralic deposits within the subject site and adjacent coastal bluff. This formation was exposed in the sea cliff portion of the coastal bluff below elevation ±29 feet NAVD88. Based on our observations along the bluff exposure, the Torrey Sandstone consists of planar- and trough cross-bedded, yellow, reddish yellow, and light gray fine-grained sandstone, with localized concretions. The Torrey Sandstone described in the GSI (2000) boring log consists of light brown and olive gray fine- to medium-grained sandstone. Regionally, this formation is described as an arkosic, subangular, moderately well indurated sandstone (Kennedy, 1975). The Torrey Sandstone is believed to have been deposited along a submerging coast on an arcuate barrier beach. This beach enclosed and later transgressed over lagoonal sediments. Its deposition ceased when submergence slowed and the shoreline retreated. Based on its elevation below the portion of the site proposed for development, it is unlikely that the Torrey Sandstone Formation will be encountered during construction.

GEOLOGIC STRUCTURE

According to Eisenberg and Abbott (1985), the Torrey Sandstone is part of the southerly limb of a northeasterly plunging synclinal fold. Regional dip is approximately 5 degrees to the northwest (Eisenberg and Abbot, 1985; Kennedy and Tan, 2007). Joints roughly oriented sub-parallel to the bluff alignment were observed in the coastal bluff exposure during our previous field work at nearby 330 Neptune Avenue (GSI, 2000). However, we did not observe any readily visible evidence of bluff-parallel joints in the sea cliff exposure below the subject site. The overlying old paralic deposits are undeformed and generally contain thick, nearly sub-horizontal beds. Rather dramatic undercutting of the Torrey Sandstone was observed within the sea cliff below the adjacent southerly property.

FAULTING AND REGIONAL SEISMICITY

Local and Regional Faults

Our review and field observations indicates that there are no known Holocene-active faults (i.e., faults exhibiting movement in the last 11,700 years) crossing the subject site (Jennings and Bryant, 2010), and the site is not within an Alquist-Priolo Earthquake Fault Zone (California Geological Survey, 2018). However, the site is situated in a seismically active region with numerous Holocene-active and potentially active pre-Holocene fault traces. These faults include, but are not limited to: the San Andreas fault; the San Jacinto fault; the Elsinore fault; the Coronado Bank fault zone; and the Newport-Inglewood - Rose Canyon fault zone (NIRCFZ). Portions of the nearby NIRCFZ are located in an Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). According to Blake (2000), the closet known active fault to the subject site is the Rose Canyon fault; located at a distance of 3.2 miles (5.1 kilometers) to the west. Portions of the Rose Canyon fault have demonstrated movement in the last 11,700 years; and therefore, are considered Holocene-active and located in an Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). Cao, et al. (2003) indicate that Rose Canyon fault is an "B" fault with a slip rate of 1.5 ($\pm$0.5) millimeters per year, and is capable of producing a maximum magnitude (M_w) 7.2 earthquake in this area.

Surface Fault Rupture

Surface fault rupture is the displacement of the ground surface caused by fault propagation extending to the surface of the earth's crust. Since there are no known Holocene-active faults crossing the site (Jennings and Bryant, 2010; California Geological Survey, 2018), the potential for surface fault rupture to adversely affect the proposed development is considered very low.

Seismicity

Deterministic Maximum Credible Site Acceleration

The acceleration-attenuation relation of Bozorgnia, Campbell, and Niazi (1999) has been incorporated into EQFAULT (Blake, 2000a). EQFAULT is a computer program developed by Thomas F. Blake (2000a), which 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 ("maximum credible") earthquake on that 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 at the site may be on the order of 0.74 g. The computer printouts of pertinent portions of the EQFAULT program are included within Appendix C.

Historical Site Acceleration

Historical site seismicity was evaluated with the acceleration-attenuation relation of Bozorgnia, Campbell, and Niazi (1999), and the computer program EQSEARCH (Blake, 2000b). 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 August 15, 2018. Based on the selected acceleration-attenuation relationship, a peak horizontal ground acceleration is estimated, which may have effected the site during the specific event listed. Based on the available data and the attenuation relationship used, the estimated maximum (peak) site acceleration during the period 1800 through August 15, 2018 was 0.55 g. A historic earthquake epicenter map and a seismic recurrence curve are also estimated/generated from the historical data. Computer printouts of the EQSEARCH program are presented in Appendix C.

Seismic Shaking Parameters

Based on the site conditions, the following table summarizes the site-specific design criteria obtained from the 2019 CBC (CBSC, 2019), Chapter 16 Structural Design, Section 1613, Earthquake Loads. The computer program "U.S. Seismic Design Maps," provided by the Structural Engineers Association of California and California Office of Statewide Health Planning and Development ([SEAOC and OSHPD], 2020), was utilized to aid with design. The short spectral response utilizes a period of 0.2 seconds.

2019 CBC SEISMIC DESIGN PARAMETERS			
PARAMETER	**VALUE**	**SITE SPECIFIC VALUE per ASCE 7-16**	**2019 CBC or REFERENCE**
Risk Category	I, II or III	-	Table 1604.5
Site Class	D	-	Section 1613.2.2/Chap. 20 ASCE 7-16 (p. 203-204)
Spectral Response - (0.2 sec), S_s	1.243 g	-	Section 1613.2.1 Figure 1613.2.1(1)
Spectral Response - (1 sec), S_1	0.441 g	-	Section 1613.2.1 Figure 1613.2.1(2)
Site Coefficient, F_a	1.0	-	Table 1613.2.3(1)
Site Coefficient, F_v	null - see Section 11.48 ASCE 7-16	2.5 (Section 21.3)	Table 1613.2.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.246 g	-	Section 1613.2.3 (Eqn 16-36)
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	null - see Section 11.48 ASCE 7-16	1.2 (Section 21.4)	Section 1613.2.3 (Eqn 16-37)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.831 g	-	Section 1613.2.4 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	null - see Section 11.48 ASCE 7-16	0.8 (Section 21.4)(3)	Section 1613.2.4 (Eqn 16-39)
PGA_M - Probabilistic Vertical Ground Acceleration may be assumed as about 50% of these values.	0.618 g	-	ASCE 7-16 (Eqn 11.8.1)

2019 CBC SEISMIC DESIGN PARAMETERS			
PARAMETER	VALUE	SITE SPECIFIC VALUE per ASCE 7-16	2019 CBC or REFERENCE
Seismic Design Category	null - see Section 11.48 ASCE 7-16	D (Section 11.6)	Section 1613.2.5/ASCE 7-16 (p. 85: Table 11.6-1 or 11.6-2)
1. $F_V = 2.5$ $S_1 > 0.2$ per Section 21.3,			
2. $S_{M1} = (1.5)S_{D1} = (1.5)(0.8) = 1.2$ per Section 21.4			
3. $S_{D1} \geq 0.2 \Rightarrow 0.8 \geq 0.2$, per Section 11.6 site is in Risk Category D			

GENERAL SEISMIC PARAMETERS	
PARAMETER	VALUE
Distance to Seismic Source (Rose Canyon fault)	3.2 mi (5.1 km)(1)
Upper Bound Earthquake (Rose Canyon)	$M_w = 7.2$(2)
(1) - Blake (2000a)	
(2) - Cao, et al. (2003)	

Conformance to the criteria above for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur in the event of a large earthquake. The primary goal of seismic design is to protect life, not to eliminate all damage, since such design may be economically prohibitive. Cumulative effects of seismic events are not addressed in the 2019 CBC (CBSC, 2019) and regular maintenance and repair following locally significant seismic events (e.g., M_w4.5) will likely be necessary.

GROUNDWATER

GSI did not encounter groundwater nor evidence of perched water in the subsurface explorations completed at the subject site to the explored depths. The regional groundwater table is anticipated to be near sea level or approximately 73 feet below the lowest site elevation.

Groundwater was not observed seeping out of the bluff face during our October 2019 field study. However, a review of oblique aerial photographs available on the California Coastal Records Project website (www.californiacoastline.org) indicated possible evidence of groundwater seepage from the face of the bluff in the past. Moreover, GSI encountered perched groundwater along the old paralic deposits/Torrey Sandstone contact in a boring advanced at nearby 330 Neptune Avenue in 2000 (GSI, 2000). GSI also encountered seepage at an approximate depth of 60 feet below the surface in this boring. This depth roughly equates to an elevation of 15 feet above sea level. Accordingly, these groundwater surfaces were modeled in our slope stability analyses.

Groundwater is not expected to be a major factor in site development. However, due to the nature of the site earth materials, seepage and/or perched groundwater conditions may continue to develop throughout the site in the future, both during and subsequent to development, especially along boundaries of contrasting permeabilities and densities (i.e., sandy/clayey fill lifts, geologic contacts, bedding, discontinuities, etc.), and should be anticipated. The manifestation of perched water is the result of numerous factors including site geologic conditions, rainfall, irrigation, broken or damaged wet utilities, etc. This potential should be disclosed to all interested/affected parties. Should perched water conditions be encountered within the property, GSI could provide recommendations for mitigation. Mitigation may include but not necessarily be limited to the installation of subdrains and/or concrete or slurry cut-off walls.

Due to the potential for post-development perched water to manifest near the surface, owing to as-graded permeability contrasts, more onerous slab design is necessary for any new slab-on-grade floor (State of California, 2020). Recommendations for reducing the amount of water and/or water vapor through slab-on-grade floors are provided in the "Soil Moisture Considerations" sections of this report. It should be noted that these recommendations should be implemented if the transmission of water or water vapor through the slab or wall is undesirable. Should these mitigative measures not be implemented, then the potential for water or vapor to pass through the foundations and slabs and resultant distress cannot be precluded, and would need to be disclosed to all interested/affected parties.

LIQUEFACTION POTENTIAL

Liquefaction

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 and/or will produce vertical strains well below 1 percent for 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 psf [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 medium- to fine-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 or two of these necessary five concurrent conditions have the potential to affect the site.

Seismic Densification

Seismic densification is a phenomenon that typically occurs in low relative density granular soils (i.e., United States Soil Classification System [USCS] soil types SP, SW, SM, and SC) that are above the groundwater table. These unsaturated granular soils are susceptible if left in the original density (unmitigated), and are generally dry of the optimum moisture content (as defined by the ASTM D 1557). During seismic-induced ground shaking, these natural or artificial soils deform under loading and volumetrically strain, potentially resulting in ground surface settlements. Some densification of the undocumented basement wall backfill could occur during a significant seismic event. In addition, unmitigated soils on the adjoining properties may experience seismic densification and influence improvements at the perimeter of the site. Special setbacks and/or foundations may be utilized for significant structures/improvements if they are located within a horizontal distance of “D” where “D” equals the depth of the potentially densifiable soils (in feet). Our evaluation assumed that the current offsite conditions will not be significantly modified by future grading at the time of the design earthquake, which is a reasonably conservative assumption.

Summary

It is the opinion of GSI that the susceptibility of the site to experience damaging deformations from seismically-induced liquefaction is relatively low owing to the dense, nature of the old paralic deposits and the underlying Torrey Sandstone Formation, and the depth to groundwater. In addition, the recommendations for remedial earthwork and foundations would further reduce any significant liquefaction potential. Some seismic densification of the undocumented basement wall backfill at the subject site and the unmitigated soils on the adjoining properties may adversely influence the existing and proposed onsite improvements. However, special setbacks and/or foundations may be utilized for significant structures/improvements if they are located within a horizontal distance of “D” where “D” equals the depth of the potentially densifiable soils (in feet). The removal and recompaction of the undocumented basement wall backfill may also be undertaken.

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 and/or surficial failures).

According to regional landslide hazard mapping performed by the State of California Department of Conservation - Division of Mines and Geology (Tan and Giffen, 1995), the subject site is located within Relative Landslide Susceptibility Subarea 4-1 which is characterized as being most susceptible to landslides. According to Tan and Giffen (1995), this subarea is generally located outside the boundaries of definite mapped landslides. However, this subarea contains unstable slopes underlain by both weak materials, and adverse geologic structure. This subarea also includes questionable landslides and oversteepened high coastal bluffs, subject to active marine erosion.

Based on our review of regional geologic maps, there is no evidence of deep-seated landslides at the subject site nor did we observe any geomorphic expressions indicative of significant on-going or past deep-seated instability or large mass wasting events at the site. The old paralic deposits and the Torrey Sandstone Formation are typically high strength materials and generally not susceptible to deep-seated slope failures. However, the coastal bluff that descends from the site is considered surficially unstable, as it is subject to both marine and subaerial erosional processes.

The Torrey Sandstone materials exposed in the seacliff, have typically experienced past toppling failure events due to the presence of nearly bluff-parallel joints and notching along the bluff toe. However, the existing seawall will provide some protection toppling failure within the portion of the seacliff located directly behind the wall. The seacliff to the north and south of the seawall is undercut and susceptible to future toppling events, which was modeled in our slope stability analysis.

GSI believes that the old paralic deposits in the middle and upper portions of the coastal bluff are susceptible to continuous raveling and debris flow failures. Given the potential for episodic, low magnitude surficial bluff failures and retreat, GSI has analyzed slope stability for the proposed development. The results of our analyses are presented in the "Slope Stability" section and Appendix E of this report.

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 California Emergency Management Agency, et al. (2009), the subject property is not located in a tsunami inundation zone. Thus, the proposed development is at low risk for tsunami inundation. However, the coastal bluff descending from the site is located within a tsunami inundation zone, and could experience some erosion from a tsunami impact. But GSI points out that 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 and/or mitigated as a result of site location, soil characteristics, freeboard, and typical site development procedures:

- Subsidence
- Seiche
- Dynamic Settlement
- Ground Lurching or Shallow Ground Rupture

It is important to keep in perspective that in the event of a major earthquake occurring on any of the nearby major faults, strong ground shaking would occur in the subject site's general area. Potential damage to any structure(s) would likely be greatest from the vibrations and impelling force caused by the inertia of a structure's mass than from those induced by the hazards considered above. Following implementation of remedial earthwork and design of foundations described herein, this potential would be no greater than that for other existing structures and improvements in the immediate vicinity that comply with current and adopted building standards.

EXCAVATION CHARACTERISTICS

Based on our experience with similar sites, excavations into the artificial fill, Quaternary old paralic deposits, and colluvium with standard heavy-duty excavation equipment in good working condition would likely range between easy and moderately difficult. However, cemented zones within the old paralic deposits could result in very difficult excavation with relatively lightweight excavation equipment (i.e., rubber-tire backhoe and mini-excavator). As such, the localized need for rock breaking equipment cannot be entirely precluded. Excavation equipment should be appropriately sized and powered for the required excavation task.

LABORATORY TESTING

General

Laboratory tests were performed on representative 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. The results of the GSI (2000) shear strength tests, performed on soil samples collected from nearby 330 Neptune Avenue are also summarized below.

Classification

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

Expansion Potential

Expansion index (E.I.) testing was performed on a representative sample of site soil in general accordance with ASTM D 4829. The results of E.I. testing are presented in the following table:

SAMPLE LOCATION AND DEPTH (FT)	SOIL TYPE	EXPANSION INDEX	EXPANSION POTENTIAL*
HA-1 @ 1¼-3½	Silty Sand (Qop)	<5	Very Low
* Classification per ASTM D 4829			

Sieve Analysis

The grain-size distribution of a sample of the site earth materials, collected from hand-auger boring HA-2 between depths of approximately 3¼ and 6⅓ feet, was evaluated in general accordance with ASTM D 422. Based on this test, this earth material is generally classified as a silty sand (Unified Soil Classification System [USCS] symbol - SM). The grain-size distribution curve is presented in Appendix D.

Direct Shear Tests

Shear testing was previously performed on undisturbed samples of the old paralic deposits and Torrey Sandstone collected during our field work at nearby 330 Neptune Avenue in preparation of GSI (2000). Shear tests were conducted in general accordance with ASTM test method D-3080. Test results are presented on the following table.

SAMPLE LOCATION AND DEPTH (FT)	COHESION (psf)	INTERNAL FRICTION (degrees)
B-1 @ 10-11 (Qop)	320	34
Base of Bluff (Torrey Sandstone)	1,000	37

Soil pH, Saturated Resistivity, Soluble Sulfates and Soluble Chlorides

GSI conducted testing on a sample of the site earth materials, collected during our site-specific field work, for an evaluation of general soil corrosivity and soluble sulfates, and soluble chlorides. Testing was performed in general accordance with ASTM G 51-95 2012, ASTM G 57-06 2012, ASTM D 516-16, and ASTM D 512-12B. 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)
HA-2 @ 3¼-6⅓	7.4	2,400	0.0160	40

Corrosion Summary

Laboratory testing indicates that the tested sample of the onsite soils is mildly alkaline with respect to soil acidity/alkalinity; is moderately corrosive to exposed, buried metals when saturated; presents negligible sulfate exposure to concrete (Exposure Classes S0, C1, and W0 per Table 19.3.1.1 of ACI 318-14); and has relatively low concentrations of soluble chlorides. It should be noted that 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/or structural engineer. Please note that the site is located in close proximity to the Pacific Ocean, considered a corrosive environment. Therefore, the proposed improvements would be subject to sea spray. Thus, corrosive effects from sea spray should be considered in the project design and construction.

SLOPE STABILITY ANALYSIS

GSI performed slope stability analyses utilizing the geologic conditions we observed in the coastal bluff and encountered in our hand-auger borings. Our slope stability analysis also included the shear strength parameters (unit weights, cohesion, and friction angle) of old paralic deposits and Torrey Sandstone samples we obtained during field work at nearby 330 Neptune Avenue in 2000 (GSI, 2000). Shear strength parameters assigned to the beach deposits were based on our experience with these earth materials. Our analyses were performed utilizing the two-dimensional slope stability computer program "GSTABL7 v.2" (Gregory, 2013). Our analyses incorporated the limit-equilibrium approaches modeled in the Modified Bishop and GLE (Spencer's) methods. Additional information regarding the methodology utilized in this program is included in Appendix E. Shear strength parameters used in the analysis are also provided in Appendix E.

A representative geologic cross-section (Geologic Cross Section X-X' [see Plates 1 and 2]) was prepared for the analysis. This section was analyzed because it represents a portion of the bluff that is absent shore protection and where the seacliff is experiencing the most severe toe erosion (basal notching). It also represents the steepest portion of the upper bluff slope. For conservatism, we have modeled a post-failure seacliff, where currently overhanging, and a steepened upper bluff slope that would be produced in response to a seacliff failure . In addition, we have projected loads from the building foundation and slab-on-grade floor but have not included the basement floor level, which increases driving forces for failure (a conservative approach).

We replicated a perched groundwater surface slightly above the geologic contact between the old paralic deposits and the Torrey Sandstone, as well as a perched groundwater surface within the Torrey Sandstone between approximate elevations 15 and 21 feet NAVD88. For pseudo-static (seismic) analyses, GSI included a seismic coefficient (*k*) equal to 0.15 which is considered relatively conservative for the site vicinity given the maximum magnitude 7.2 design earthquake along the Rose Canyon fault.

We obtained static and seismic factors-of-safety (FOS) respectively greater than 1.5 and 1.1 for static and seismic conditions for both an upper bluff failure (see Plates E-1 and E-2 in Appendix E). It is the opinion of GSI that an upper bluff failure is the most likely global slope failure mechanism at the subject site. The criteria for the geologic setback recognized by the City of Encinitas is the greater of 40 feet, or the static FOS $\geq$ 1.5, or 75-year retreat rate. The results of the slope stability analyses show that the existing residential structure with the proposed addition has acceptable factors-of-safety at 40 feet landward of the coastal bluff edge. Therefore, GSI recommends a development setback of 40 feet landward of the bluff edge per City of Encinitas Local Coastal Program (LCP) minimum requirements. The recommended 40-foot development setback is shown on Plates 1 and 2. Recently, GSI has been requested by the City of Encinitas to show the location within coastal bluff properties where the additive of the static 1.5 FOS and the 75-year retreat distance occurs. Thus, we have also shown this position on Plates 1 and 2.

Surficial Slope Stability

Based on published and accepted erosion rates, our analyses, and our observations, the coastal bluff is inherently surficially unstable. However, based on our aforementioned findings regarding site-specific coastal bluff retreat, a proposed residential structure prescriptively sited 40 feet from the bluff setback line, would not be adversely affected from retreat over its 75-year design life.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on our field exploration, laboratory testing, and geotechnical engineering analysis, it is our opinion that the site appears suitable to receive the proposed residential development from a geotechnical engineering and geologic viewpoint, 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 development are:

- Bluff stability and bluff retreat throughout the design life of the proposed improvements.
- Earth materials characteristics and depth to competent 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).
- Potential for perched groundwater to occur during and after development.
- Non-structural zone on un-mitigated perimeter conditions (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 have been completed using the information provided and obtained during our field work. In the event that any significant changes are made to proposed site development, the conclusions and recommendations contained in this report shall not be considered valid unless the 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 grading to verify and/or further evaluate geologic conditions. Although unlikely, if adverse geologic structures are encountered, supplemental recommendations and earthwork may be warranted.

3. Based on our experience, the bluff adjacent to the subject site and proposed development would have higher susceptibility to experiencing a failure within the upper bluff slope than a gross failure of the entire bluff slope. Slope stability analyses modeling a failure of the upper bluff indicate that the location along Geologic Cross Section X-X’ where the static and seismic FOS against this failure

mode are at least 1.5 and 1.1, respectively, are approximately 24 and 30 feet landward of the coastal bluff edge. Thus, the proposed residence can be constructed at the minimum 40-foot bluff edge setback prescribed by the City of Encinitas. In addition, our slope stability analyses indicate that this setback distance should provide sufficient protection from coastal bluff retreat over the 75-year design life of the proposed development, even without the existing seawall that currently provides protection from marine erosion. With regular and periodic maintenance, this seawall will continue to defend against bluff retreat in the future.

4. All undocumented fill, colluvium (if any), and near-surface weathered portions of the old paralic deposits are considered potentially compressible in their existing state and therefore, should not be used for the support of planned settlement-sensitive improvements (i.e., new foundations, new slab-on-grade floors, underground utilities, walls, hardscape, etc.) and/or new planned fills. These earth materials should be removed and reused as properly engineered fill for support of the proposed improvements and planned fills. A 2-sack slurry may also be used to backfill remedial excavations. <u>If slurry backfill is used, the project structural engineer should evaluate the effects of uncured, fluid slurry on the existing basement walls, prior to placement. In addition, the project landscape architect should specify the hold-down depth for slurry backfill relative to finish grade to allow for planting.</u> Alternatively, new foundations (if required) can be deepened to extend through unsuitable soils and into the underlying dense old paralic deposits. New slab-on-grade floors (if proposed) can be constructed upon potentially compressible soils provided that they are designed such that support from these soils is not necessary (i.e., structural slab). If any existing footing lacking the herein recommended depths of embedment or currently bearing upon potentially compressible soils is to receive new structural loads, it should be underpinned into dense old paralic deposits. Due to limited working space within the site remedial grading will be challenging.

   Based on the available subsurface data, potentially compressible earth materials, within the areas of proposed development will likely extend to depths of approximately ¾ foot to 6⅓, and perhaps as much as 10 feet below the existing grades. The thickest sections of unsuitable earth materials are likely associated with the existing basement wall backfill. There is some potential that compressible earth materials could extend to greater depths, locally.

5. Laboratory testing indicates that the E.I. of a representative sample of the onsite earth materials is less than 5. This correlates to a very low expansion potential. Thus, the onsite soils do not meet the criteria of expansive soils indicated in Section 1803.5.3 of the 2019 CBC (CBSC, 2019). Based on the available data, specific structural design nor remedial earthwork is warranted for the mitigation of expansive soils on a preliminary basis.

6. Based on our review of the results of soil pH, saturated resistivity, soluble sulfates, and soluble chlorides testing, GSI concludes that the tested soil sample is mildly

alkaline with respect to soil acidity/alkalinity; is moderately corrosive to exposed, buried metals when saturated; presents negligible sulfate exposure to concrete (Exposure Classes S0, C1, and W0 per Table 19.3.1.1 of ACI 318-14); and has relatively low chloride concentrations. GSI does not consult in corrosion engineering. Therefore, additional comments and recommendations may be obtained from a qualified corrosion engineer based on the level of corrosion protection desired or required for the project, as determined by the Project Architect, Civil Engineer, and/or Structural Engineer. Owing to the site's proximity to the Pacific Ocean, the effects of sea spray should be considered in the design and construction of the proposed development.

7. In general and based upon the available data to date, regional groundwater is not expected to be encountered during construction of the proposed site improvements, nor are anticipated to adversely affect site development. However, there is potential for perched water 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 water to occur should be disclosed to all interested/affected parties.

8. It should be noted, that the 2019 CBC (CBSC, 2019a) indicates that remedial grading be performed across all areas to be graded under a grading permit, not just within the influence of the proposed improvements. Relatively deep removals may also necessitate a special zone of consideration, on perimeter/confining areas. This zone would be approximately equal to the thickness of potentially compressible earth materials if remedial grading cannot be performed onsite and offsite. The width of this zone would be considered equal to the depth of the remedial grading excavations adjacent to property boundaries or existing improvements that need to remain in serviceable use both during and following construction. Any settlement-sensitive improvements, constructed within this zone, may require deepened foundations, reinforcements, 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.

9. On a preliminary basis, unsupported temporary excavation walls ranging between 4 and 20 feet in gross overall height, completed into <u>existing</u> artificial <u>fill</u> and colluvium should be constructed in accordance with Cal/OSHA guidelines for Type "C" soils (i.e., 1½:1 [h:v] gradient), provided groundwater, seepage, or running sands are <u>not</u> present. All temporary excavation walls should be observed by a licensed engineering geologist or geotechnical engineer prior to worker entry. Temporary slope gradients may need to be altered to flatter gradients should potentially adverse condition be exposed. Shored excavations will be necessary where existing improvements and property do not allow for the recommended temporary slope gradients. Limited space on the property could make shoring installation challenging.

10. Site soils are considered erosive. As such, the proper control of surface drainage is considered essential in minimizing the adverse effects of erosion on the coastal bluff. Surface drainage should be evaluated by a licensed civil engineer.

11. 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 development.

12. 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 Encinitas, and the Grading Guidelines presented in Appendix F, except where specifically superceded in the text of this report. Prior to grading, a GSI representative should be present at the preconstruction meeting to provide additional grading guidelines, if needed, and 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 and/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. Per the City of Encinitas, no grading should be performed within 40 feet of the coastal bluff edge.

Site Preparation

Debris, vegetation, and other deleterious material should be removed from the building area prior to the start of earthwork construction.

Remedial Removals (Unsuitable Surficial Materials)

Due to the relatively loose condition of undocumented artificial fills, any colluvium, and the weathered old paralic deposits, these materials should be removed to expose dense, unweathered old paralic deposits and then be reused as engineered fill in areas to receive proposed settlement-sensitive improvements or new planned fills. Alternatively, the remedial excavations could be backfilled with a 2-sack sand-cement slurry. If slurry backfill is used, the project structural engineer should evaluate the effects of uncured, fluid slurry on the existing basement walls, prior to placement. In addition, the project landscape architect should specify the hold-down depth for slurry backfill relative to finish grade to allow for planting. Based on the available subsurface data, potentially compressible earth materials, within the areas of proposed development will likely extend to depths of approximately ¾ foot to perhaps as much as 10 feet below the existing grades. However, potentially compressible earth materials may be expected to extend to greater depths locally. For improvements that will not be founded into or directly supported by unweathered old paralic deposits, remedial grading excavations should be completed below a 1:1 (h:v) projection down from the bottom, outermost edge of proposed settlement-sensitive improvements and/or limits of new planned fills unless constrained by property lines or existing structures to remain in service both during and following construction. Due to limited space within the property remedial grading with excavation equipment

Fill Placement

1. Subsequent to ground preparation, fill materials should be cleansed of major vegetation and debris, brought to at least optimum moisture content, placed in thin 6- to 8-inch lifts, and mechanically compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557). Where a proposed improvement would receive support from both old paralic deposits and recompacted fills or recompacted fills and slurry, the fill should be compacted to at least 95 percent of the laboratory standard (ASTM D 1557).

2. Any import materials should be observed and determined suitable by the geotechnical engineer prior to placement on the site. Import material (if required) should be very low expansive (E.I. $\leq$ 20 with a plasticity index (P.I.) $\leq$ 14). Foundation designs may be altered if import materials have a greater expansion value than the onsite materials encountered in this investigation. If the import material originates 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.

Temporary Slopes

On a preliminary basis, temporary excavations greater than 4 feet, but less than 20 feet in overall height, completed into the onsite earth materials should conform to CAL-OSHA and/or OSHA requirements for Type "C" soil conditions (i.e., 1½:1 [h:v] gradient) provided that groundwater, running sands, and/or other adverse conditions are not present. All temporary excavations should be observed by a licensed engineering geologist or geotechnical engineer prior to worker entry. Although not anticipated, based on the available data, if temporary slopes conflict with property boundaries, shoring or alternating slot excavations may be necessary. The need for shoring or alternating slot excavations should be further evaluated during the grading plan review stage.

Alternating Slot Excavations

Alternating "A", "B", and "C" slot excavations should be performed when completing remedial earthwork below a 1:1 (h:v) plane projected down from property lines or existing improvements to remain in service. 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 12-foot width of undisturbed soils or recompacted fill. Open slots should be observed by GSI prior to backfill.

PRELIMINARY RECOMMENDATIONS - FOUNDATIONS

General

The following recommendations for the design and construction of any new foundations (including underpin footings) and slab-on-grade floors are considered preliminary, and are based on assumed loading conditions and current E.I. data. Final foundation design recommendations will be provided at the conclusion of grading based on the actual loading conditions and the E.I. of finish grade soils. The following recommendations assume that the foundation will be underlain by at least 7 feet of very low expansive soils (E.I. $\leq$ 20 and P.I. $\leq$ 14).

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

Shallow Spread Footing Design (Including Underpin Footings)

1. Any new foundation system should be designed and constructed in accordance with guidelines presented in the 2019 CBC.

2. An allowable bearing value of 2,000 pounds per square foot (psf) may be used for design of continuous footings that maintain a minimum width of 12 inches and a minimum embedment of 12 inches into dense, unweathered old paralic deposits, compacted fill placed in accordance with the recommendations previously provided in this report, or a 2-sack slurry. This value may be increased by 20 percent for each additional 12 inches of footing embedment to a maximum value of 2,500 psf. In addition, this value may be increased by one-third when considering short duration seismic or wind loads. The same allowable bearing value may be used in the design of isolated footings that have a minimum dimension of at least 24 inches square and a minimum embedment of 24 inches into unweathered old paralic deposits, compacted fill placed in accordance with the recommendations previously provided in this report, or a 2-sack slurry placed on bedrock.

3. Passive earth pressure for a footing laterally supported by very low expansive soils and slurry may be computed as an equivalent fluid having a density of 250 pcf, with a maximum earth pressure of 2,500 psf.

4. An allowable coefficient of friction between soil and concrete of 0.35 may be used with the dead load forces.

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

6. Footings for structures adjacent to retaining walls should be deepened so as to extend below a 1:1 projection from the heel of the wall should this condition occur. Alternatively, walls may be designed to accommodate structural loads from buildings or appurtenances as described in the "Retaining Wall" section of this report.

7. The design of concrete slab-on-grade floors underlain by left-in-place potentially compressible soils should be in accordance with the recommendations of the Project Structural Engineer.

Foundation Settlement

Shallow spread footings founded into unweathered old paralic deposits, recompacted fills, or a 2-sack slurry per the recommendations contained herein should be minimally designed to accommodate a total settlement of 2 inches and a differential settlement of at least 1 inch in a 40-foot span (angular distortion = 1/480).

Footing Setbacks

All new footings should maintain a minimum 40-foot horizontal setback from the bluff edge. Footings adjacent to any other site slopes with gradients steeper than 5:1 (h:v) should be setback in accordance with the guidelines presented in 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 slope face.

Construction

The following foundation construction recommendations are presented as a minimum criteria from a geotechnical engineering standpoint. Current data indicates that the onsite earth materials are very low expansive (i.e., E.I. $\leq$ 20 and P.I. $\leq$ 14). Recommendations for foundations underlain by at least 7 feet of very low expansive soils are presented herein.

Recommendations by the project's design-structural engineer or architect, which may exceed the geotechnical engineer's recommendations, should take precedence over the following minimum requirements. Final foundation design will be provided based on the expansion potential of soils exposed near finish grade at the conclusion of grading/earthwork.

1. Exterior and interior continuous footings should be founded at a minimum depth of 12 inches for one-story floor loads, 18 inches for two-story floor loads, and 24 inches for three-story floor loads, into unweathered old paralic deposits, compacted fill placed in accordance with the recommendations previously provided in this report, or a 2-sack slurry. Isolated column and panel pad footings, should be 24 inches square and should be founded at a minimum depth of 24 inches into unweathered old paralic deposits, recompacted fill placed in accordance with the recommendations previously provided in this report, or a 2-sack slurry. All footings should be reinforced with four No. 4 reinforcing bars, two placed near the top and two placed near the bottom of the footing. Continuous footing widths should be 12, 15, and 18 inches for one-, two-, and three-story floor loads, respectively. Isolated, interior and exterior footings should be tied to the perimeter foundation in at least one direction with a grade beam.

2. A grade beam, reinforced as above, and at least 12 inches square, should be provided across large (e.g., garage doorway) entrances. The base of the grade beam should be at the same elevation as the bottom of adjoining footings.

3. Unless otherwise superceded by the project structural engineer, concrete slab-on-grade floors directly underlain by dense, unweathered old paralic deposits, recompacted fills, placed in accordance with the recommendations previously provided in this report, or a 2-sack slurry, should be a minimum of 5 inches thick and should be reinforced with No. 3 reinforcing bar at 18 inches on center in both directions. All slab reinforcement should be supported to ensure placement near the vertical midpoint of the concrete. "Hooking" of reinforcement is not considered an acceptable method of positioning the reinforcement. Columns should be structurally isolated from the slab-on-grade floors.

4. The project structural engineer should consider the use of transverse and longitudinal control joints to help control slab cracking due to concrete shrinkage or expansion. Two of the best ways to control this movement are: 1) add a sufficient amount of reinforcing steel to increase the tensile strength of the slab; and 2) provide an adequate amount of control and/or expansion joints to accommodate anticipated concrete shrinkage and expansion. Transverse and longitudinal crack control joints should be spaced no more than 13 feet on center and constructed to a minimum depth of T/4, where "T" equals the slab thickness in inches. Per PCA and ACI guidelines, joints are commonly spaced at distances equal to 24 to 30 times the slab thickness. Joint spacing that is greater than 15 feet require the use of load transfer devices (dowels or diamond plates).

5. Garage slabs should be reinforced as above and poured separately from the structural footings and minimally quartered with expansion joints or saw cuts. A positive separation from the footings should be maintained with expansion joint material to permit relative movement.

6. Slab subgrade pre-saturation is not required for these soil conditions. GSI recommends that the moisture content of the subgrade soils should be equal to, or greater than, optimum moisture content in the slab areas, prior to vapor retarder placement.

7. Soil generated from footing excavations to be used onsite should be moisture conditioned to at least optimum moisture content and compacted to at least 90 percent minimum relative compaction, if it is to be placed in the yard/right-of-way areas. Any fills placed within the building footprint should be moisture conditioned to at least optimum moisture content and compacted to at least 90 or 95 percent minimum relative compaction, depending on if the footing and/or slab-on-grade floor span between the old paralic deposits and the recompacted fill or recompacted fill and slurry. Placed fill materials must not alter positive drainage patterns that direct drainage away from the structural area and toward the street.

SOIL MOISTURE TRANSMISSION CONSIDERATIONS

GSI has evaluated the potential for vapor or water transmission through new concrete floor slabs (if any), in light of typical floor coverings and improvements. Please note that 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 water or vapor through any new foundation or slab-on-grade floors. Foundation systems and slabs shall not allow water or water vapor to enter into the structure 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, 2019). These recommendations may be exceeded or supplemented by a “water proofing” specialist, Project Architect, or Structural Consultant. Thus, the Client will need to evaluate the following in light of a cost vs. benefit analysis (owner expectations and repairs/replacement), along with disclosure to all interested/affected parties. It should also be noted that moisture vapor transmission will occur in new slab-on-grade floors as a result of chemical reactions taking place within the curing concrete. Moisture 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. 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 E.I. test results presented herein, and known soil conditions in the region, the anticipated typical water vapor transmission rates, floor coverings, and improvements (to be chosen by the Client and/or Project Architect) that can tolerate vapor transmission rates without significant distress, the following alternatives are provided:

- New concrete slab-on-grade floors, including garage slabs, should be thicker than 5 inches.

- New concrete slab underlayment should consist of a 15-mil vapor retarder, or equivalent, with all laps sealed per the 2019 CBC and the manufacturer's recommendation. The vapor retarder should comply with the ASTM E 1745 - Class A criteria (i.e., Stego Wrap or approved equivalent), and be installed in accordance with ACI 302.1R-04 and ASTM E 1643.

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

- New concrete slabs, including the garage areas, should be underlain by 2 inches of clean, washed sand (SE $\geq$ 30) above a 15-mil vapor retarder (ASTM E-1745 - Class A, per Engineering Bulletin 119 [Kanare, 2005]) 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-04 (2004) states "If a cushion or sand layer is desired between the vapor retarder and the slab, care must be taken to protect the sand layer from taking on additional water from a source such as rain, curing, cutting, or cleaning. Wet cushion or sand layer has been directly linked in the past to significant lengthening of time required for a slab to reach an acceptable level of dryness for floor covering applications." Therefore, additional observation and/or testing will be necessary for the cushion or sand layer for moisture content, and relatively uniform thicknesses, prior to the placement of concrete.

- For very low expansive soil conditions, the vapor retarder should be underlain by 2 inches of sand (SE $\geq$ 30) placed directly on the prepared, moisture conditioned, subgrade and should be sealed to provide a continuous retarder under the entire slab, as discussed above. The underlying 2-inch sand layer may be omitted provided testing indicates the SE of the slab subgrades soils is greater than or equal to 30.

- Concrete should have a maximum water/cement ratio of 0.50. This does not supercede Table 19.3.2.1 of ACI (2014) for corrosion or other corrosive requirements. Additional concrete mix design recommendations should be provided by the structural consultant and/or waterproofing specialist. Concrete finishing and workablity should be addressed by the Structural Consultant and a waterproofing specialist.

- Where slab water/cement ratios are as indicated herein, and/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 slabs are designed and/or treated for more uniform moisture protection.

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

- Additional recommendations regarding water or 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/moisture vapor transmission through the slab 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 and water-proofing 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 recommendation are for the design and construction of new retaining walls (if proposed). Recommendations for the design and construction of conventional cast-in-place concrete and masonry retaining walls are included herein. Recommendations for specialty walls (i.e., crib, earthstone, geogrid, 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 very low expansive select materials (typically Class 2 permeable filter material or Class 3 aggregate base) or very low expansive native onsite materials are used to backfill any retaining wall (E.I. $\leq$ 20 and P.I. $\leq$ 14). The type of backfill (i.e., select or native), should be specified by the wall designer, and clearly shown on the retaining wall plans. It is currently unknown if the onsite earth materials qualify as select backfill. Thus, additional testing would be necessary if select backfill is used in the design of the retaining walls. The use of waterproofing should be considered for site retaining walls in order to reduce the potential for efflorescence staining.

Preliminary Retaining Wall Foundation Design

Preliminary foundation design for retaining walls should incorporate the following recommendations:

Minimum Footing Embedment - 18 inches below the lowest adjacent grade (excluding any landscape layer [upper 6 inches]) into dense, unweathered old paralic deposits, recompacted fill prepared in accordance with the recommendations in this report, or a 2-sack slurry.

Minimum Footing Width - 24 inches.

Allowable Bearing Pressure - An allowable bearing pressure of 2,500 pcf 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 18 inches into unweathered old paralic deposits, recompacted fill prepared in accordance with the recommendations in this report, or a 2-sack slurry. This pressure may be increased by one-third for short-term wind and/or seismic loads.

Passive Earth Pressure - A passive earth pressure of 250 pcf with a maximum earth pressure of 2,500 psf may be used in the preliminary design of retaining wall foundations provided the foundation is embedded into very low expansive old paralic deposits, recompacted fill (E.I. $\leq$ 20 and P.I. $\leq$ 14) placed per the recommendations in this report, or a 2-sack slurry.

Lateral Sliding Resistance - A 0.35 coefficient of friction may be utilized for a concrete to soil contact when multiplied by the dead load. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.

Backfill Soil Density - Soil densities ranging between 130 pcf and 135 pcf may be used in the design of retaining wall foundations. This assumes an average engineered fill compaction of at least 90 percent of the laboratory standard (ASTM D 1557).

Any retaining wall footings near the perimeter of the site will likely need to be deepened into unweathered old paralic deposits for adequate vertical and lateral bearing support. All retaining wall footing setbacks from slopes should comply with the recommendations previously provided in this report.

Restrained Walls

Any retaining walls that will be restrained prior to placing and compacting backfill material or 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 native backfill, respectively. 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

The recommendations presented below are for cantilevered retaining walls up to about 10 feet high. Design parameters for site walls less than 3 feet in height may be superceded by San Diego Regional Standard Design (SDRSD). However, it should be noted that the design of SDRSD retaining walls requires the use of select backfill materials. 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 where vehicular traffic could occur within horizontal distance "H" from the back of the retaining wall (where "H" equals the wall height). The traffic surcharge may be taken as 100 psf/ft and 300 psf/t in the upper 5 feet of backfill for light passenger vehicles and heavy-axle load (HS20) vehicles, respectively. 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. Equivalent fluid pressures for the design of cantilevered retaining walls are provided in the following table:

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.
(3) E.I. = 0 to 50, SE $\geq$ 30, P.I. < 15, E.I. < 21, and $\leq$ 15% passing No. 200 sieve.

Seismic Surcharge

For retaining walls incorporated into the residential structure, 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, or retaining walls that could present ingress/egress constraints to emergency personnel and equipment in the event of failure, GSI recommends that the walls be evaluated for seismic surcharge in general accordance with 2019 CBC and ASCE 7-10 requirements. The retaining walls in this category should maintain an overturning Factor-of-Safety (FOS) of approximately 1.25 when the seismic surcharge (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. This seismic surcharge pressure (seismic increment) may be taken as 15H where "H" for restrained walls is the dimension previously noted as the height of the backfill to the bottom of the footing. For cantilevered walls, a seismic increment of 15H should be applied as an inverted triangular pressure distribution from 0.6H from the bottom of the footing to the top of the wall. 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. Please note this is for local wall stability only.

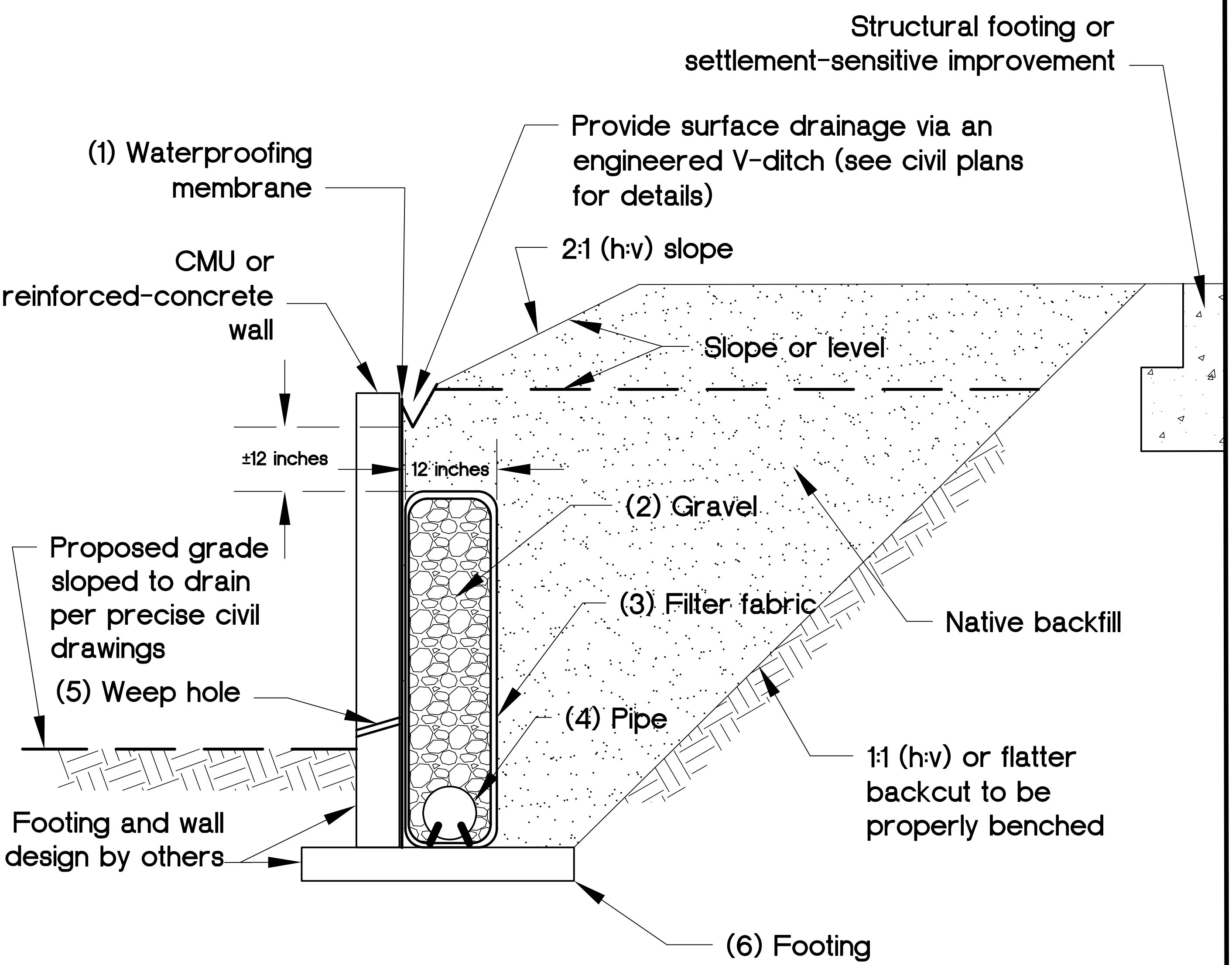
The 15H is derived from a Mononobe-Okabe solution for both restrained cantilever walls. This accounts for the increased lateral pressure due to shakedown or movement of the sand fill soil in the zone of influence from the wall or roughly a 45° - ϕ/2 plane away from the back of the wall. The 15H seismic surcharge is derived from the formula:

$$P_h = \tfrac{3}{8} \bullet a_h \bullet \gamma_t H$$

Where:	P_h	=	Seismic increment
	a_h	=	Probabilistic horizontal site acceleration with a percentage of "g"
	γ_t	=	Total unit weight (120 to 125 pcf for site soils @ 90% relative compaction).
	H	=	Height of the wall from the bottom of the footing or point of pile fixity.

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 pipe encased in either Class 2 permeable filter material or ¾-inch to 1½-inch gravel wrapped in approved filter fabric (Mirafi 140 or equivalent). The backdrain should flow via gravity (minimum 1 percent fall) toward an approved drainage facility. 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 up to E.I. = 20, 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 (Typical Retaining Wall Backfill and Drainage Detail). For limited access and confined areas, (panel) drainage behind the wall may be constructed in accordance with Detail 2 (Retaining Wall Backfill and Subdrain Detail Geotextile Drain). For more onerous expansive situations, backfill and drainage behind the retaining wall should conform with Detail 3 (Retaining Wall And Subdrain Detail Clean Sand Backfill). 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 onerous wall design will be necessary. Retaining wall backfill should be moisture conditioned to at least optimum moisture content, placed in



(1) Waterproofing membrane.

(2) Gravel: Clean, crushed, $\frac{3}{4}$ to $1\frac{1}{2}$ inch.

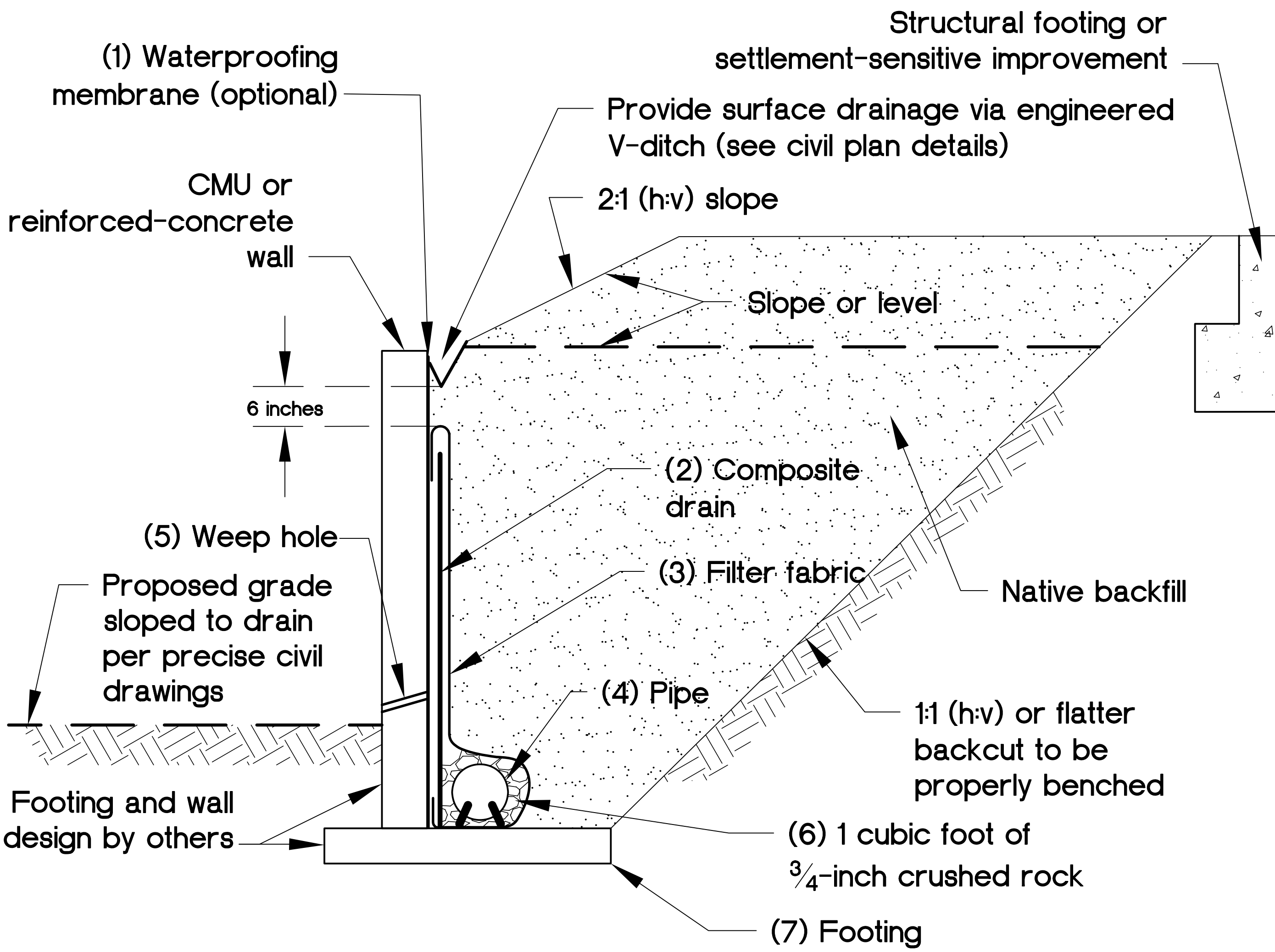
(3) Filter fabric: Mirafi 140N or approved equivalent.

(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 hole: Minimum 2-inch diameter placed 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, use level fill or cut natural earth materials. An additional "heel" drain will likely be required by geotechnical consultant.

GeoSoils, Inc.	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.

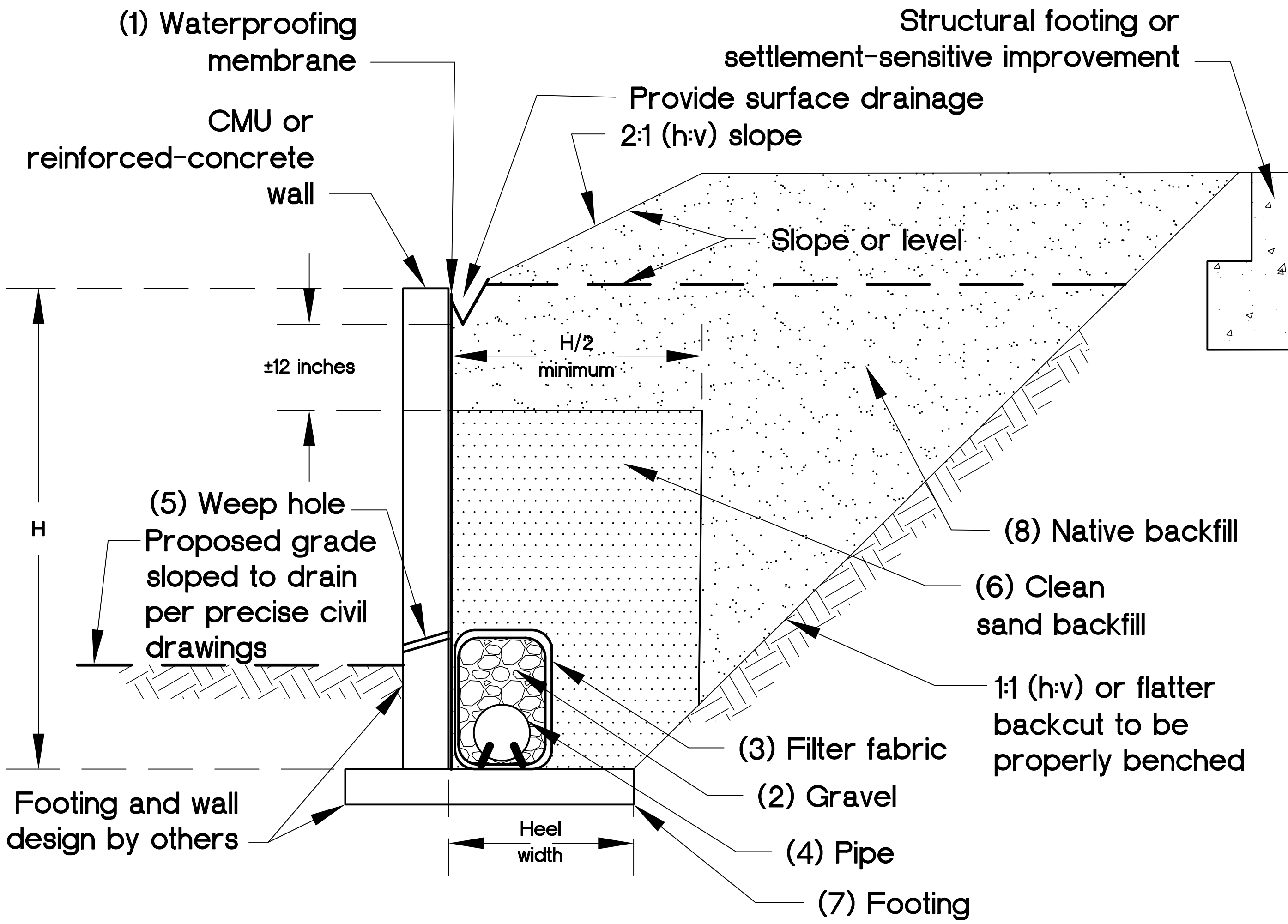
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(5) Weep hole: Minimum 2-inch diameter placed 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, use 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, $\frac{3}{4}$ to $1\frac{1}{2}$ 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: Minimum 2-inch diameter placed 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, use 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. ≥ 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

relatively thin lifts, and compacted to at least 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.

Outlets should consist of a 4-inch diameter solid PVC or ABS pipe spaced no greater than $\pm$100 feet apart, with a minimum of two outlets, one on each end. The use of weep holes, only, in walls higher than 2 feet, is not recommended. The surface of the backfill should be sealed by pavement or the top 18 inches compacted with native soil (E.I. $\leq$ 50). 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 cut to fill or fill to slurry, the wall designer may specify either:

a) A minimum of a 2-foot overexcavation and recompaction of cut materials for a distance of 2H, from the point of transition.

b) Increase of the amount of reinforcing steel and wall detailing (i.e., expansion joints or crack control joints) such that a 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 native formational material (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.

SHORING DESIGN AND CONSTRUCTION

Shoring of Excavations

Shoring may be necessary to support existing improvements and adjacent property during construction. If required, excavations should be retained either by a cantilever or braced shoring system deriving passive support from cast-in-place soldier piers and lagging (pile and lagging-shoring system), as needed. A restrained shoring system, using tiebacks or soil nails, may not be appropriate for this site due to the potential for the proximity of neighboring properties. Based on our experience with similar sites, if the shoring system cannot tolerate lateral movement on the order of 1 to 2 inches, we recommend the utilization of an internal braced (i.e., raked) shoring system. <u>Due to the limited working space within the property, shoring installation may be challenging.</u>

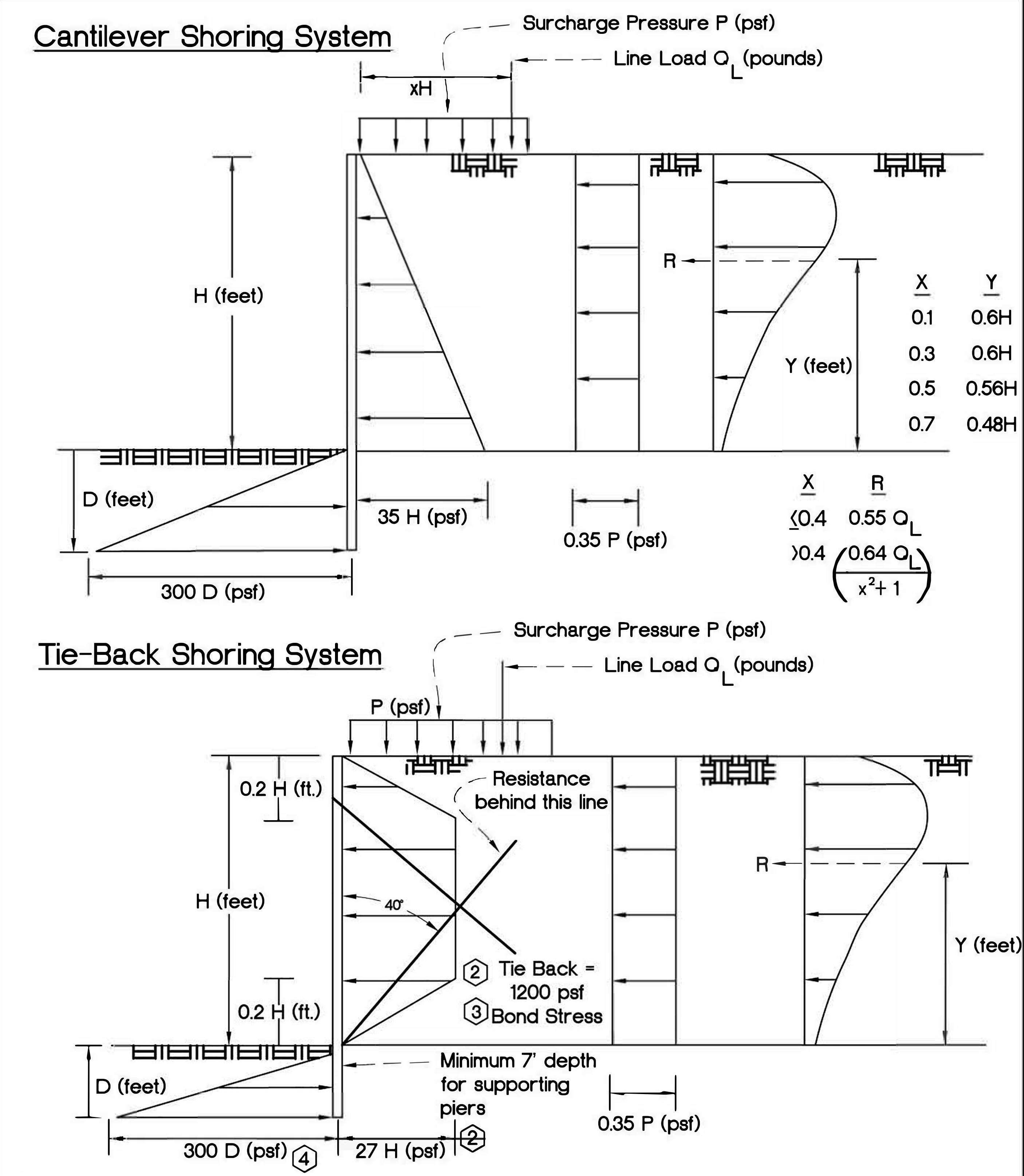
Shoring of excavations of this size is typically performed by specialty contractors with knowledge of the City of Encinitas ordinances as well as, the local soil conditions. In general, local shoring contractors are design-build entities that provide both the shoring design and installation.

Since the design of retaining systems is sensitive to surcharge pressures behind the excavation, we recommend that this office be consulted if unusual load conditions are anticipated. Care should be exercised when excavating into the on-site soils since caving or sloughing of these materials is possible. Observation and geologic logging of soldier pile excavations should be performed by the geotechnical consultant during construction.

Shoring of the excavation is the responsibility of the contractor. Extreme caution should be used to minimize damage to existing improvements, underground utilities, and/or structures caused by settlement or reduced lateral support. Accordingly, we recommend that the adjacent improvements/structures be surveyed prior to and during shoring construction to evaluate the effects of the shoring on these structures. Photo-documentation is also advisable. Unless incorporated into the shoring design, construction equipment storage or traffic, and/or soil or construction material stockpiles should not be operated or stored within "H" feet of the top of any shored excavations (where "H" equals the height of the retained earth). Temporary/permanent provisions should be made to direct any potential runoff away from the top of shored excavations. All applicable surcharges from vehicular traffic and existing structures within "H" of a shored excavation should be evaluated.

<u>Lateral Pressure</u>

The active and passive pressures to be utilized for temporary shoring designs are provided in Figure 9. These criteria assume level backfill conditions within 2H of the shoring wall (where "H" equals the height of the retained soils), and that hydrostatic pressure is not allowed to build up behind excavation walls. The recommendations for shoring design assume excavations up to about 13 feet high. A braced system may allow for deeper excavation. However, this would be at the sole discretion of the shoring designer. An empirical equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. This does <u>not</u> include other superimposed loading conditions such as traffic, structures, seismic events, or adverse geologic conditions. For temporary shoring walls greater than 4 feet in height that will be exposed for a duration greater than 6 months, a seismic increment of 15H (uniform pressure [psf/ft]) should be considered in the design for level backfill conditions. The seismic load should be applied as an inverted triangular pressure distribution at 0.6H up from the point of fixity to the top of the retained earth materials. Traffic surcharge from light passenger vehicles (if any) should be minimally applied as 100 psf per foot of depth (psf/ft) in the upper 5 feet of the shoring if traffic is within "H" of the back of the shoring, where "H" equals the height of the



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 9

W.O. 7638-A2-SC	DATE 05/21	SCALE None

retained soils. For heavy-axle (HS20) vehicle loads, the surcharge should be minimally applied as 300 psf/ft in the upper 5 feet of the shoring if traffic is within “H” of the back of the shoring, where “H” equals the height of the retained soils. Due to the effects of soil movements on nearby improvements, including underground utilities, shoring lagging should be designed using a K_o value of 0.5 and a maximum value of 300 pounds per lineal foot

When considering nearby improvements located above a 1:1 (h:v) plane projected up from the toe of the shored excavation, GSI recommends that a Boussinesq approach be used in evaluating surcharge. This approach may model the loads for an estimated vertical load of a footing, and may be used as a line load on the back of the shoring. A seismic increase of the footing by one-third of the estimated bearing load should be considered for footing surcharge on shoring walls.

Shoring Construction Recommendations

1. The excavation and installation of the soldier piles should be observed and documented by the Project Geotechnical Engineer and Special Inspector to further evaluate the geologic conditions within the influence of the shoring wall 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. Due to the possible presence of cobbles, the contractor should periodically recheck the drilled shaft for plumbness.

3. Although unlikely, casing should be provided in the drilled excavations if significant seepage and/or sloughing/caving are encountered. The bottom of the casing should be at least 4 feet below the top of the concrete as the concrete is poured and the casing is withdrawn. Also unlikely, dewatering may be required for concrete placement if significant seepage is encountered during excavation. 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. The concrete mix design and the corrosion protection of reinforcing steel should be in accordance with the shoring designer’s specifications. All concrete should be delivered through a tremie pipe. We recommend that concrete be placed through the tremie pipe immediately subsequent to approval of the excavation and reinforcing steel placement. The costs for cleaning and/or redrilling soldier pile excavations in the event that concrete is not immediately placed after excavation approval and steel placement shall be the sole responsibility of the contractor. Care should be taken to prevent striking the walls of the excavations with the tremie pipe during concrete placement. Vibration of concrete to reduce the potential for segregation should be performed.

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

7. Concrete used in the shoring construction should be tested by a qualified materials testing consultant for strength and mix design.

8. Excavation for lagging should not commence until the soldier pile concrete reaches its 28-day compressive strength.

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 designer should be notified of any unusual conditions encountered during installation.

<u>Monitoring of Shoring</u>

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

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

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

4. Initial monitoring and video-/photo-documentation should be performed prior to any excavation.

5. Once the excavation has commenced, periodic readings should be taken weekly until the site has been brought to the design grades. If the monitoring data shows that the performance of the shoring system is 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 engineer in writing, and should be distributed to the geotechnical engineer, structural engineer, civil engineer, and the shoring contractor. Once initiated, bi-weekly readings should continue until the site has been brought to the design grades. Thereafter, readings can be made monthly at the discretion of the shoring designer. Additional readings should be taken when requested by the special inspector, the shoring designer, structural engineer, geotechnical engineer, or the building official.

6. Monitoring reading should be submitted to the shoring engineer and geotechnical engineer within three business days after they are conducted. Monitoring readings should be accurate to within 0.01 feet. Results are to be submitted in tabular form showing at least the initial date of monitoring and reading, current monitoring date and reading and difference between the two readings.

7. If the total cumulative horizontal or vertical movement (from start of shoring construction, base reading) of nearby improvements and soldier piles reaches ½ inch and 1 inch, respectively, all excavation activities should be suspended until the geotechnical engineer and shoring 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 engineer and shoring engineer/designer.

Monitoring of Structures

1. The contractor should complete a written and photographic log of existing improvements located within 100 feet or three times the depth of the shoring (whichever is greater) prior to shoring construction. A licensed surveyor should document all existing substantial cracks (i.e., greater than ⅛ inch horizontal or vertical separation) within the City of Encinitas right-of-way (ROW).

2. The contractor should also document the existing condition of the City of Encinitas ROW prior to the start of shoring construction. This should include underground utilities within Neptune Avenue or ROW that are within that influence of the grading and shoring.

3. The contractor should monitor all existing improvements for movement or cracking that may result from the onsite excavations.

4. If excessive movement or visible cracking occurs, the shoring contractor should stop work and shore/reinforce the excavation, and contact the shoring engineer and the geotechnical engineer.

5. Monitoring of the existing building or adjacent structures should be made at reasonable intervals as required by the shoring engineer. Monitoring should be performed by a licensed surveyor. All readings and measurements should be submitted to the shoring engineer and geotechnical engineer.

6. Prior to commencing shoring construction, a pre-construction meeting should take place between the contractor, shoring engineer, surveyor, geotechnical engineer. The monitoring locations on existing offsite improvements should be identified during the meeting.

Construction Monitoring/Excavation Observation (All Excavations)

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

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 the existing structure. Secondly, the responsibility for damage to the existing structure can be determined 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 both the existing improvements and the shoring and/or bracing. Locations and type of the monitoring devices should be selected as soon as the total shoring system is designed and approved. The program of monitoring should be agreed upon between the project team, the site surveyor and the geotechnical engineer of record, prior to excavation. Exact locations of reference points may be dictated by surface points on roadways and sidewalks near the top of the excavation. The points on the shoring should be placed under or very near the points on the existing improvements.

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 backfill has been brought up to final 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 and at critical times during the installation of shoring or support. The reading should be plotted by the surveyor and then reviewed by the geotechnical engineer.

In addition to the monitoring system, it would be prudent for the geotechnical engineer and the contractor to make a complete inspection of the existing structure(s) both before and after construction. The inspection should be directed toward detecting any signs of damage, particularly those caused by settlement. Notes, pictures, and/or video documentation should be made should be made where necessary.

Observation

It is recommended that all excavations be observed by the engineering geologist or geotechnical engineer. Should the observation reveal any unforseen hazard, the engineering geologist or geotechnical engineer will recommend treatment. Please inform us at least 24 hours prior to any required site observation.

TOP-OF-SLOPE WALLS/FENCES/IMPROVEMENTS

Slope Creep

Although unlikely, some soils at the site may be expansive and therefore, may become desiccated when allowed to dry. Such soils are susceptible to surficial slope creep, especially with seasonal changes in moisture content. Typically in southern California, during the hot and dry summer period, these soils become desiccated and shrink, thereby developing surface cracks. The extent and depth of these shrinkage cracks depend on many factors such as the nature and expansivity of the soils, temperature and humidity, and extraction of moisture from surface soils by plants and roots. When seasonal rains occur, water percolates into the cracks and fissures, causing slope surfaces to expand, with a corresponding loss in soil density and shear strength near the slope surface. With the passage of time and several moisture cycles, the outer 3 to 5 feet of slope materials experience a very slow, but progressive, outward and downward movement, known as slope creep. For slope heights greater than 10 feet, this creep related soil movement will typically impact all rear yard flatwork and other secondary improvements that are located within about 15 feet from the top of slopes, such as swimming pools, concrete flatwork, etc., and in particular top of slope fences/walls. This influence is normally in the form of detrimental settlement, and tilting of the proposed improvements. The dessication/swelling and creep discussed above continues over the life of the improvements, and generally becomes progressively worse. Accordingly, the owner/developer should provide this information to all interested/affected parties.

Top of Slope Walls/Fences

Due to the potential for slope creep for slopes higher than about 10 feet, some settlement and tilting of the walls/fence with the corresponding distresses, should be expected. To mitigate the tilting of top of slope walls/fences, we recommend that the walls/fences be constructed on a combination of grade beam and caisson foundations with creep forces taken into account. The grade beam should be at a minimum of 12 inches by 12 inches in cross-section, supported by drilled caissons, 12 inches minimum in diameter, placed at a maximum spacing of 6 feet on center, and with a minimum embedment length of 7 feet below the bottom of the grade beam. The strength of the concrete and grout should be evaluated by the structural engineer of record. The proper ASTM tests for the concrete and mortar should be provided along with the slump quantities. The concrete used should be appropriate to mitigate sulfate corrosion, as warranted. The design of the grade beam and caissons should be in accordance with the recommendations of the project structural engineer, and include the utilization of the following geotechnical parameters:

Creep Zone: 5-foot vertical zone below the slope face and projected upward parallel to the slope face.

Creep Load: The creep load projected on the area of the grade beam should be taken as an equivalent fluid approach, having a density of 60 pcf. For the caisson, it should be taken as a uniform 900 pounds per linear foot of caisson's depth, located above the creep zone.

Point of Fixity: Located a distance of 1.5 times the caisson's diameter, below the creep zone.

<u>Passive Resistance</u>: Passive earth pressure of 250 psf per foot of depth per foot of caisson diameter, to a maximum value of 2,500 psf may be used to determine caisson depth and spacing, provided that they meet or exceed the minimum requirements stated above. To determine the total lateral resistance, the contribution of the creep prone zone above the point of fixity, to passive resistance, should be disregarded.

<u>Allowable Axial Capacity</u>:

Shaft capacity: 300 psf applied below the point of fixity over the surface area of the shaft.

Tip capacity: 3,000 psf in approved compacted fill or unweathered formational materials.

DRIVEWAY, FLATWORK, AND OTHER IMPROVEMENTS

Although unlikely, some of the soil materials on site may be expansive. The effects of expansive soils are cumulative, and typically occur over the lifetime of any improvements. On relatively level areas, when the soils are allowed to dry, the dessication 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 owner notify all interested/affected parties of this long-term potential for distress. To reduce the likelihood of distress, the following recommendations are presented for all exterior flatwork:

1. The subgrade area for concrete slabs should be compacted to achieve a minimum 90 percent relative compaction, and then be presoaked to 2 to 3 percentage points above (or 125 percent of) the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. If very low expansive soils are present, only optimum moisture content, or greater, is required and specific presoaking is not warranted. The moisture content of the subgrade should be proof tested within 72 hours prior to pouring concrete.

2. Concrete slabs 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 pouring concrete. If very low expansive soils are present, the rock or gravel or sand may be deleted. The layer or subgrade should be wet-down completely prior to pouring concrete, to minimize loss of concrete moisture to the surrounding earth materials.

3. Exterior slabs should be a minimum of 4 inches thick. Driveway slabs and approaches should additionally have a thickened edge (12 inches) adjacent to all landscape areas, to help impede infiltration 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 and/or expansion joints to accommodate anticipated concrete shrinkage and expansion.

   In order to reduce the potential for unsightly cracks, non-residential/garage slabs should be reinforced at mid-height with a minimum of No. 3 bars placed at 18 inches on center, in each direction. If subgrade soils within the top 7 feet from finish grade are very low expansive soils (i.e., E.I. $\leq$ 20 and P.I. $\leq$ 14), then 6x6-W1.4xW1.4 welded-wire mesh may be substituted for the rebar, provided the reinforcement is placed on chairs, at slab mid-height. 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 6 feet. The slabs should be separated from the foundations and sidewalks with expansion joint filler material.

5. No traffic should be allowed upon the newly poured concrete slabs until they have been properly cured to within 75 percent of design strength. Concrete compression strength should be a minimum of 2,500 psi.

6. The use of thick expansion joint filler material should be used to separate driveways, sidewalks, and patio slabs from the house. 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 structurally tied to the house.

8. Overhang structures should be supported on the slabs, or structurally designed with continuous footings tied in at least two directions. If very low expansion soils are present, footings need only be tied in one direction.

9. Any masonry landscape 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. Utilities should be enclosed within a closed utilidor (vault) or designed with flexible connections to accommodate differential settlement and expansive soil conditions.

11. 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 Neptune Avenue, as indicated herein. 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 homeowner.

12. Air conditioning (A/C) units should be supported by 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-erosive outlet.

13. 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.

ONSITE INFILTRATION-RUNOFF RETENTION SYSTEMS

General

Owing to relatively recent requirements by the State Water Resources Control Board, onsite infiltration-runoff retention systems (OIRRS) may be necessary to address Storm Water Best Management Practices (BMPs) or Low Impact Development (LID) principles for the project. To that end, some guidelines should/must be followed in the planning, design, and construction of such systems. Such facilities, if improperly designed or implemented without consideration of the geotechnical aspects of site conditions, can contribute to flooding, saturation of bearing materials beneath site improvements, slope instability, and possible concentration and contribution of pollutants into the groundwater or storm drain and/or utility trench systems.

A key factor in these systems is the infiltration rate (often referred to as the percolation rate) which can be ascribed to, or determined for, the earth materials within which these systems are installed. Additionally, the infiltration rate of the designed system (which may include gravel, sand, mulch/topsoil, or other amendments, etc.) will need to be considered. The project infiltration testing is very site specific, any changes to the location of the proposed OIRRS and/or estimated size of the OIRRS, may require additional infiltration testing. GSI anticipates that relatively impermeable old paralic deposits will occur near the surface at the conclusion of the currently proposed site development.

Some of the methods which are utilized for onsite infiltration include percolation basins, dry wells, bio-swale/bio-retention, permeable pavers/pavement, infiltration trenches, filter boxes and subsurface infiltration galleries/chambers. Some of these systems are constructed using native and import soils, perforated piping, and filter fabrics while others employ structural components such as stormwater infiltration chambers and filters/separators. Every site will have characteristics which should lend themselves to one or more of these methods, but not every site is suitable for OIRRS. In practice, OIRRS are usually initially designed by the project design civil engineer. Selection of methods should include (but should not be limited to) review by licensed professionals including the geotechnical engineer, hydrogeologist, engineering geologist, project civil engineer, landscape architect, environmental professional, and industrial hygienist. Applicable governing agency requirements should be reviewed and included in design considerations.

The following geotechnical guidelines should be considered when designing onsite infiltration-runoff retention systems:

- Based on our review of the United States Department of Agriculture Soil Survey (http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx),the onsite soil consist of the Marina loamy coarse sand, 9 to 30 percent slopes. The most limiting layer to transmit water (Ksat) of this soil is characterized as moderately high to high (0.57 inches per hour [in/hr] to 1.98 in/hr). This soil falls into Hydrologic Soil Group (HSG) "B." According to County of San Diego (2007), HSG "B" soils have moderate infiltration rates. However, the underlying old paralic deposits likely have slower infiltration rates, on a preliminary basis. Full or partial infiltration into the onsite soils may be possible, but is not recommended owing to it potential effects on the stability of the coastal bluff.

- Infiltration of stormwater into the onsite soils is not advised from a geotechnical perspective, as such practice could negatively affect the stability of the coastal bluff. Thus, if permanent storm water BMPs/LIDs are necessary, we recommend that storm water treatment be performed in lined and drained bio-retention basins constructed near the easterly property line. Stormwater basins should not extend below a 1:1 (h:v) plane projected down from the bottom, outboard edge of existing and proposed improvements, or adjacent property.

- Impermeable liners and subdrains should be used along the bottom of bio-retention swales/basins located within the influence of slopes or other settlement-sensitive improvements. The liner should extend a few inches above the 100-year (Q_{100}) water elevation inside the basin and should be covered by a minimum 12-inches of soil absent of rock fragments and angular grains. Impermeable liners should consist of a 30-mil polyvinyl chloride (PVC) membrane with the following 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) 2.6 (kN/m).

- Subdrains should consist of at least 4-inch diameter Schedule 40 or SDR 35 drain pipe with perforations oriented down. The drain pipe should be sleeved with a filter sock. The subdrain should be directed toward an approved drainage facility identified by the Project Civil Engineer.

- If landscaping is proposed <u>within</u> the bio-retention basin, consideration should be given to the type of vegetation chosen since some trees/shrubs could damage the liner and adversely affect adjacent surface improvements). Over-watering landscape areas above, or adjacent to, the proposed bio-retention basin could adversely affect performance of the system.

- Areas adjacent to, or within, the bio-retention basin that are subject to inundation should be properly protected against scouring, undermining, and erosion, in accordance with the recommendations of the design engineer.

- Seismic shaking may result in the formation of a seiche which could potential overtop the banks of the bioretention basin and result in down-gradient flooding and scour.

- As with any storm water LID/BMP, proper care will need to be provided. Best management practices should be followed at all times, especially during inclement weather. Provisions for the management of any siltation, debris within the OIRRS, and/or overgrown vegetation (including root systems) should be considered. An appropriate inspection schedule will need to adopted and provided to all interested/affected parties.

- Any designed system will require regular and periodic maintenance, which may include rehabilitation and/or complete replacement of the filter media (e.g., sand, gravel, filter fabrics, topsoils, mulch, etc.) or other components utilized in construction, so that the design life exceeds 15 years. Due to the potential for piping and adverse seepage conditions, a burrowing rodent control program should also be implemented onsite.

- All or portions of these systems may be considered attractive nuisances. Thus, consideration of the effects of, or potential for, vandalism should be addressed.

- Newly established vegetation/landscaping (including phreatophytes) may have root systems that will influence the performance of the bio-retention basin.

- The potential for surface flooding, in the case of system blockage, should be evaluated by the design engineer.

- Any proposed utility backfill materials (i.e., inlet/outlet piping and/or other subsurface utilities) located within or near the proposed area of the bio-retention basin may become saturated. This is due to the potential for piping, water migration, and/or seepage along the utility trench line backfill. If utility trenches cross and/or are proposed near the bio-retention basin, cut-off walls or other water barriers will need to be installed to mitigate the potential for piping and excess water entering the utility backfill materials. Planned or existing utilities may also be subject to piping of fines into open-graded gravel backfill layers unless separated from the overlying or adjoining bio-retention basin by geotextiles and/or slurry backfill.

- The use of storm water LID/BMPs above existing utilities that might degrade/corrode with the introduction of water/seepage should be avoided.

- A vector control program may be necessary as stagnant water contained in the bio-retention basin may attract mammals, birds, and insects that carry pathogens.

DEVELOPMENT CRITERIA

Slope Maintenance and Planting

Water has been shown to weaken the inherent strength of all earth materials. Slope stability is significantly reduced by overly wet conditions. Positive surface drainage away from slopes and the coastal bluff should be maintained and only the amount of irrigation necessary to sustain plant life should be provided for planted slopes. Over-watering should be avoided as it adversely affects site improvements, and causes perched groundwater conditions. Graded slopes constructed utilizing onsite materials would be erosive. Eroded debris may be minimized and surficial slope stability enhanced by establishing and maintaining a suitable vegetation cover soon after construction. Compaction to the face of fill slopes would tend to minimize short-term erosion until vegetation is established. Plants selected for landscaping should be light weight, deep rooted types that require little water and are capable of surviving the prevailing climate. Jute-type matting or other fibrous covers may aid in allowing the establishment of a sparse plant cover. Utilizing 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 (ungraded) slope areas is generally not recommended. These recommendations regarding plant type, irrigation practices, and rodent control should be provided to the homeowner. Over-steepening of slopes should be avoided during building construction activities and landscaping.

Drainage

Adequate lot surface drainage is a very important factor in reducing the likelihood of adverse performance of foundations, hardscape, and slopes. Surface drainage should be sufficient to prevent ponding of water anywhere on a lot, and especially near structures and tops of slopes/top of bluff. 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 flow uncontrolled down any descending slope/bluff. Water should be directed away from foundations and not allowed to pond and/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 percent sloping away from structures, 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 utilized 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

Cut and fill slopes will be subject to surficial erosion during and after grading. 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 Maintenance and Design Considerations

Only the amount of irrigation necessary to sustain plant life should be provided. Over-watering the landscape areas will adversely affect proposed site improvements. We would 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 utilized. 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. Graded slope areas should be planted with drought resistant vegetation. 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.

Gutters and Downspouts

As previously discussed in the drainage section, the installation of gutters and downspouts should be considered to collect roof water that may otherwise infiltrate the soils adjacent to the structures. If utilized, the downspouts should be drained into PVC collector pipes or other non-erosive devices (e.g., paved swales or ditches; below grade, solid tight-lined PVC pipes; etc.), that will carry the water away from the house, to an appropriate outlet, in accordance with the recommendations of the design civil engineer. Downspouts and gutters are not a requirement; however, from a geotechnical viewpoint, provided that positive drainage is incorporated into project design (as discussed previously).

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 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 in the future, any additional improvements (e.g., pools, spas, etc.) are planned for the site, recommendations concerning the geological or geotechnical aspects of design and construction of said improvements could be provided upon request. Pools and/or spas should not be constructed without specific design and construction recommendations from GSI, and this construction recommendation should be provided to all interested parties/affected parties. 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, utility trench and retaining wall backfills, flatwork, etc.

Tile Flooring

Tile flooring can crack, reflecting cracks in the concrete slab below the tile, although small cracks in a conventional slab may not be significant. Therefore, the designer should consider additional steel reinforcement for concrete slabs-on-grade 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 backfilling after rough grading has been completed. This includes completion of grading in the street, driveway approaches, driveways, parking areas, and utility trench and retaining wall backfills.

Footing Trench Excavation

All footing excavations should be observed by a representative of this firm subsequent to trenching and prior to concrete form and 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 excavation, a deeper footing or removal and recompaction of the subgrade materials would be recommended at that time. Footing trench spoil and any excess soils generated from utility trench excavations should be compacted to a minimum relative compaction of 90 percent, if not removed from the site.

Trenching/Temporary Construction Backcuts

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 [except as specifically superceded within the text of this report]), should be anticipated. All excavations should be observed by an engineering geologist or soil 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/or subcontractors, or homeowners, etc., that may perform such work.

Utility Trench Backfill

1. All interior utility trench backfill should be brought to at least 2 percent above optimum moisture content and then compacted to obtain a minimum relative compaction 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 utilized and jetted or flooded into place. Observation, probing and testing should be provided to evaluate the desired results.

2. Exterior trenches adjacent to, and within areas extending below a 1:1 plane projected from the outside bottom edge of the footing, and all trenches beneath hardscape features and in slopes, should be compacted to at least 90 percent of the laboratory standard. Sand backfill, unless excavated from the trench, should not be used in these backfill areas. Compaction testing and observations, along with probing, should be accomplished to evaluate the desired results.

3. All trench excavations should conform to CAL-OSHA, state, and local safety codes. Utilities crossing grade beams, perimeter beams, or footings should either pass below the footing or grade beam utilizing 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/or testing be performed by GSI at each of the following construction stages:

- During grading/recertification, including remedial earthwork.
- During excavation.
- During placement of subdrains or other subdrainage devices, prior to placing fill and/or backfill.
- After excavation of building footings, retaining wall footings, and free standing walls footings, prior to the placement of reinforcing steel or concrete.
- Prior to pouring any slabs or 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 barriers (e.g., visqueen, etc.).
- During placement of backfill for area drain, interior plumbing, underground utility line trenches, and retaining wall backfill.
- 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 developer or homeowner improvements, such as flatwork, spas, pools, walls, etc., 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/or 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/or improvement's designer should confirm to GSI and the governing agency, in writing, that the proposed foundations and/or improvements can tolerate the amount of differential settlement and/or 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 and/or further geotechnical studies may be warranted.

LIMITATIONS

The materials encountered on the project site and utilized 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

TEST PIT AND HAND-AUGER BORING LOGS
AND GSI (2000) BORING LOG

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.

W.O. 7638-A-SC
Bevand
312 Neptune Avenue, Encinitas
Logged By: RBB
October 3, 2019

LOG OF EXPLORATORY TEST PIT

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-1	±86	0-½	SP				**<u>ARTIFICIAL FILL - UNDOCUMENTED:</u>** SAND, brownish gray, dry, loose; fine to medium grained, abundant roots, fill is up to approximately ¾-foot thick at westerly side of test pit.
		½-¾	SP				**<u>WEATHERED QUATERNARY OLD PARALIC DEPOSITS</u>**: SAND, dark yellowish brown, dry, medium dense; fine to medium grained trace roots.
		¾-3¼	SP	UND @ 1 UND @ 2 SM BAG @ 3-3¼	5.2 5.1	112.1 105.7	**<u>QUATERNARY OLD PARALIC DEPOSITS:</u>** SAND, dark yellowish brown and reddish yellow, dry to damp, dense to very dense; fine to medium grained, moderately cemented.
	UND - Relatively Undisturbed Ring Sample SM BAG = Small Bag Sample						Total Depth = 3¼' No Groundwater/Caving encountered Existing footing is embedded approximately 1¼ feet below the existing grade. Backfilled 10/3/19

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LOG OF EXPLORATORY HAND-AUGER BORING

HAND-AUGER NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
HA-1	±76	0-½	SP				**ARTIFICIAL FILL - UNDOCUMENTED:** SAND, dark gray and light gray, damp, loose; fine to medium grained, trace SILT, abundant roots.
		½-1¼	SM	UND @ 1	5.6	104.7	**WEATHERED QUATERNARY OLD PARALIC DEPOSITS**: SILTY SAND, light brown, dry, medium dense; fine to medium grained trace coarse grains.
		1¼-3¼	SM	SM BAG @ 1¼-3½			**QUATERNARY OLD PARALIC DEPOSITS:** SILTY SAND, reddish yellow, damp, dense; fine to medium grained, trace coarse grains, trace clay.
	UND - Relatively Undisturbed Ring Sample SM BAG = Small Bag Sample						Total Depth = 3½ No Groundwater/Caving Encountered Backfilled 10/3/19

GeoSoils, Inc.

W.O. 7638-A-SC
Bevand
312 Neptune Avenue, Encinitas
Logged By: RBB
October 3, 2019

LOG OF EXPLORATORY HAND-AUGER BORING

HAND-AUGER NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
HA-2	±84	0-¾	SP				**ARTIFICIAL FILL - UNDOCUMENTED:** SAND, dark gray and light gray, dry, very loose; fine to medium grained, trace SILT.
		¾-3¼	SP				SAND, brown, dry, loose; fine to medium grained.
		3¼-6⅓	SM	SM BAG @ 3¼-6⅓			SILTY SAND, variegated reddish brown and light brownish gray, moist, medium dense; fine to medium grained.
	SM BAG = Small Bag Sample						Total Depth = 6⅓' No Groundwater/Caving Encountered Backfilled 10/3/19

BORING LOG

GeoSoils, Inc.

W.O. 2866-A-SC

PROJECT: REFOLD
330 Neptune Avenue

BORING HA-1 SHEET 1 OF 1

DATE EXCAVATED 4-19-00

SAMPLE METHOD: Hand Auger

Standard Penetration Test

Undisturbed, Ring Sample

Water Seepage into hole

Depth (ft.)	Sample: Bulk	Sample: Undis-turbed	Sample: Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
				SM				**ARTIFICIAL FILL** @ 0-4', SILTY SAND, brown, moist to wet, loose; fine grained, well sorted, sub-rounded, roots and rootlets present.
5								Total Depth = 4' No groundwater encountered Backfilled 04-19-00
10								
15								
20								
25								

APPENDIX C

SEISMICITY DATA

```
***********************
*                     *
*    E Q F A U L T    *
*                     *
*    Version 3.00     *
*                     *
***********************
```

DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 7638-A-SC

DATE: 10-30-2019

JOB NAME: BEVAND

CALCULATION NAME: 7638

FAULT-DATA-FILE NAME: C:\Program Files\EQFAULT1\CGSFLTE.DAT

SITE COORDINATES:
SITE LATITUDE: 33.0545
SITE LONGITUDE: 117.3005

SEARCH RADIUS: 62.2 mi

ATTENUATION RELATION: 11) Bozorgnia Campbell Niazi (1999) Hor.-Pleist. Soil-Cor.
UNCERTAINTY (M=Median, S=Sigma): S Number of Sigmas: 1.0
DISTANCE MEASURE: cdist
SCOND: 0
Basement Depth: 5.00 km Campbell SSR: 0 Campbell SHR: 0
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\Program Files\EQFAULT1\CGSFLTE.DAT

MINIMUM DEPTH VALUE (km): 3.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

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	3.2(5.1)	7.2	0.738	XI
NEWPORT-INGLEWOOD (Offshore)	10.4(16.7)	7.1	0.378	IX
CORONADO BANK	17.7(28.5)	7.6	0.320	IX
ELSINORE (JULIAN)	27.9(44.9)	7.1	0.148	VIII
ELSINORE (TEMECULA)	27.9(44.9)	6.8	0.121	VII
PALOS VERDES	40.3(64.8)	7.3	0.116	VII
ELSINORE (GLEN IVY)	40.8(65.7)	6.8	0.081	VII
SAN JOAQUIN HILLS	42.0(67.6)	6.6	0.098	VII
EARTHQUAKE VALLEY	42.6(68.6)	6.5	0.063	VI
SAN JACINTO-ANZA	50.7(81.6)	7.2	0.085	VII
SAN JACINTO-SAN JACINTO VALLEY	52.3(84.2)	6.9	0.067	VI
NEWPORT-INGLEWOOD (L.A.Basin)	52.6(84.6)	7.1	0.077	VII
SAN JACINTO-COYOTE CREEK	53.7(86.4)	6.6	0.053	VI
ELSINORE (COYOTE MOUNTAIN)	54.7(88.1)	6.8	0.060	VI
CHINO-CENTRAL AVE. (Elsinore)	55.1(88.6)	6.7	0.078	VII
WHITTIER	59.0(94.9)	6.8	0.055	VI

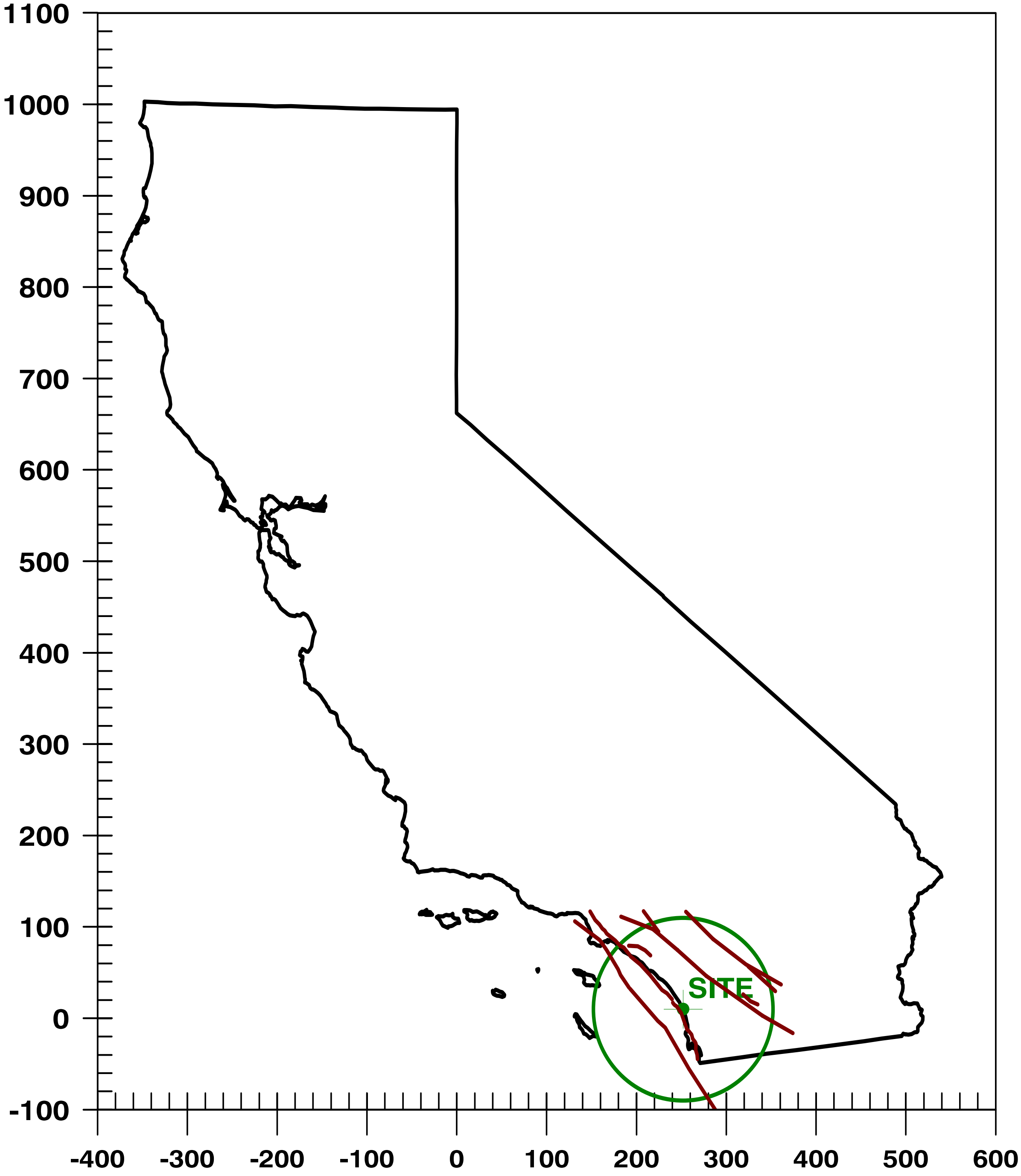
-END OF SEARCH- 16 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE ROSE CANYON FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 3.2 MILES (5.1 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.7376 g

CALIFORNIA FAULT MAP

BEVAND

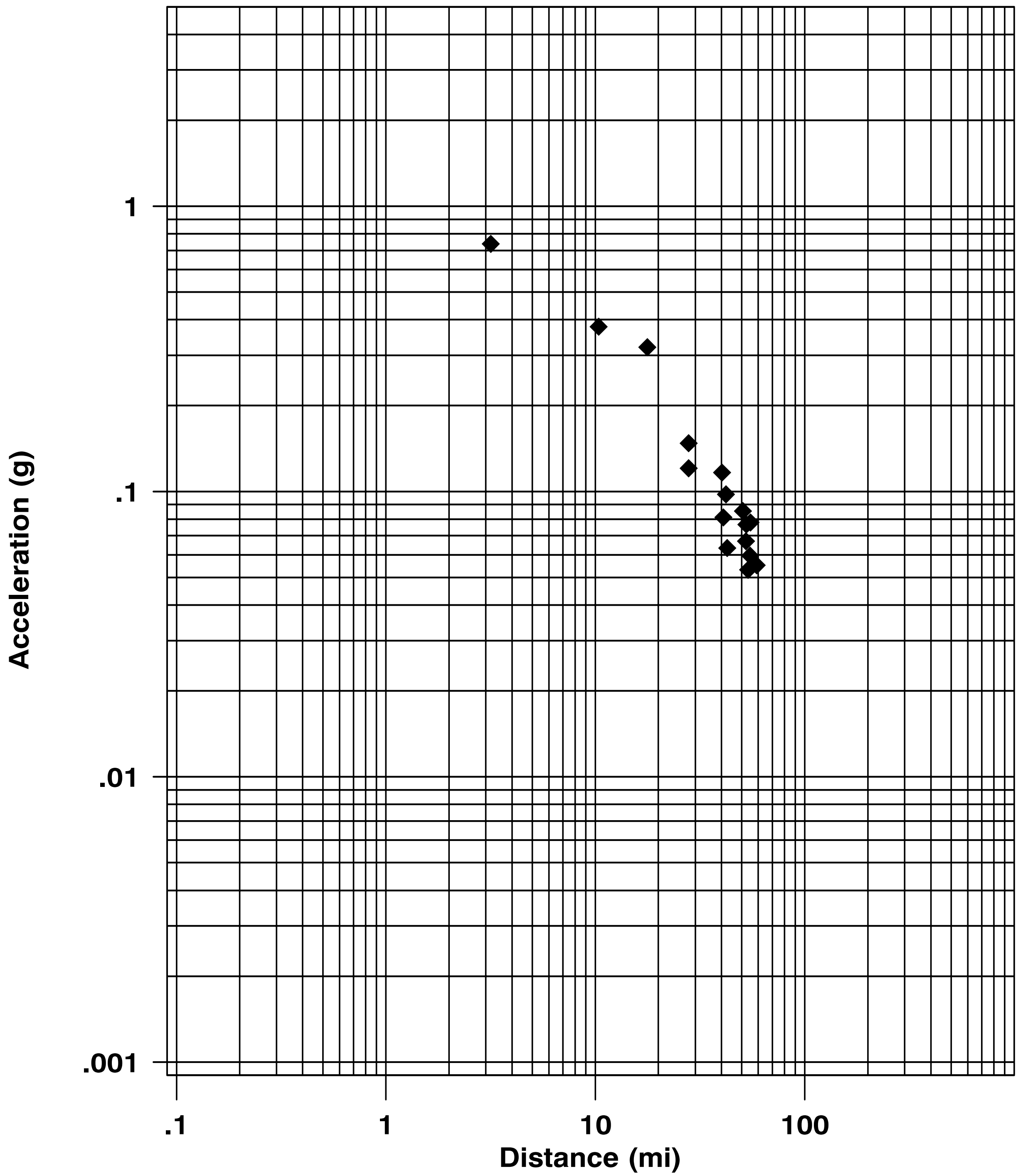


W.O. 7638-A2-SC

PLATE C-3

MAXIMUM EARTHQUAKES

BEVAND



W.O. 7638-A2-SC
PLATE C-4

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*************************
*                       *
*     E Q S E A R C H   *
*                       *
*      Version 3.00     *
*                       *
*************************
```

ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 7638-A-SC

DATE: 10-30-2019

JOB NAME: BEVAND

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

SITE COORDINATES:
SITE LATITUDE: 33.0545
SITE LONGITUDE: 117.3005

SEARCH DATES:
START DATE: 1800
END DATE: 2019

SEARCH RADIUS:
62.2 mi
100.1 km

ATTENUATION RELATION: 11) Bozorgnia Campbell Niazi (1999) Hor.-Pleist. Soil-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: 0   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]
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.547	X	3.8(6.0)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.062	VI	17.8(28.6)
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.052	VI	21.1(33.9)
DMG	32.7000	117.2000	05/27/1862	20 0 0.0	0.0	5.90	0.075	VII	25.2(40.5)
T-A	32.6700	117.1700	12/00/1856	0 0 0.0	0.0	5.00	0.040	V	27.6(44.4)
T-A	32.6700	117.1700	10/21/1862	0 0 0.0	0.0	5.00	0.040	V	27.6(44.4)
T-A	32.6700	117.1700	05/24/1865	0 0 0.0	0.0	5.00	0.040	V	27.6(44.4)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.038	V	33.5(53.9)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.048	VI	33.9(54.6)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.030	V	36.1(58.2)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.031	V	41.7(67.2)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.024	IV	44.9(72.3)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.043	VI	44.9(72.3)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.024	IV	44.9(72.3)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.031	V	46.1(74.2)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.021	IV	50.2(80.7)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.022	IV	50.4(81.0)
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.064	VI	51.0(82.1)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.021	IV	51.0(82.1)
GSG	33.4200	116.4890	07/07/2010	235333.5	14.0	5.50	0.027	V	53.2(85.6)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.022	IV	53.3(85.8)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.022	IV	53.3(85.8)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.020	IV	54.3(87.4)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.046	VI	54.3(87.4)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.043	VI	54.6(87.9)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.026	V	54.9(88.4)
GSP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.020	IV	55.1(88.7)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.019	IV	55.5(89.3)
GSP	33.4315	116.4427	06/10/2016	080438.7	12.3	5.19	0.021	IV	56.0(90.0)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.020	IV	56.7(91.3)
T-A	32.2500	117.5000	01/13/1877	20 0 0.0	0.0	5.00	0.019	IV	56.7(91.3)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.029	V	58.6(94.4)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.033	V	58.7(94.4)
DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.023	IV	61.2(98.6)
GSP	32.3290	117.9170	06/15/2004	222848.2	10.0	5.30	0.020	IV	61.6(99.1)

****

-END OF SEARCH- 35 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2019

LENGTH OF SEARCH TIME: 220 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 3.8 MILES (6.0 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 6.8

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.547 g

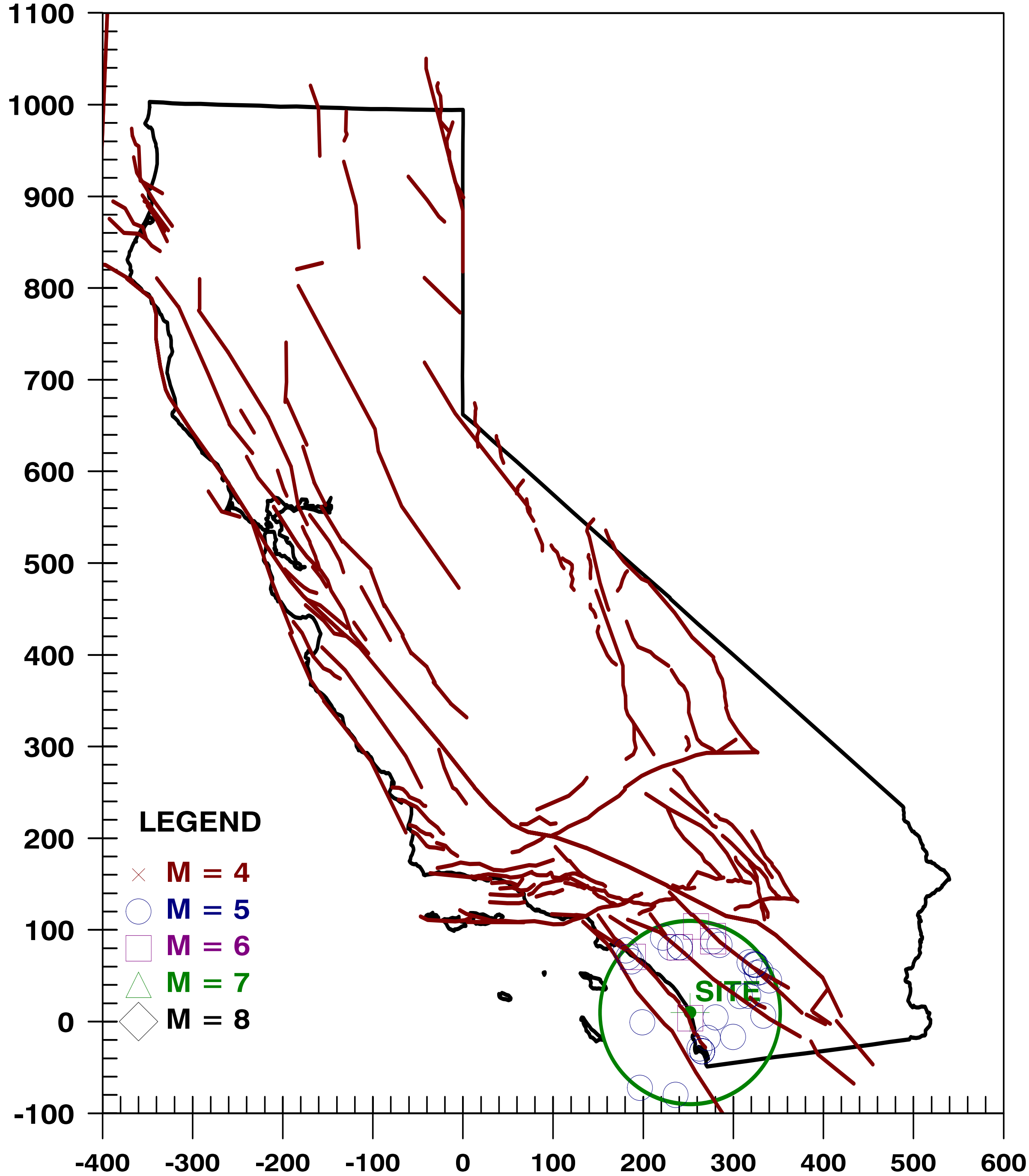
COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
a-value= 0.924
b-value= 0.392
beta-value= 0.904

TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	35	0.15982
4.5	35	0.15982
5.0	35	0.15982
5.5	13	0.05936
6.0	6	0.02740
6.5	2	0.00913

EARTHQUAKE EPICENTER MAP

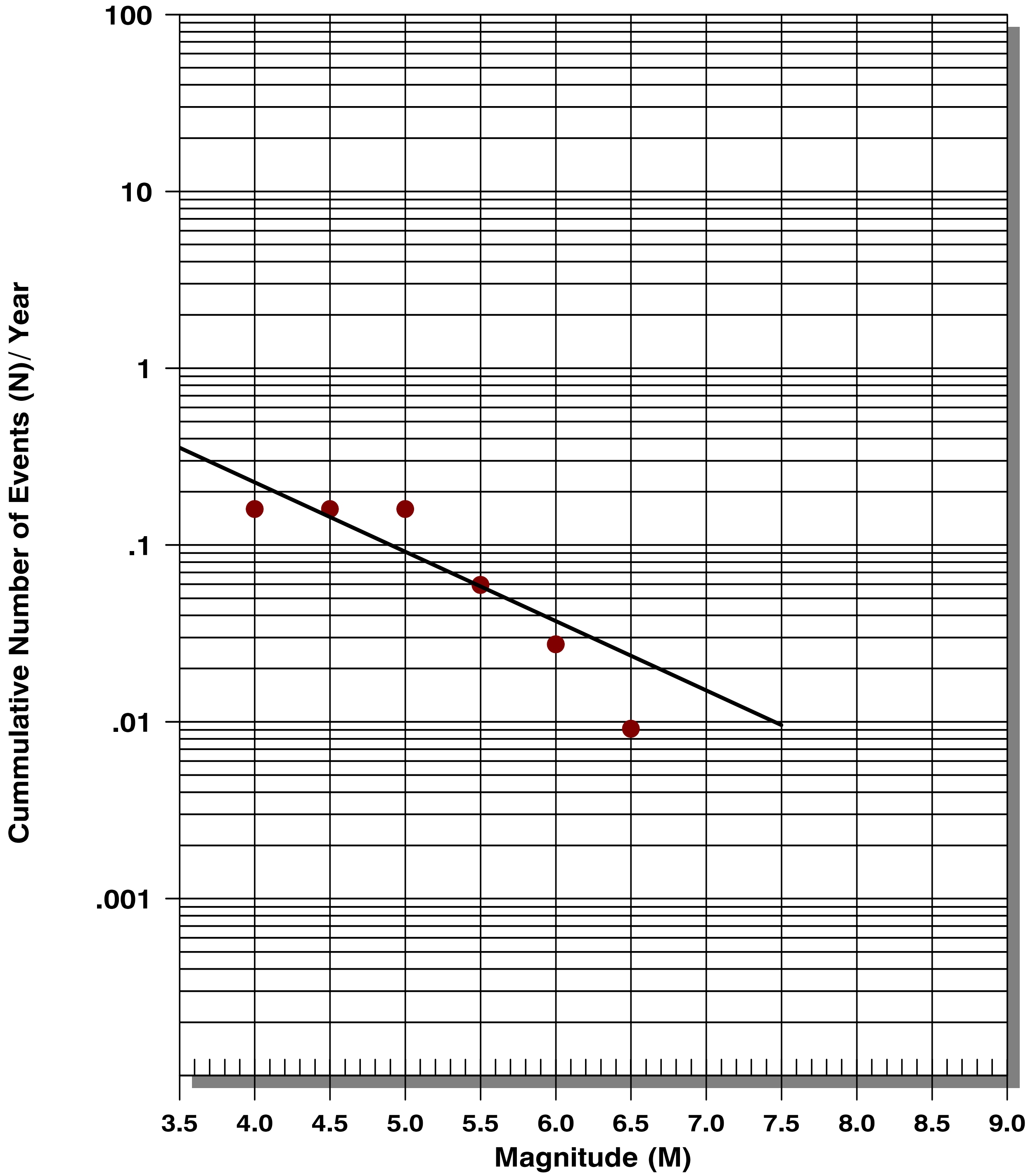
BEVAND



W.O. 7638-A2-SC
PLATE C-8

EARTHQUAKE RECURRENCE CURVE

BEVAND

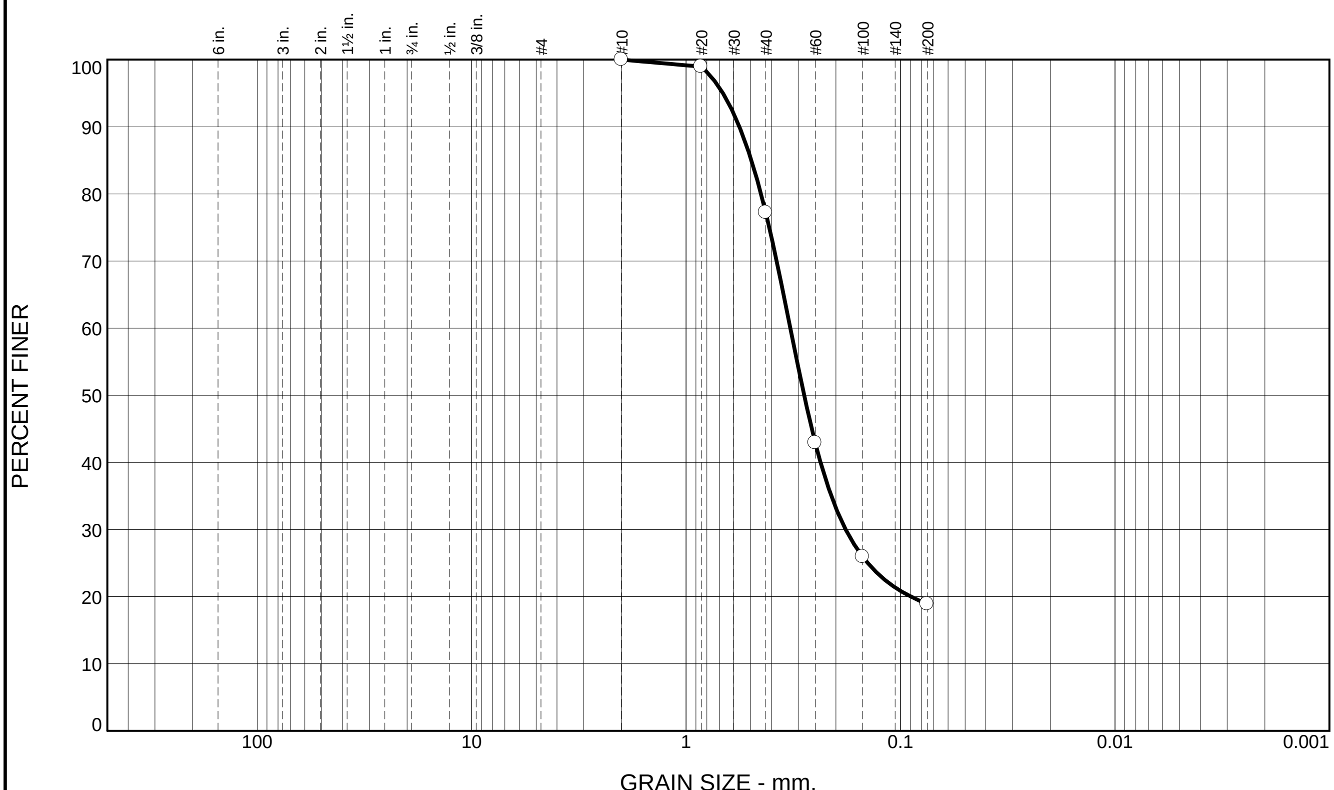


W.O. 7638-A2-SC
PLATE C-9

APPENDIX D

LABORATORY DATA

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	22.8	58.3	18.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.0		
#40	77.2		
#60	42.9		
#100	25.9		
#200	18.9		

* (no specification provided)

Soil Description

Reddish Brown Silty Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.5639 D_{85}= 0.4958 D_{60}= 0.3267
D_{50}= 0.2816 D_{30}= 0.1801 D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Source of Sample: HA-2 **Depth:** 3.25-6.33
Sample Number: HA-2

Date: 11-6-19

GeoSoils, Inc.

Client: Bevand
Project: 312 Neptune Ave.

Project No: 7638-A-SC **Plate**

Tested By: TR/MS **Checked By:** TR

W.O. 7638-A2-SC
PLATE D-1

SoilCor
CORROSION & THERMAL SCIENCES
42184 Remington Ave, Temecula CA 92590
ph (951) 795-3135 • fx (951) 894-2683

Work Order No.: 19K2141
Client: GeoSoils, Inc.
Project No.: 7638-A-SC
Project Name: Bevand
Report Date: November 12, 2019

Laboratory Test(s) Results Summary

The subject soil sample was processed with the U.S. Standard No. 10 Sieve and tested for pH (ASTM G 51-95 2012), Soil Resistivity (ASTM G 57-06 2012), Sulfate Ion Content (ASTM D 516-16) and Chloride Ion Content (ASTM D 512-12B). The test results follow:

Sample Identification	pH (H+)	As Rec'd Resistivity (ohm-cm)	Saturated Resistivity (ohm-cm)	Sulfate Content (mg/L)	Chloride Content (mg/L)
HA-2 @ 3-6.5 ft	7.4	10,800	2,400	160	40

*ND=No Detection

We appreciate the opportunity to serve you. Please do not hesitate to contact us with any questions or clarifications regarding these results or procedures.

Ahmet K. Kaya, Laboratory Manager

APPENDIX E

SLOPE STABILITY ANALYSIS

APPENDIX E

SLOPE STABILITY ANALYSIS

INTRODUCTION OF GSTABL7 v.2 COMPUTER PROGRAM

Introduction

GSTABL7 v.2 is a fully integrated slope stability analysis program. It permits the engineer to develop the slope geometry interactively and perform slope analysis from within a single program. The slope analysis portion of GSTABL7 v.2 uses a modified version of the popular STABL program, originally developed at Purdue University.

GSTABL7 v.2 performs a two dimensional analysis to compute the factor of safety (FOS) for a layered slope. This program can be used to search for the most critical surface or the FOS may be determined for specific surfaces. GSTABL7, Version 2, is programmed to handle:

1. Heterogenous soil systems
2. Anisotropic soil strength properties
3. Reinforced slopes
4. Nonlinear Mohr-Coulomb strength envelope
5. Pore water pressures for effective stress analysis using:
    a. Phreatic and piezometric surfaces
    b. Pore pressure grid
    c. R factor
    d. Constant pore water pressure
6. Pseudo-static earthquake loading
7. Surcharge boundary loads
8. Automatic generation and analysis of an unlimited number of circular, noncircular and block-shaped failure surfaces
9. Analysis of right-facing slopes
10. Both SI and Imperial units

General Information

If the reviewer wishes to obtain more information concerning slope stability analysis, the following publications 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. 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.

3. 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.

4. 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.

GSTABL7 v.2 Features

The present version of GSTABL7 v.2 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 planes.

2. Slope geometry and surcharge boundary loads.

3. Apparent dip of bedding plane can be modeled in an anisotropic angular range (i.e., from 0 to 90 degrees. For this analysis, GSI incorporated isotropic soil strengths for all earth materials, excepting the Torrey Sandstone. For the Torrey Sandstone, we used an anisotropic angular range between 2 and17 degrees from the horizontal oriented in an out-of-slope direction, owing to its cross-bedded nature, our observations, and regional geologic information (Kennedy and Tan, 2007).

4. Pseudo-static earthquake loading. A seismic coefficient (*k)* of 0.15 and a peak horizontal ground acceleration of 0.499 g were used in the analyses.

5. Static and seismic soil strength parameters used in the slope stability analyses are provided in Table E-1.

TABLE E-1 - SOIL STRENGTH PARAMETERS

SOIL MATERIALS	SOIL UNIT WEIGHT (pcf)		STATIC SHEAR STRENGTH PARAMETERS	
	Total	Saturated	C (psf)	Φ (degrees)
Quaternary Beach Deposits (Qb)	115.0	120.0	0.0	33.0
Undocumented Artificial Fill (Afu)	130.0	N/A	50.0	32.0
Quaternary Old Paralic Deposits (Qop)	130.0	135.0	300.0	34.0
Tertiary Torrey Sandstone - Parallel Bed (Tt)	130.0	135.0	800.00	35.0
Tertiary Torrey Sandstone - Cross-Bed (Tt)	130.0	135.0	1,000.0	37.0

Seismic Discussion

Seismic stability analyses were approximated using a pseudo-static approach. The major difficulty in the pseudo-static approach arises from the appropriate selection of the seismic coefficient used in the analysis. The use of a static inertia force equal to this acceleration during an earthquake (rigid-body response) would be extremely conservative for several reasons including: (1) only low height, stiff/dense embankments or embankments in confined areas may respond essentially as rigid structures; (2) an earthquake's inertia force is enacted on a mass for a short time period. Therefore, replacing a transient force by a pseudo-static force representing the maximum acceleration may be considered overly conservative; (3) assuming that total pseudo-static loading is applied evenly throughout the embankment for an extended period of time is an incorrect assumption, as the length of the failure surface analyzed is usually much greater than the wave length of seismic waves generated by earthquakes; and (4) the seismic waves would place portions of the mass in compression and some in tension, resulting in only a limited portion of the failure surface analyzed moving in a downslope direction, at any one instant of earthquake loading.

The coefficients usually suggested by regulating agencies, counties and municipalities are in the range of 0.05g to 0.25g. For example, past regulatory guidelines within the city and county of Los Angeles indicated that the slope stability pseudostatic coefficient = 0.1 to 0.15*i*.

The method developed by Krinitzsky, Gould, and Edinger (1993) which was in turn based on Taniguchi and Sasaki (1986), was referenced. This method is based on empirical data and the performance of existing earth embankments during seismic loading. Our review of “Guidelines for Evaluating and Mitigating Seismic Hazards in California” California Department of Conservation, California Geological Survey ([CGS], 2008) indicates the

State of California recommends using pseudo-static coefficient of 0.15*i* for design earthquakes of M 8.25 or greater and using 0.1 for earthquake parameter M 6.5. Therefore, for reasonable conservatism, a seismic coefficient of 0.15*i* was used in our analysis for a M7.2 event on the Rose Canyon fault. GSI also incorporated a peak horizontal ground acceleration (PGA_M) of 0.499 g into the seismic analysis.

Output Information

Output information includes:

1. All input data.

2. FOS for the 10 most critical surfaces for static and pseudo-static stability situation.

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

4. Note, that in the analysis, ±4,999 trial surfaces were analyzed for each section for either static or pseudo-static analyses.

Results of Slope Stability Calculations

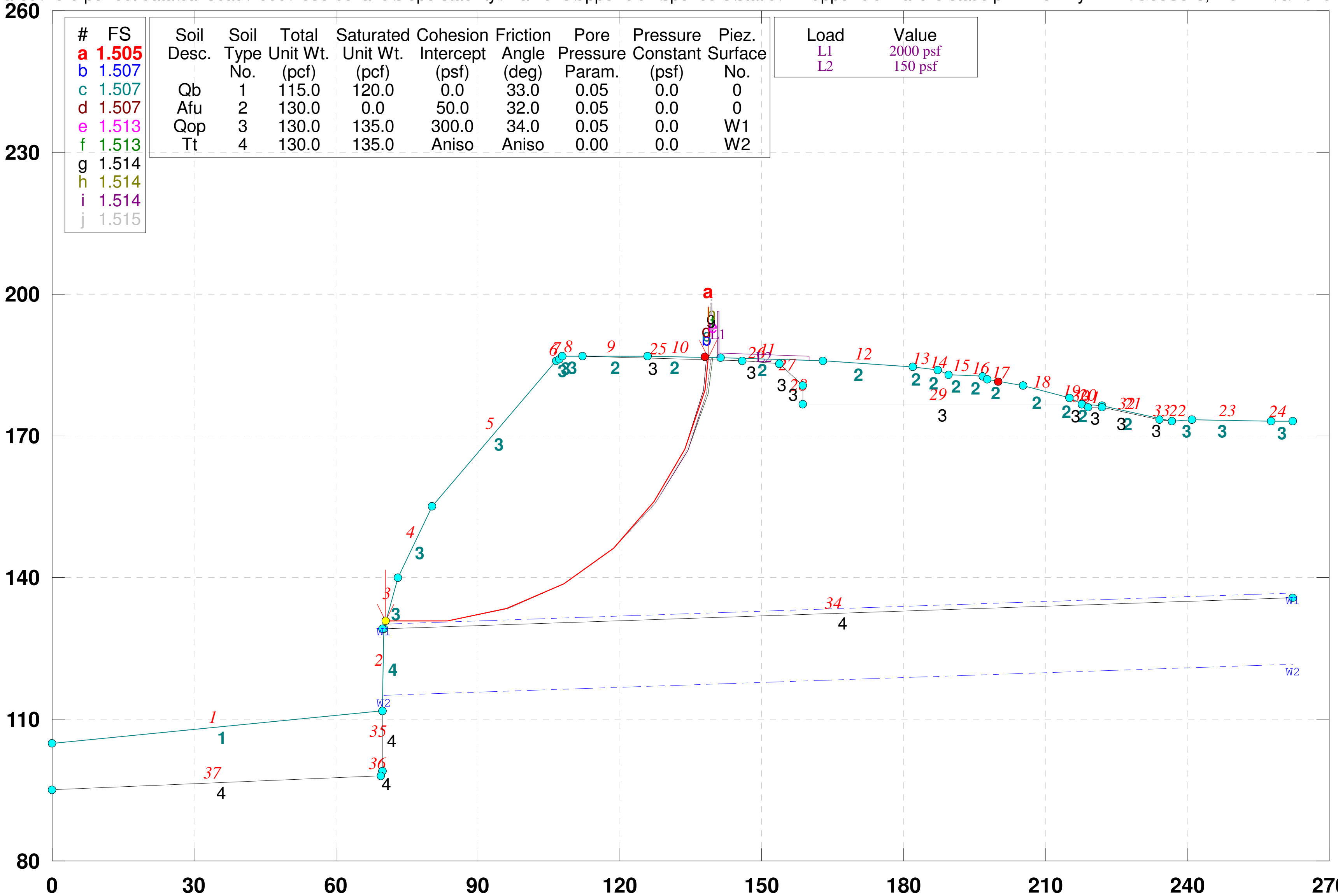
Table E-2 provides a summary of the results of our stability analyses along Geologic Cross Section X-X' (see Plates 1 and 2). Computer printouts from the GSTABL7 program are also included herein.

TABLE E-2 - SUMMARY OF SLOPE STABILITY ANALYSES

LOCATION	FACTOR-OF-SAFETY (FOS) EXISTING SLOPE CONDITION		METHOD	COMMENTS
	STATIC	SEISMIC		
Section X-X' Upper Bluff Failure	1.5 (See Plate E-1)	1.15 (See Plate E-2)	GLE (Spencer's)	Adequate Static and Seismic FOS
Section X-X' Upper Bluff Failure	1.2 (See Plate E-3)	----	GLE (Spencer's)	--------

BEVAND/7638-A-SC SECTION X-X' UPPER BLUFF STATIC

x:\shared\word perfect data\carlsbad\7600\7638 bevand\slope stability\final runs\upper bluff\spencers\static\x-x' upper bluff failure static.pl2 Run By: RBB/GeoSoils, Inc. 12/3/2019 10:04AM



#	FS
a	1.505
b	1.507
c	1.507
d	1.507
e	1.513
f	1.513
g	1.514
h	1.514
i	1.514
j	1.515

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qb	1	115.0	120.0	0.0	33.0	0.05	0.0	0
Afu	2	130.0	0.0	50.0	32.0	0.05	0.0	0
Qop	3	130.0	135.0	300.0	34.0	0.05	0.0	W1
Tt	4	130.0	135.0	Aniso	Aniso	0.00	0.0	W2

Load	Value
L1	2000 psf
L2	150 psf

GSTABL7 v.2 FSmin=1.505

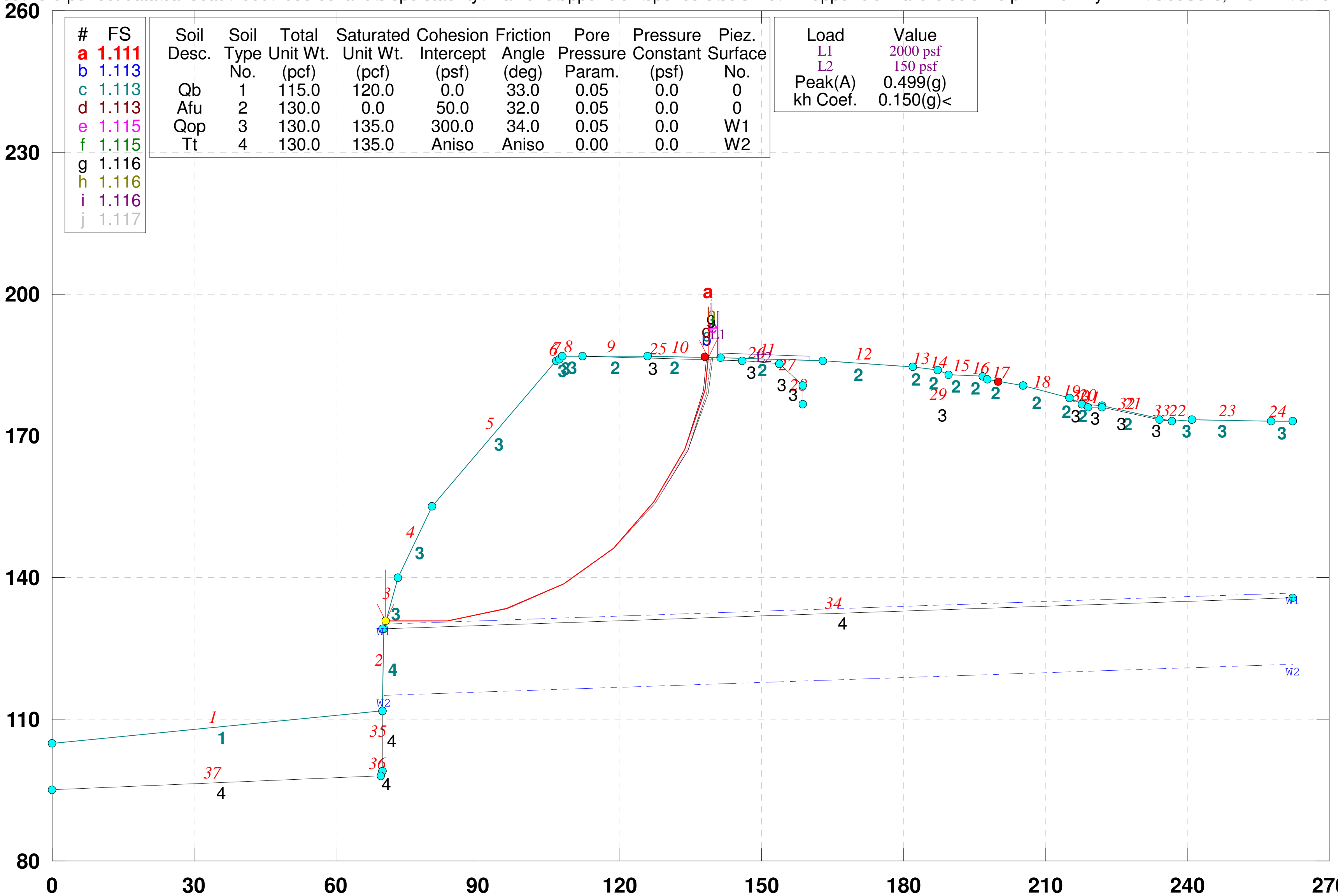
Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

W.O. 7638-A2-SC

PLATE E-1

BEVAND/7638-A-SC SECTION X-X' UPPER BLUFF SEISMIC

x:\shared\word perfect data\carlsbad\7600\7638 bevand\slope stability\final runs\upper bluff\spencers\seismic\x-x' upper bluff failure seismic.pl2 Run By: RBB/GeoSoils, Inc. 12/3/2019 10:05AM



GSTABL7 v.2 FSmin=1.111

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

W.O. 7638-A2-SC

PLATE E-2

BEVAND/7638-A-SC SECTION X-X' UPPER BLUFF STATIC

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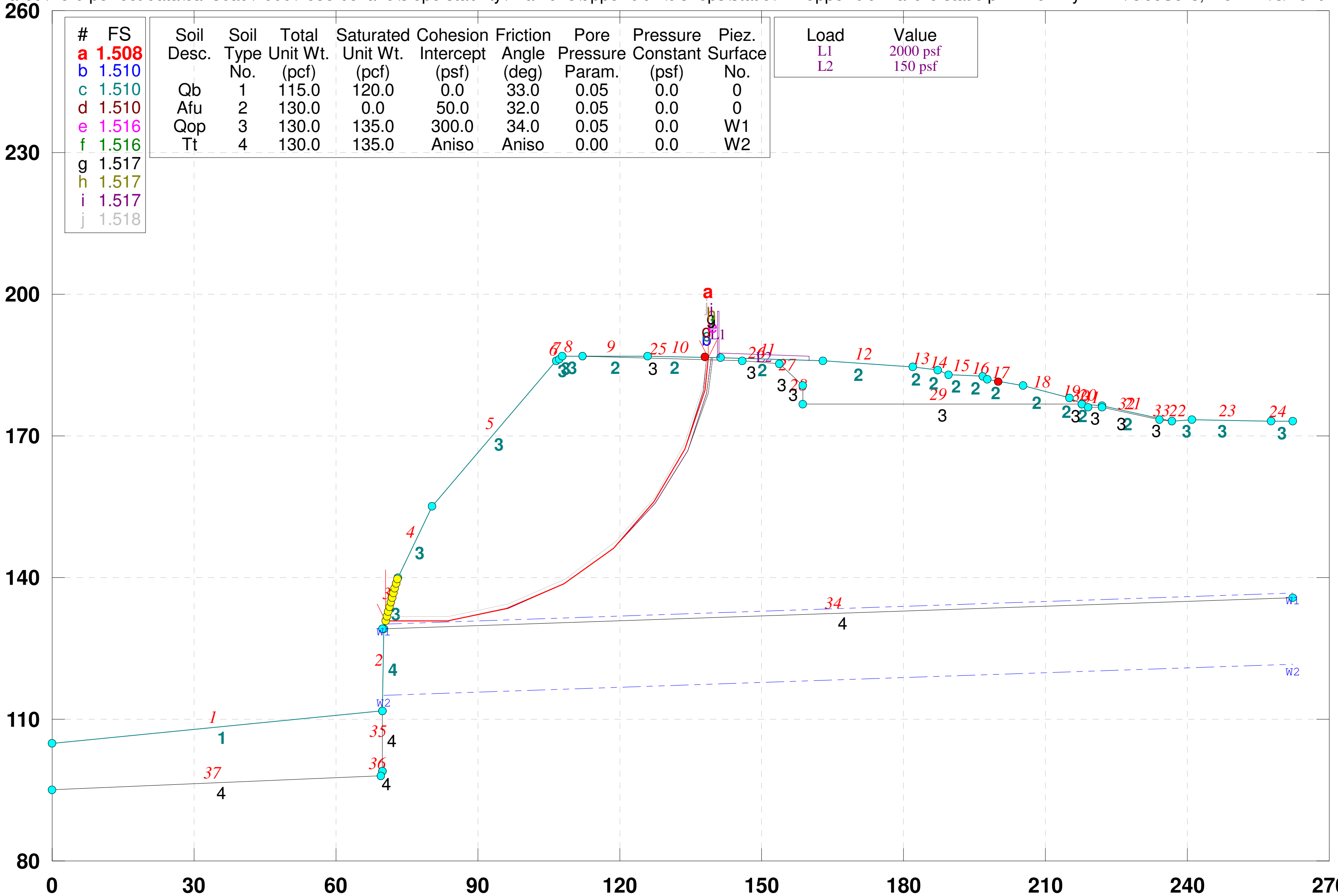


#	FS
a	1.508
b	1.510
c	1.510
d	1.510
e	1.516
f	1.516
g	1.517
h	1.517
i	1.517
j	1.518

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qb	1	115.0	120.0	0.0	33.0	0.05	0.0	0
Afu	2	130.0	0.0	50.0	32.0	0.05	0.0	0
Qop	3	130.0	135.0	300.0	34.0	0.05	0.0	W1
Tt	4	130.0	135.0	Aniso	Aniso	0.00	0.0	W2

Load	Value
L1	2000 psf
L2	150 psf

GSTABL7 v.2 FSmin=1.508

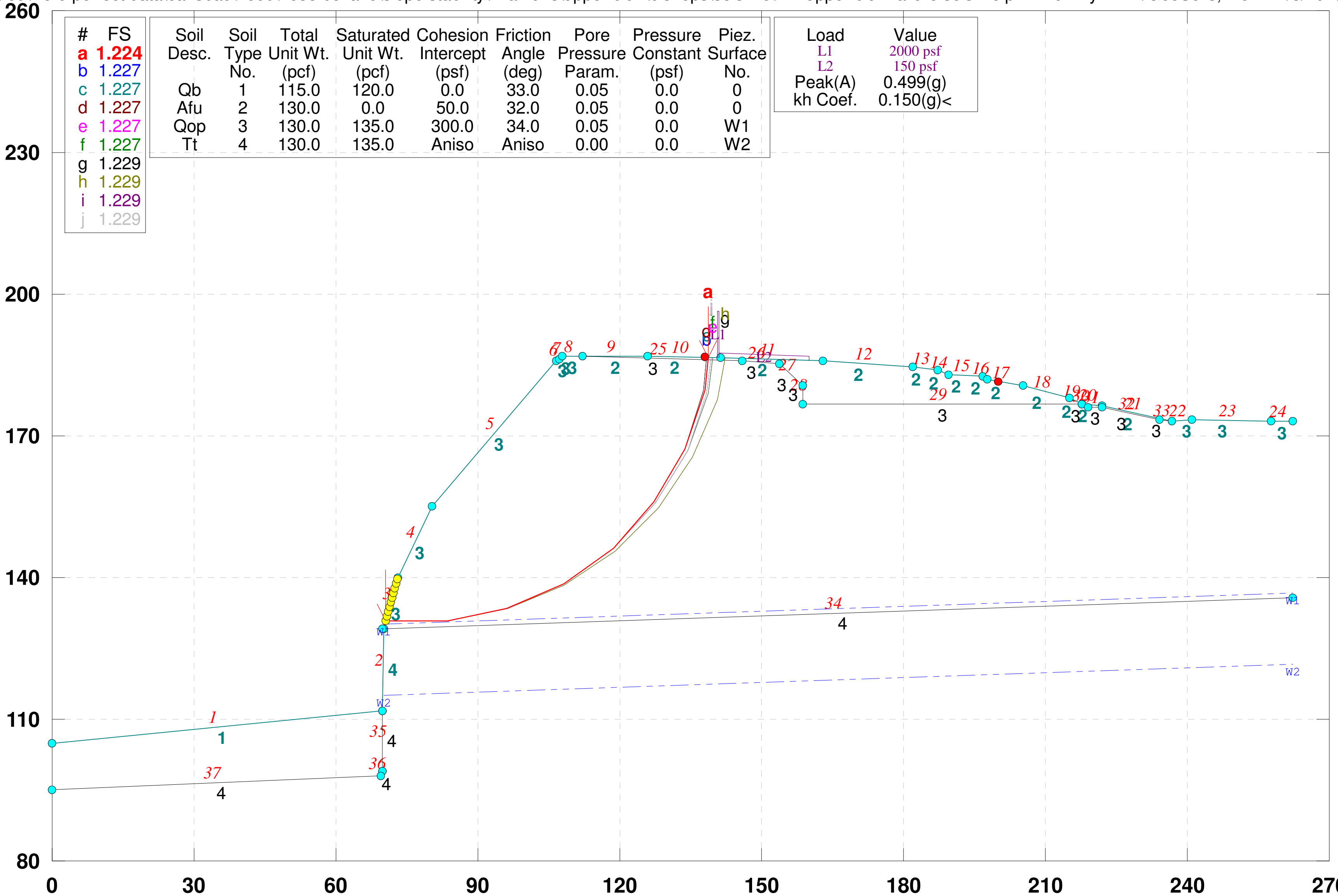
Safety Factors Are Calculated By The Modified Bishop Method

W.O. 7638-A2-SC

PLATE E-3

BEVAND/7638-A-SC SECTION X-X' UPPER BLUFF SEISMIC

x:\shared\word perfect data\carlsbad\7600\7638 bevand\slope stability\final runs\upper bluff\bishops\seismic\x-x' upper bluff failure seismic.pl2 Run By: RBB/GeoSoils, Inc. 12/3/2019 10:10AM



GSTABL7 v.2 FSmin=1.224

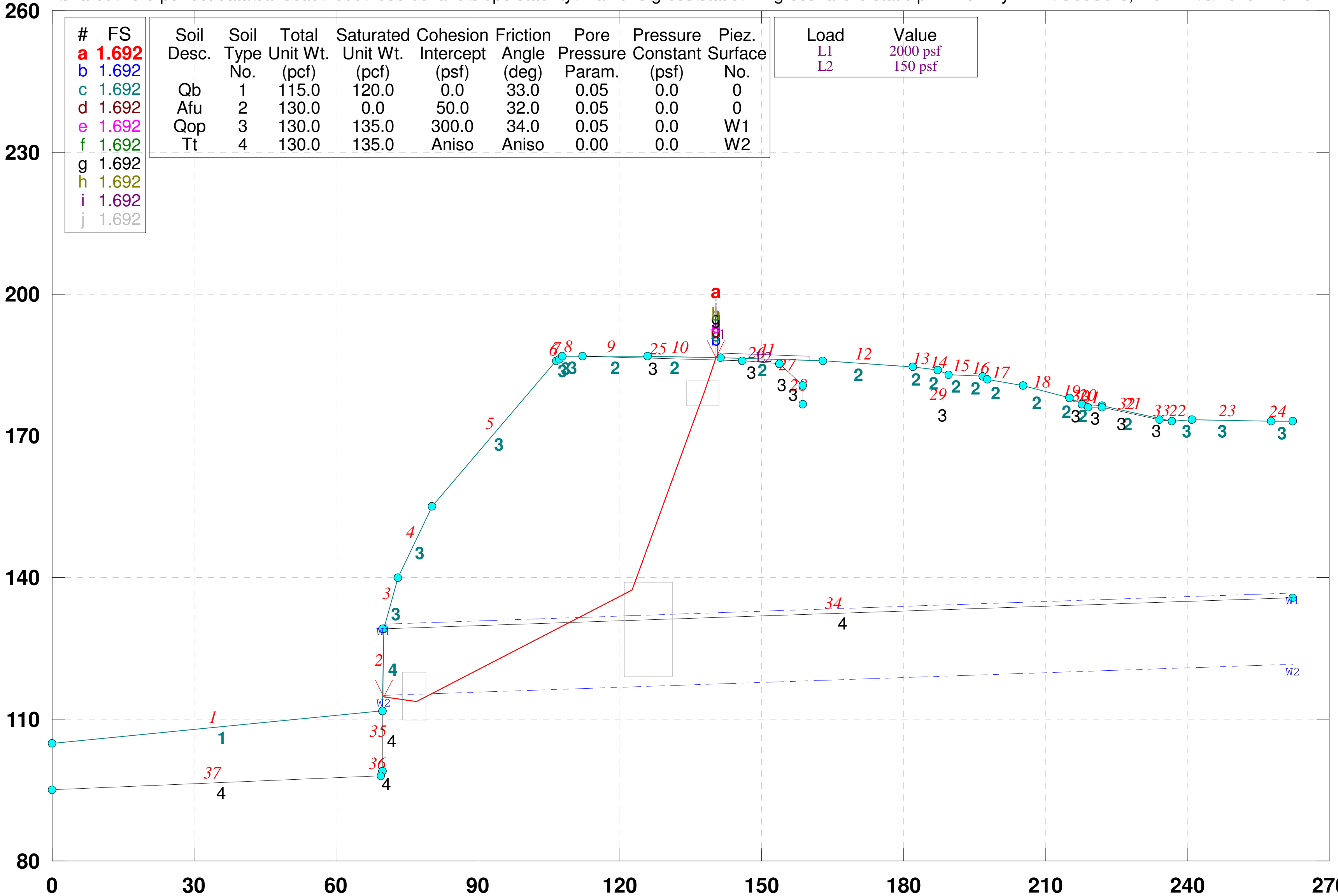
Safety Factors Are Calculated By The Modified Bishop Method

W.O. 7638-A2-SC

PLATE E-4

BEVAND/7638-A-SC SECTION X-X' GROSS STATIC

x:\shared\word perfect data\carlsbad\7600\7638 bevand\slope stability\final runs\gross\static\x-x' gross failure static.pl2 Run By: RBB/GeoSoils, Inc. 12/3/2019 10:25AM



#	FS
a	1.692
b	1.692
c	1.692
d	1.692
e	1.692
f	1.692
g	1.692
h	1.692
i	1.692
j	1.692

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qb	1	115.0	120.0	0.0	33.0	0.05	0.0	0
Afu	2	130.0	0.0	50.0	32.0	0.05	0.0	0
Qop	3	130.0	135.0	300.0	34.0	0.05	0.0	W1
Tt	4	130.0	135.0	Aniso	Aniso	0.00	0.0	W2

Load	Value
L1	2000 psf
L2	150 psf

GSTABL7 v.2 FSmin=1.692

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

W.O. 7638-A2-SC

PLATE E-5

BEVAND/7638-A-SC SECTION X-X' GROSS SEISMIC

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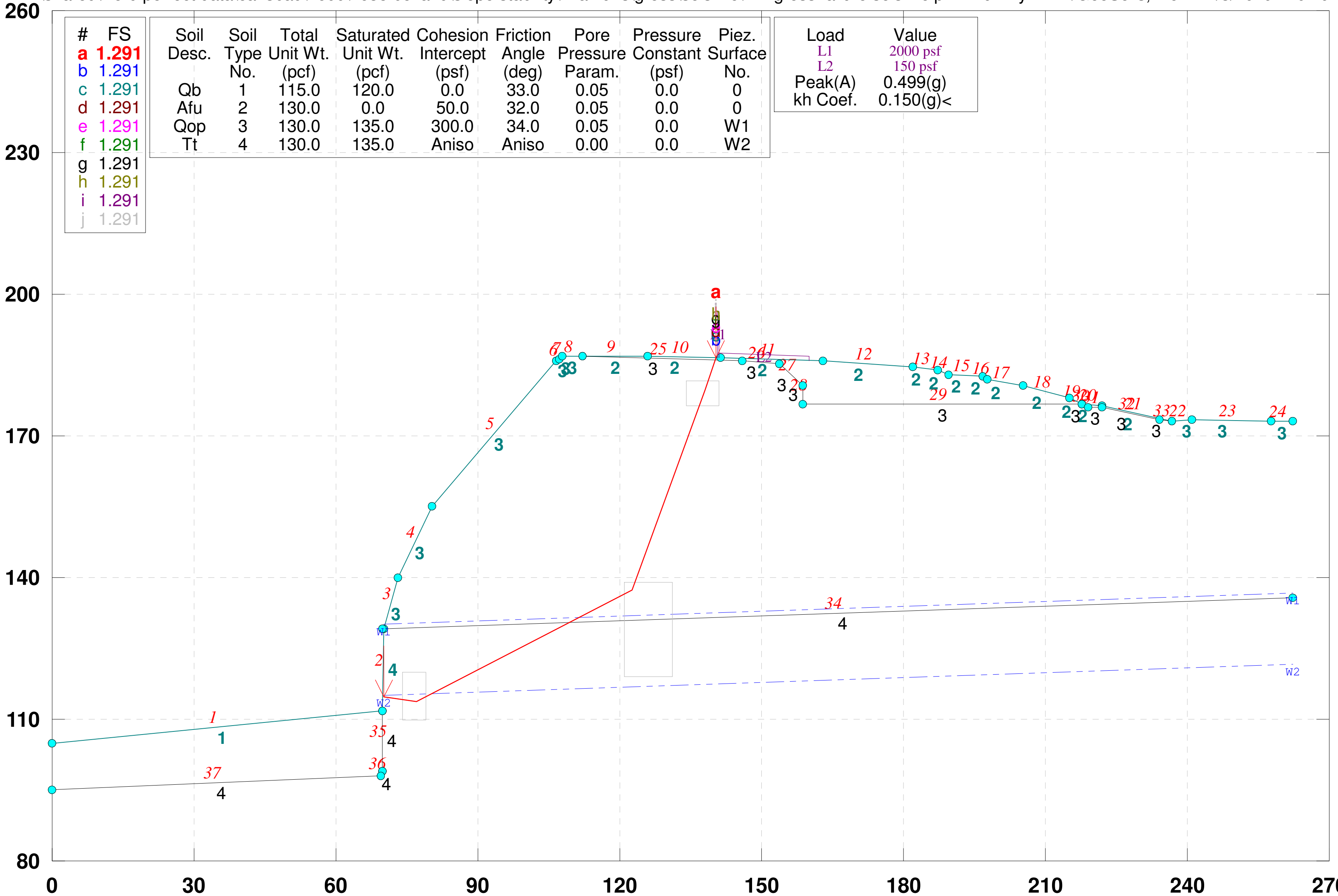
a	1.291
b	1.291
c	1.291
d	1.291
e	1.291
f	1.291
g	1.291
h	1.291
i	1.291
j	1.291

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qb	1	115.0	120.0	0.0	33.0	0.05	0.0	0
Afu	2	130.0	0.0	50.0	32.0	0.05	0.0	0
Qop	3	130.0	135.0	300.0	34.0	0.05	0.0	W1
Tt	4	130.0	135.0	Aniso	Aniso	0.00	0.0	W2

Load	Value
L1	2000 psf
L2	150 psf
Peak(A)	0.499(g)
kh Coef.	0.150(g)<



GSTABL7 v.2 FSmin=1.291

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

W.O. 7638-A2-SC

PLATE E-6

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), and/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 utilized 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 utilized 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/or the developer's representative.

If import material is required for grading, representative samples of the materials to be utilized 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/or 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/or 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, and/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/or 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/or be planted in accordance with the project specifications and/or as recommended by a landscape architect. Such protection and/or 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.

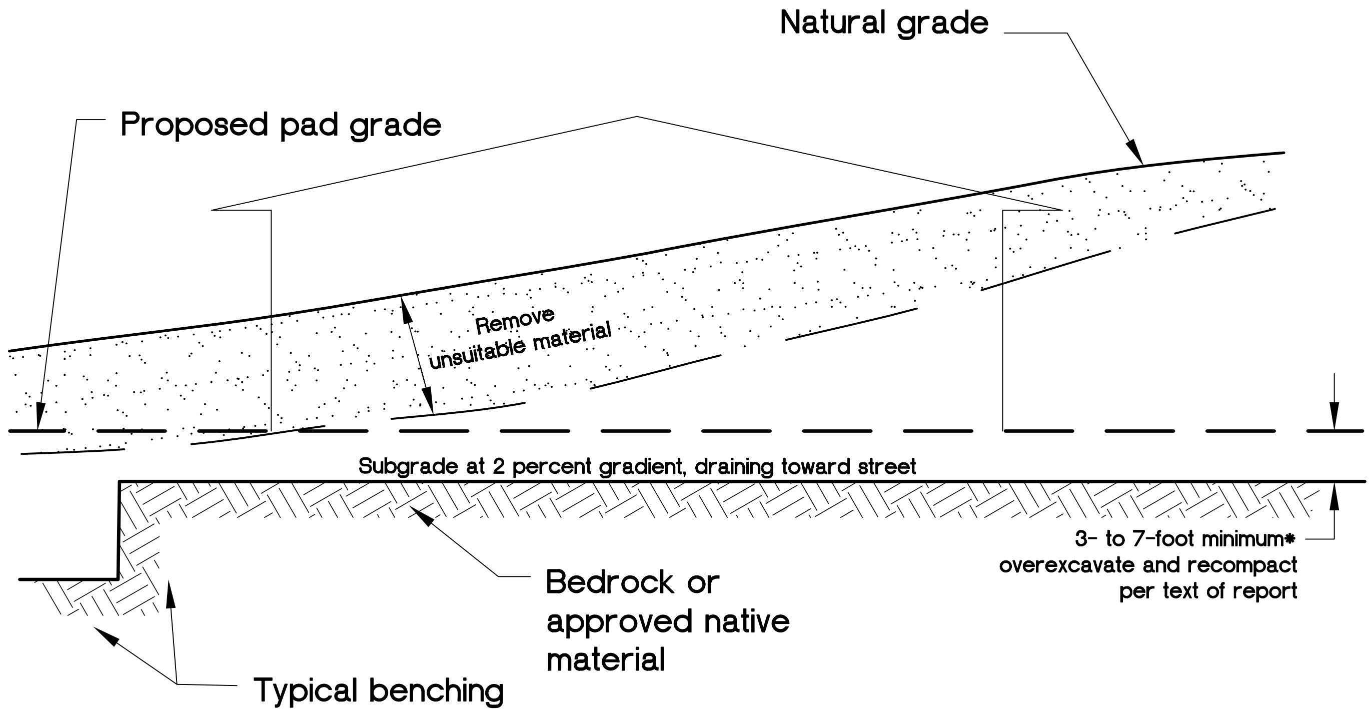
Trench and Vertical Excavation

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.

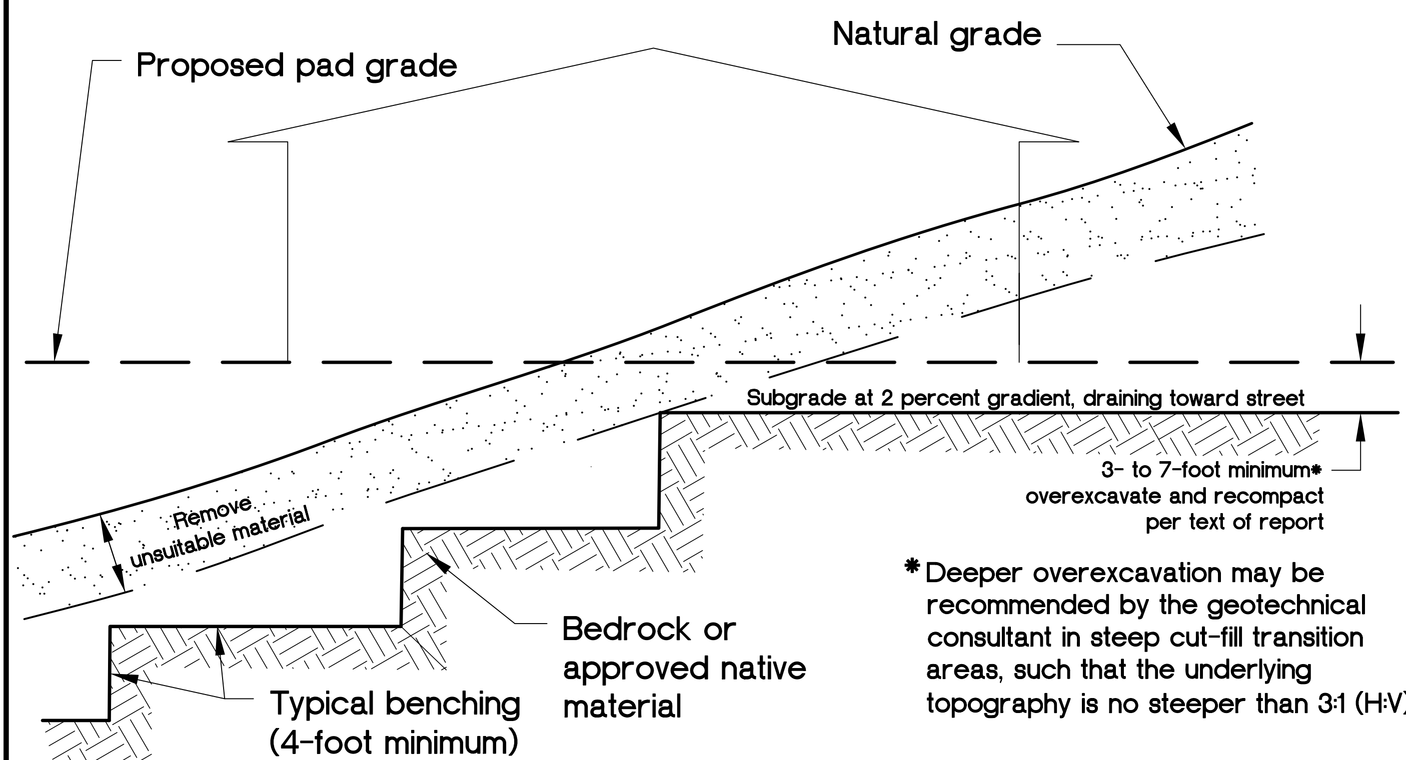
All trench excavations or vertical cuts in excess of 5 feet deep, which any person enters, should be shored or laid back. Trench access should be provided in accordance with Cal/OSHA and/or state and local standards. Our personnel are directed not to enter any trench by being lowered or "riding down"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



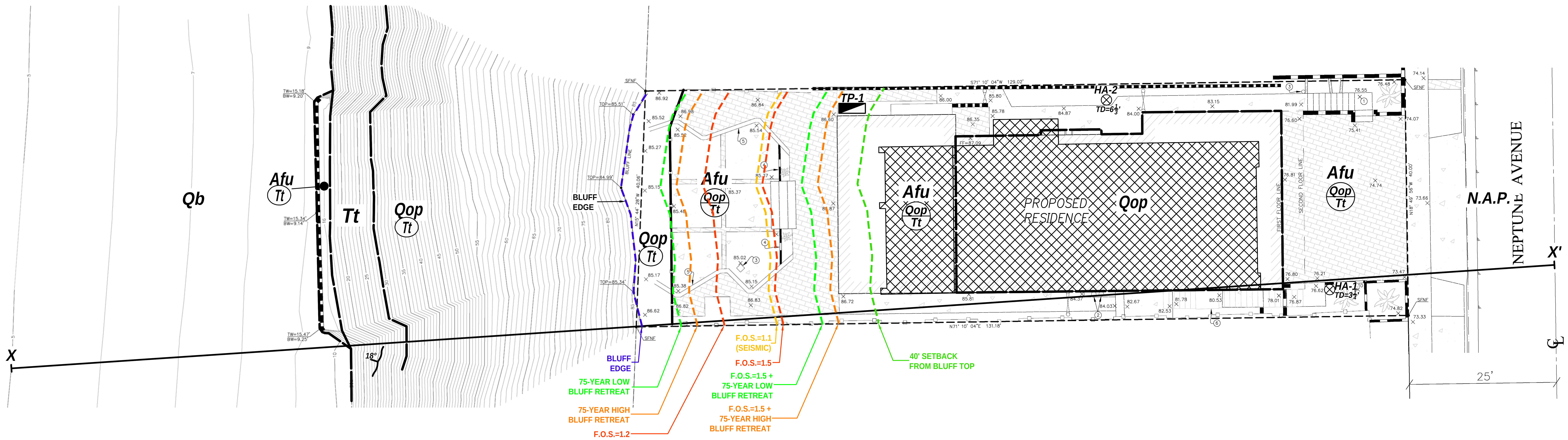
CUT LOT OR MATERIAL-TYPE TRANSITION



CUT-FILL LOT (DAYLIGHT TRANSITION)

GeoSoils, Inc.	TRANSITION LOT DETAILS	Plate F-12

TOPOGRAPHIC SURVEY



LEGEND

- PRECISE BOUNDARY
- RECORD BOUNDARY
- EDGE OF PAVEMENT
- FLOWLINE
- CONCRETE
- WOOD FENCE
- CONCRETE MASONRY UNIT WALL
- CONCRETE MASONRY UNIT FENCE
- GATE
- CONCRETE
- BRICK
- TREE

1. CONCRETE STEPS
2. GAS METER
3. DRAIN
4. BRICK RETAINING WALL
5. RAILROAD TIES RETAINING WALL
6. STAIRS ABOVE GROUND

×100.00 GROUND ELEVATION
SFNF SEARCHED FOR, NOT FOUND

LEGAL DESCRIPTION

LOT 3 IN BLOCK "B" OF SEASIDE GARDENS IN THE CITY OF ENCINITAS, COUNTY OF SAN DIEGO, STATE OF CALIFORNIA, ACCORDING TO MAP NO. 1800 FILED IN THE OFFICE OF THE COUNTY RECORDER OF SAN DIEGO COUNTY, CALIFORNIA AUGUST 6, 1924.

BOUNDARY NOTE

- BOUNDARY PLOTTED PER RECORD INFORMATION PROVIDED IN MAP NO. 1800.
- **THIS IS NOT A PRECISE BOUNDARY SURVEY**
- DUE TO THE LACK OF MONUMENTATION ON AND AROUND THE SUBJECT PROPERTY A CORNER RECORD FILED WITH THE COUNTY OF SAN DIEGO IS NECESSARY AND STRONGLY ADVISED TO ACCURATELY AND PRECISELY LOCATE ALL PROPERTY CORNERS.

BENCH MARK

- BENCHMARK FOR THIS SURVEY IS POINT ID NO. 1025, DESIGNATED AS ENC-25, A BRASS DISC SET IN MONUMENT WELL, LOCATED IN THE INTERSECTION OF SOUTH EL PORTAL STREET AND LA MESA AVENUE, 100 FEET NORTH ALONG THE SOUTHEASTERLY CURB LINE OF SOUTH EL PORTAL STREET IN THE CITY OF ENCINITAS, CA., HAVING A PUBLISHED NAVD 88 ELEVATION OF 60.60'.

GSI LEGEND

- **Qb** — QUATERNARY BEACH DEPOSITS
- **Afu** — ARTIFICIAL FILL – UNDOCUMENTED
- **Qop** — QUATERNARY OLD PARALIC DEPOSITS, CIRCLED WHERE BURIED
- **Tt** — TERTIARY TORREY SANDSTONE, CIRCLED WHERE BURIED
- ? — APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN
- 18° — BEDDING ATTITUDE (CROSS BEDDING) WITH DIP IN DEGREES
- TP-1 — APPROXIMATE LOCATION OF EXPLORATORY TEST PIT
- HA-2 TD=6½' — APPROXIMATE LOCATION OF HAND-AUGER BORING WITH TOTAL DEPTH IN FEET
- X X' — GEOLOGIC CROSS SECTION LINE
- **N.A.P.** — NOT A PART OF THIS STUDY

N E S W

SCALE: 1" = 10'

0 10 20 30

ALL LOCATIONS ARE APPROXIMATE

This document or efile is not a part of the Construction Documents and should not be relied upon as being an accurate depiction of design.

BASE MAP PROVIDED BY:

RANCHO COASTAL
ENGINEERING & SURVEYING
SINGLE SOURCE DEVELOPMENT CONSULTANT
310 VIA VERA CRUZ, #205
SAN MARCOS, CA. 92078
(760) 510-3152 Ph / (760) 510-3153 Fax

PRIVATE CONTRACT		
SHEET 1	COUNTY OF SAN DIEGO APN: 256-352-03	1 SHEETS
TOPOGRAPHIC SURVEY **MARC BEVAND** 312 NEPTUNE AVENUE ENCINITAS, CA 92024		
MAILING ADDRESS:	TELEPHONE:	
SURVEYOR OF WORK: JAKE D. LOGAN PLS 9042	CHECKED BY:	JOB NO. 7811

GeoSoils, Inc.

GEOTECHNICAL MAP

Plate 1

W.O. 7638-A2-SC	DATE: 05/21	SCALE: 1" = 10'

GSI LEGEND

Qb	QUATERNARY BEACH DEPOSITS
Afu	ARTIFICIAL FILL – UNDOCUMENTED
Qop	QUATERNARY OLD PARALIC DEPOSITS
Tt	TERTIARY TORREY SANDSTONE
?	APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN
	APPROXIMATE GROUNDWATER ELEVATION (PER GSI [2000])
17°	APPARENT DIP OF BEDDING (CROSS BEDDING) WITH DIP IN DEGREES
2°	APPARENT DIP OF BEDDING (REGIONAL BEDDING) WITH DIP IN DEGREES
	CURRENT TIDAL RANGE 2021
	TIDAL RANGE 2096 W/ 0.5% PROBILITY OF EXCEEDANCE W/ 6.5 FT SLR (75 YEARS)

ALL LOCATIONS ARE APPROXIMATE

This document or efile is not a part of the Construction Documents and should not be relied upon as being an accurate depiction of design.

GeoSoils, Inc.

GEOLOGIC CROSS SECTION X-X'

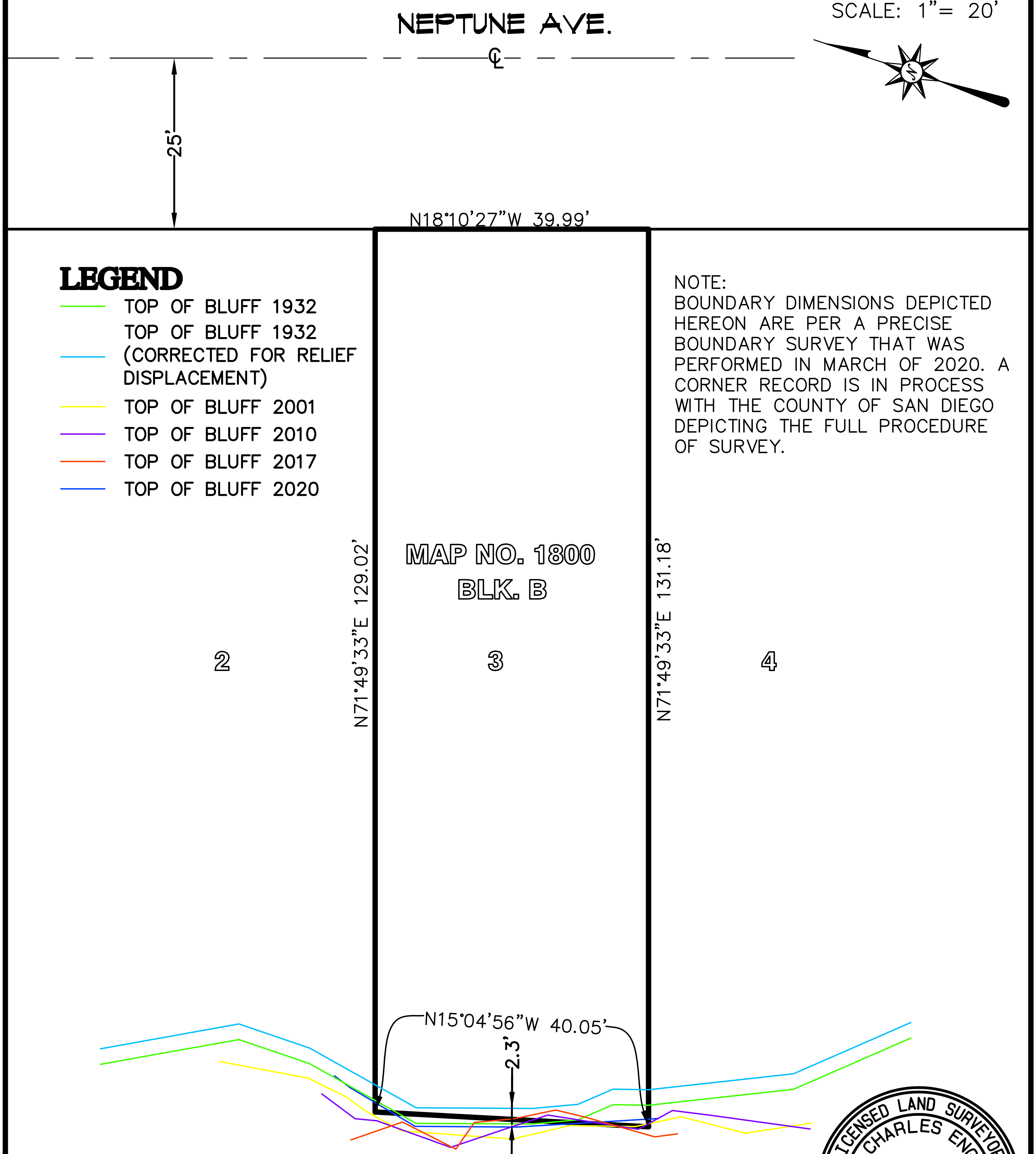
Plate 2

W.O. 7638-A2-SC | DATE: 05/21 | SCALE: 1"=10'

Exhibit B

TOP OF BLUFF EXHIBIT

SCALE: 1"= 20'



NOTE:
BOUNDARY DIMENSIONS DEPICTED HEREON ARE PER A PRECISE BOUNDARY SURVEY THAT WAS PERFORMED IN MARCH OF 2020. A CORNER RECORD IS IN PROCESS WITH THE COUNTY OF SAN DIEGO DEPICTING THE FULL PROCEDURE OF SURVEY.

CLS

COASTAL LAND SOLUTIONS, INC.

577 SECOND STREET
ENCINITAS, CA 92024
PH (760) 230-6025
FAX (760) 230-6026

CLS#1674

Sean C Englert

6-9-2020

SEAN C. ENGLERT, L.S. 7959 DATE

LICENSED LAND SURVEYOR
SEAN CHARLES ENGLERT
L.S. 7959
STATE OF CALIFORNIA

TOP OF BLUFF EXHIBIT

SOURCES AND SYNOPSIS:

TOP OF BLUFF 1932 – DIGITIZED FROM FILE "c-1980_59_1800ppi.tif" DOWNLOADED FROM http://mil.library.ucsb.edu/ap_indexes/FrameFinder/. ROTATED INTO CAD FILE AT TRUE SCALE (1:9600) HOLDING THE DIGITIZED CENTERLINE INTERSECTIONS OF NEPTUNE AVE. WITH S. EL PORTAL ST. AND ROSETA ST. THE DISTANCE BETWEEN THE DIGITIZED CENTERLINES OF NEPTUNE AVE. AND LA MESA AVE. IS 265.98' VS 265' PER MAP NO. 1800, POSSIBLY DUE TO IMAGE DISTORTION ALONG THIS AXIS, TO THE RESOLUTION (1 PIXEL = 0.44') OF THE IMAGE AND INTERPRETATION OF THE IMAGE. TOTAL ESTIMATED ERROR ALONG THIS AXIS IS 1'± IN 265'. THERE MAY BE AN IMAGE DISTORTION ALONG THE NEPTUNE AVE. CENTERLINE AXIS. (885' IMAGE VS 890' MEASURED). THE IMAGE WAS CORRECTED FOR DISTORTION IN BOTH THESE DIRECTIONS IN A SEPARATE CAD FILE TO TEST HOW SCALING THE APPROPRIATE FACTOR IN EACH DIRECTION AND OVERLAYING ON THE SURVEYED BOUNDARY WOULD AFFECT THE BLUFF LOCATION WITH THE SAME RESULT FOR THE BLUFF LOCATION OBTAINED FROM THE UNSCALED IMAGE. THE BLUFF LOCATION IS BLURRED AND APPLYING SCALING FACTORS DIDN'T RESULT IN A DISTINGUISHABLE SHIFT IN THE IMAGE BLUFF LINE. APPLYING A RELIEF DISPLACEMENT CORRECTION TO THE DIGITIZED BLUFF LOCATION AT THE MIDPOINT OF THE REAR LOT LINE RESULTS IN 2.3'± SHIFT OF THE DIGITIZED BLUFF LINE NORTHEASTWARD.

TOP OF BLUFF 2001 – DIGITIZED FROM FILE "ccc-bqk-c_15-23.tif" DOWNLOADED FROM http://mil.library.ucsb.edu/ap_indexes/FrameFinder/. ROTATED AND SCALED (SCALE FACTOR = 12286.76) INTO CAD FILE HOLDING CURB LOCATIONS AT THE NEPTUNE AVE. INTERSECTIONS WITH S. EL PORTAL ST. AND ROSETA ST. ESTIMATED ERROR IN DIGITIZING DUE TO THE RESOLUTION (1 PIXEL = 1.7') OF THE IMAGE AND INTERPRETATION (2'±) OF THE IMAGE. TOTAL ESTIMATED ERROR IS 3.7'±. THE BLUFF LINE LIES AT THE APPROXIMATE N–S CENTERLINE OF THE IMAGE SO APPLYING A RELIEF DISPLACEMENT CORRECTION WOULD RESULT IN A NEGLIGIBLE SHIFT OF THE DIGITIZED BLUFF LINE IN THE EAST–WEST DIRECTION.

TOP OF BLUFF 2010 – DIGITIZED FROM GOOGLE EARTH IMAGE ROTATED AND SCALED INTO CAD FILE HOLDING CURB LOCATIONS AT NEPTUNE AVE. INTERSECTIONS WITH S. EL PORTAL ST. AND ROSETA ST. ESTIMATED ERROR IN DIGITIZING DUE TO THE RESOLUTION (1 PIXEL = 0.3') OF THE IMAGE AND INTERPRETATION (2'±) OF THE IMAGE. TOTAL ESTIMATED ERROR IS 2.3'±. NO DISPLACEMENT CORRECTION APPLIED SINCE THE CAMERA ELEVATION IS UNKNOWN.

TOP OF BLUFF 2017 – DIGITIZED FROM GOOGLE EARTH IMAGE ROTATED AND SCALED INTO CAD FILE HOLDING CURB LOCATIONS AT NEPTUNE AVE. INTERSECTIONS WITH S. EL PORTAL ST. AND ROSETA ST. ESTIMATED ERROR IN DIGITIZING DUE TO THE RESOLUTION (1 PIXEL = 0.3') OF THE IMAGE AND INTERPRETATION (2'±) OF THE IMAGE. TOTAL ESTIMATED ERROR IS 2.3'±. NO DISPLACEMENT CORRECTION APPLIED SINCE THE CAMERA ELEVATION IS UNKNOWN.

TOP OF BLUFF 2020 – SURVEYED MARCH 9, 2020.

CLS

COASTAL LAND SOLUTIONS, INC.

577 SECOND STREET
ENCINITAS, CA 92024
PH (760) 230-6025
FAX (760) 230-6026

6-9-2020

SEAN C. ENGLERT, L.S. 7959 DATE

LICENSED LAND SURVEYOR
SEAN CHARLES ENGLERT
L.S. 7959
STATE OF CALIFORNIA

CLS#1674

Exhibit C

Geopacifica, Inc.

Memo

To: Takuma Easland , Planning

From: James Knowlton, Geotechnical Consultant

Date: 9/12/2024

Re: Review of Erosion Rate Evaluation, 312 Neptune Avenue, Encinitas, California

---

In response to your request, I have reviewed the erosion rate evaluation provided by the applicant's geotechnical consultant for the coastal bluff located at 312 Neptune Avenue, Encinitas, California.

Based on the review of the geotechnical report and extensive experience with adjacent properties on Neptune Avenue, Geopacifica agreed with the applicant's proposed annualized erosion rate of 0.22 feet per year which differs from the California Coastal Commission's standard annualized erosion rate of 0.49 feet per year. The lower erosion rate in this area corresponds with the presence of very dense sandstone bedrock at the base of the coastal bluff which results in a much lower erosion rate than other areas of the coastal bluff in Encinitas. The California Coastal Commission staff recognizes that there are area varying erosion rates along the Encinitas shoreline based upon the differing soil types encountered at the base of the coastal bluffs and allows the city to review (through their geotechnical consultant) the engineering justification for the site specific erosion rate and make the determination that the erosion rate is justified for the site. In combination with the required Factor of Safety, the geotechnical setback was determined to be less than the minimum required 40 feet from bluff edge setback. Therefore, the 40 feet from bluff edge setback was used for the planning and design of the proposed project. The seawall at the base of the bluff and basement walls were not considered in the calculation of the annualized erosion rate. It is a requirement of the City of Encinitas and the California Coastal Commission that the calculation of the Factor-of-Safety setback and the erosion rate of the coastal bluff be performed assuming that there are no engineered devices (sea walls, rock rip-rap, etc.) at the base of the coastal bluff or on the coastal bluff.

Exhibit D

EXISTING TOPOGRAPHY AND DEMOLITION PLAN

EXISTING LEGEND

- PRECISE BOUNDARY
- RECORD BOUNDARY
- EDGE OF PAVEMENT
- FLOWLINE
- CONCRETE
- WOOD FENCE
- CONCRETE MASONRY UNIT WALL
- CONCRETE MASONRY UNIT FENCE
- BASEMENT WALLS
- GATE
- CONCRETE
- BRICK
- TREE

1. CONCRETE STEPS
2. GAS METER
3. DRAIN
4. BRICK RETAINING WALL
5. RAILROAD TIES RETAINING WALL
6. STAIRS ABOVE GROUND
7. WATER METER

×100.00 GROUND ELEVATION

SFNF SEARCHED FOR, NOT FOUND

LEGAL DESCRIPTION

LOT 3 IN BLOCK "B" OF SEASIDE GARDENS IN THE CITY OF ENCINITAS, COUNTY OF SAN DIEGO, STATE OF CALIFORNIA, ACCORDING TO MAP NO. 1800 FILED IN THE OFFICE OF THE COUNTY RECORDER OF SAN DIEGO COUNTY, CALIFORNIA AUGUST 6, 1924.

BOUNDARY NOTE

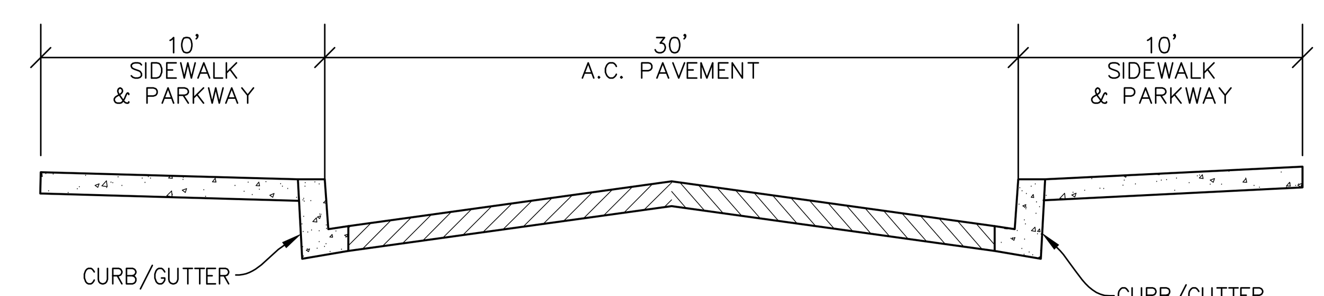
- BOUNDARY PLOTTED PER RECORD INFORMATION PROVIDED IN MAP NO. 1800.
- **THIS IS NOT A PRECISE BOUNDARY SURVEY**
- DUE TO THE LACK OF MONUMENTATION ON AND AROUND THE SUBJECT PROPERTY A CORNER RECORD FILED WITH THE COUNTY OF SAN DIEGO IS NECESSARY AND STRONGLY ADVISED TO ACCURATELY AND PRECISELY LOCATE ALL PROPERTY CORNERS.

BENCH MARK

- BENCHMARK FOR THIS SURVEY IS POINT ID NO. 1025, DESIGNATED AS ENC-25, A BRASS DISC SET IN MONUMENT WELL, LOCATED IN THE INTERSECTION OF SOUTH EL PORTAL STREET AND LA MESA AVENUE, 100 FEET NORTH ALONG THE SOUTHEASTERLY CURB LINE OF SOUTH EL PORTAL STREET IN THE CITY OF ENCINITAS, CA., HAVING A PUBLISHED NAVD 88 ELEVATION OF 60.60'.

SURVEY NOTE:

TOPOGRAPHIC SURVEY WAS PREPARED BY RANCHO COASTAL ENGINEERING & SURVEYING, INC. JAKE D. LOGAN L.S. NO. 9042 OR UNDER HIS DIRECTION. IN CONFORMANCE WITH THE PROFESSIONAL LAND SURVEYORS ACT.
DATE OF SURVEY WAS DONE IN SEPTEMBER 2019.



NEPTUNE X-SECTION

NOT-TO-SCALE

5 2.5 0 5 10

SCALE IN FEET
SCALE: 1" = 5'

RANCHO COASTAL
ENGINEERING & SURVEYING
SINGLE SOURCE DEVELOPMENT CONSULTANT
310 VIA VERA CRUZ, #205
SAN MARCOS, CA. 92078
(760) 510-3152 Ph / (760) 510-3153 Fax

ENGINEER OF WORK

I HEREBY DECLARE THAT I AM THE ENGINEER OF WORK FOR THIS PROJECT AND THAT I HAVE EXERCISED RESPONSIBLE CHARGE OVER THE DESIGN OF THE PROJECT.

DATE: 05/28/24

DOUGLAS E. LOGAN
C 39726

EXPIRES: 12/31/25

REGISTERED PROFESSIONAL ENGINEER
DOUGLAS E. LOGAN
C 39726
CIVIL
STATE OF CALIFORNIA

PRIVATE CONTRACT

SHEET 1	CITY OF ENCINITAS APN: 256-352-03	2 SHEETS

EXISTING TOPOGRAPHY AND DEMOLITION PLAN:

MARC BEVAND
312 NEPTUNE AVENUE
ENCINITAS, CA 92024

MAILING ADDRESS:	TELEPHONE:
ENGINEER OF WORK: DOUGLAS E. LOGAN RCE 39726	CHECKED BY:
	JOB NO. 7811

PRELIMINARY — NOT FOR CONSTRUCTION

ENGINEER'S NAME: RANCHO COASTAL ENGINEERING & SURVEYING
PHONE NO. (760) 510-3152 // EMAIL: DOUG@RCESD.COM

December 6, 2024

Delivered via email

To: Karl Schwing
District Director, San Diego Coast
California Coastal Commission

Re: Th20a: Appeal No. A-6-ENC-24-0046 (Bevand, Encinitas)

Honorable Commissioners,

The Surfrider Foundation is a nonprofit grassroots organization dedicated to the protection and enjoyment of our world's ocean, waves, and beaches, for all people, through a powerful network. Thank you for the opportunity to comment on this project.

As an appellant of this project, Surfrider agrees with the grounds for appeal and supports the Staff Report's determination that Substantial Issue must be found for the proposed new development at 312 Neptune Ave., Encinitas CA. . We concur with all of Staff's cited issues as to why this development does not conform with the City of Encinitas' LCP.

First, the existing home at this property currently benefits from the protection of a large seawall in front of 312, 354, 370, 378, 396, and 402 Neptune Ave. This seawall was constructed in 1993 and enlarged in 1995. However, new development shall not require protective devices per the Coastal Act:

> *30253 Minimization of adverse impacts*
> *New development shall do all of the following:*
> *(b) Assure stability and structural integrity, and neither create nor contribute significantly to erosion, geologic instability, or destruction of the site or surrounding area or in any way require the construction of protective devices that would substantially alter natural landforms along bluffs and cliffs.*

The applicant's geotechnical analysis claims an annualized erosion rate of 0.22 ft/year, and that the seawall was not considered in the calculation of the erosion rate. However, this erosion rate does not align with the highly studied erosion rates determined by the Army Corps of Engineers and certified by the Coastal Commission via a Federal Consistency Determination for the Solana Beach - Encinitas Storm Damage Protection project. Appendix B of the Army Corps Coastal Engineering Encinitas-Solana Beach Shoreline Final Study[1] provides details on retreat rates in different segments of the cities, and the subject site at 312 Neptune Ave Encinitas is on the border between reaches 3 and 4:

Table 2.2-1 Study Area Reaches

Reach	Range		Approx. Length (mi)
	From	To	
1	Encinitas City Limit	Beacon's Beach	1.1
2	Beacon's Beach	700 Block, Neptune Ave.	0.3
3	700 Block, Neptune Ave.	Stone Steps	0.5
4	Stone Steps	Moonlight Beach	0.5
5	Moonlight Beach	Swami's	1.0
6	Swami's	San Elijo Lagoon Entrance	1.1
7	San Elijo Lagoon	Table Tops	1.2
8	Table Tops	Fletcher Cove	0.8
9	Fletcher Cove	Solana Beach City Limit	0.8

The Army Corps study determined retreat rates, even in the absence of sea level rise, to be 5 times the rate proposed by the applicant's geotechnical consultant. The Army Corps found 1.2 ft/yr for Reach 3, and 1.0 ft/yr for Reach 4, which is at odds with the applicant's geotechnical consultant's estimate of 0.22 ft/year:

Table 5.2-6 Modeled Bluff Retreat Averaged Over 1000 Simulations Under Without SLR conditions

Reach	Cumulative Bluff Retreat Over 50 years (ft)	Annualized Bluff Retreat (ft/yr)	Geologically Averaged Bluff Retreat Rate (ft/yr)*
1	0.0	0.0	0.2
2	14.1	0.3	0.3 – 0.5
3	80.4	1.6	1.2
4	44.3	0.9	1.0
5	48.6	1.0	0.2 – 0.6
6	0.3	0.007	0.1 – 1.0
7	N/A	N/A	N/A
8	83.7	1.7	0.4 – 1.2
9	92.5	1.9	0.4 – 1.2

Appendix C describes Reaches 3 and 4 in more detail:

[1] https://www.spl.usace.army.mil/Portals/17/docs/projectsstudies/Encinitas_Solana/Appendices_A_D_(VolumeII).pdf

7.3.3 Reach 3

As with the observed marine erosion, the upper bluff along this reach has experienced significant failures in the last seven years, particularly north of North El Portal, where today much of the upper bluff has near-vertical scarps, with significant sections of the upper bluff exceeding 70 degrees inclination. The upper terrace deposits are unstable at this inclination, and significant bluff failures are anticipated to continue in order for the upper bluff to reequilibrate, even with the low seawalls now protecting a significant portion of this reach. Although seawalls have essentially eliminated all marine erosion north of North El Portal up to the northern end of Reach 3, a 400±-foot section of coastal bluff remains highly unstable, with additional upper-bluff failures expected to reduce the currently oversteepened inclination of this section of coastal bluff. The southern portion of Reach 3, although not having experienced the same level of upper-bluff failures as the northern portion, currently has extensive notching at the base of the sea cliff and in the absence of seawalls, the entire sea cliff along this remaining unprotected south portion of the reach is expected to fail, with corresponding and significant upper-bluff failures. <u>The rate of bluff-top retreat for Reach 3, where unprotected by coastal bluff stabilization, is estimated to be 1.2 ft per year,</u> recognizing that the upper slopes in this area are currently very steep and much of the lower sea cliff exhibits significant notching indicative of to the bluff top, with bluff-top retreat rates approaching sea-cliff retreat rates.

7.3.4 Reach 4

Reach 4 is nearly identical to the south portion of Reach 3, south of North El Portal, with significant notching and a significant potential for sea-cliff type failures immediately triggering upper-bluff type failures. Nonetheless, Reaches 3 and 4 have been subdivided, with slightly less bluff-top retreat estimated over the next 50 years due primarily to the lack of extensive lower sea-cliff failures in Reach 4 and the associated more stable upper-bluff slopes, which will provide a modest lag in estimated bluff-top retreat compared to the rate of marine erosion. <u>The estimated rate of bluff-top retreat for the next 50 years in the absence of any stabilization measures is 1 foot per year, with the rate of sea-cliff retreat being 1.1 ft per year.</u>

It seems unlikely that the applicant's geotechnical consultant calculated the retreat rate in the absence of the seawall when compared to the determination made above in the expert information provided by the Army Corps of Engineering.

Second, we find the new 312 Neptune Ave development to be inconsistent with the city of Encinitas' Local Coastal Program, including but not limited to this Land Use Plan policy on page LU-50:

Coastal Bluffs: The coastal bluffs are part of the dynamic land-ocean interface that is continually changing. Changes in the patterns of weather, severe storms, and even man-made factors can accelerate the weathering processes that effect the coastline. In recent years, a number of homes and other improvements have been damaged due to bluff failure and there is no indication that these bluffs will become inactive in the near future. For this reason, future intensification of development near the bluff edges is discouraged under the land use policy.

LU-50

Replacement of an existing one-story 2,017-square foot duplex with a new two-story, 2,665-square foot single-family residence with conversion of a portion of the existing basement into 568-square foot ADU squarely falls under the category of '*intensification of development near the bluff edges,*' and per the Encinitas LCP should be discouraged.

Third, we find this development to be inconsistent with the city's Implementing Ordinances, including but not limited to this prohibition:

> *§ 30.34.020. Coastal Bluff Overlay Zone*
> *B.1. With the following exceptions, no principal structure, accessory structure, facility or improvement shall be constructed, placed or installed within 40 feet of the top edge of the coastal bluff. Exceptions are as follows:*
>
> a. *Principal and accessory structures closer than 40 feet but not closer than 25 feet from the top edge of the coastal bluff, as reviewed and approved pursuant to subsection C, Development Processing and Approval, of this section. This exception to allow a minimum setback of no less than 25 feet shall be limited to additions or expansions to existing principal structures which are already located seaward of the 40-foot coastal blufftop setback, provided the proposed addition or expansion is located no further seaward than the existing principal structure, is set back a minimum of 25 feet from the coastal blufftop*

edge and the applicant agrees to remove the proposed addition or expansion, either in part or entirely, should it become threatened in the future. <u>Any new construction shall be specifically designed and constructed such that it could be removed in the event of endangerment</u> and the property owner shall agree to participate in any comprehensive plan adopted by the City to address coastal bluff recession and shoreline erosion problems in the City.

Converting a basement to an ADU is inconsistent with the policy that requires the development to be removable in the future in event of endangerment.

For these reasons, along with the additional LCP inconsistencies pointed out in the Staff Report, we support finding Substantial Issue with the city of Encinitas' Planning Department's approval of this new development at 312 Neptune Ave. Thank you for the opportunity to comment.

Sincerely,

Kristin Brinner & Jim Jaffee
Co-Leads of the Beach Preservation Committee
San Diego County Chapter, Surfrider Foundation

Mitch Silverstein
San Diego County Policy Manager
Surfrider Foundation