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W17b

A-6-ENC-22-0059

December 14, 2022

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Coastal Property Rights, Land Use & Litigation

W17b

December 9, 2022

Hon. Chair Donne Brownsey and Commissioners
California Coastal Commission
455 Market St, Suite 300,
San Francisco, CA 94105

RE: Appeal No. A-6-ENC-22-0059 / Newman / 216 Neptune Avenue, Encinitas

Dear Hon. Chair Brownsey and Commissioners:

We represent Wes Newman, owner of the property located at 216 Neptune Avenue in the City of Encinitas. We have reviewed the Staff Report in preparation for the December 14, 2022, substantial issue (SI) hearing on the appeal of the City of Encinitas' approval of Coastal Development Permit No. 003343-2019.

The project approved by the City of Encinitas on September 15, 2022, allows for the construction of a new 3,825 sq. ft. two-story single-family residence with a 728 sq. ft. attached basement garage and a 1,054 sq. ft. basement living area on a 7,437 sq. ft. coastal bluff lot.

The November 23, 2022, Coastal Commission staff report centers on four issues addressed below.

Geologic Stability and Bluff Setbacks

The proposed project design has considered the specific constraints and natural features that exist on the project site. As a result, the project has been designed to comply with the requirements and criteria in the Coastal Bluff Overlay (EMC Section 30.24.020).

As required by the City's Local Coastal Program, a site-specific geotechnical analysis has been completed for the project by Geosoils, Inc. ("GSI") and reviewed by the City's third-party geotechnical consultant and the California Coastal Commission. This analysis demonstrated that the project would not have an adverse effect on the stability of the coastal bluff and would not endanger life and property. Further, GSI concluded that the existing structure with proposed improvements is expected to be reasonably safe from failure and erosion over its lifetime, without having to propose any new shore or bluff protection to protect the structure in the future, provided that the recommendations contained in the geotechnical study are correctly incorporated into the project design.

The Staff's first concern (that a 70-year design life was used) was merely the result of a typographical error in the City's September 15, 2022, staff report (*see* page 2). This 70-year number was not used in GSI's analysis or reports; instead, the correct 75-year analysis was performed. Elsewhere in the City's staff report, the correct 75-year figure was noted (*see* page 5). Specifically, the City staff report states: "*Based on the Geotechnical Report provided by Geosoils, Inc with concurrence from the City's Third-Party Geotechnical Engineer (GeoPacifica), the Factor of Safety plus Erosion Rate calculated setback is proposed at 51 feet from the bluff edge. The erosion rate used ranges from 1.4 to seven feet* ***over 75 years****. This is consistent with Coastal Commission's recommendation of a 50-foot minimum setback from bluff edge."* (emphasis added.)

Further confusing the issues, Coastal Staff appears to have limited their review to GSI's initial April 14, 2020, report. GSI submitted two additional letters: the geotechnical response letter to the City dated April 13, 2021, and the response letter to Coastal Staff dated March 22, 2022. Additionally, Coastal Staff did not consider the response letter to the City from GeoPacifica dated June 9, 2022. The reason why a 51-foot bluff setback was used is obscured by solely basing their analysis on the initial report. In short, the 51-foot setback was suggested by Coastal Staff in their February 3, 2022, comment letter to City staff (*see* page 2).

We are providing these facts to clarify the administrative record. The Coastal Commission Staff presented a letter report on the proposed project on February 3, 2022, which made several comments regarding site stability and geologic setback. The staff quoted and referenced a report by Benumoff/Griggs (1999) giving erosion rates for Encinitas bluffs. However, based upon statements of the report's principal author, Mr. Ben Benumoff, the study has been misused, and its data are taken out of context. The erosion rate quoted in the study was intended for site-specific locations along the Encinitas coastline.

The Coastal Commission Staff letter also referenced the Scape equation (Ashton et al. 2011) and took the position that the GSI may have used a lower site-specific factor than appropriate, citing examples of differing rates. The Staff letter also references USGS CoSMoS bluff retreat tool. The City of Encinitas has yet to adopt this tool.

GSI responded to the comments by the Coastal Commission Staff in a report dated March 22, 2022. GSI presented rebuttals to the use of Benumoff/Griggs erosion rates, the SCAPE equation of m = 0.5, and the referenced USGS CoSMoS bluff retreat rates. GSI concluded that the recommended 40-foot setback is valid based on actual physical science and the data used by the consultant. Based upon an additional evaluation of the SCAPE equation and comments from Coastal Staff suggesting a 51-foot setback, GSI concluded that although a setback of 40 feet would be valid, Staff's suggested setback of 51 feet could be used instead as a conservative setback based on the use of the erosion rates requested by Coastal Staff.

The 51-foot setback is further supported by the approval of a 52-foot setback for a proposed structure in the 100 block of 5th Street in Encinitas, just south of the proposed project. The factor of safety setback and erosion rates were almost identical to the values used for this project. Therefore, the blufftop setback should no longer be considered an issue.

Future Shoreline Protection and Assumption of Risk

According to the Staff Report, "*The City 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. Section 30.34.020(D) prohibits new development from requiring future shoreline protection*."

Although not explicitly titled as a shoreline protection waiver restriction, Condition BD 03 states: "*The property owner/applicant shall execute and record a covenant to the satisfaction of the Development Services Department setting forth the terms and conditions of this approval prior to the issuance of Building Permits. Said covenant shall also provide that the property owner shall be responsible for maintaining the approved structure(s) in good visual and structural condition in a manner satisfactory to the Development Services Director.*"

Mr. Newman accepts Coastal Staff's requirement to waive any future rights to shoreline protection. Therefore, the project is consistent with Section 30.34.020(D). Further, Mr. Newman accepts the assumption of risk to ensure that current and future property owners are aware of the limitations on the site and deter any future requests for shoreline protection. Therefore, this can no longer be considered an issue.

Basement and Future Removal of Development

According to the Staff Report, "*…the proposed basement walls could act as shoreline protection in the future if erosion occurs on the site, inconsistent with Section 30.34.020.C.2.c of the certified IP. Once exposed, a basement would essentially serve the same purpose as a shoreline protective device in the same manner that caissons and deepened foundations do*."

Due to the natural topography of this lot, this property cannot incorporate an integral garage without the garage being defined as a "basement." The driveway at the street is at elevation +63.0'. The middle of the site is at approximately +76.0'. Given this steep slope, it is not possible to construct the integral garage outside of the front yard setback without it being technically defined as a "basement."

The position that a basement would serve as a shoreline protection device, such as caissons and deepened foundations, is not backed by engineering science or geology. For example, caissons on the bluff generally run 60 feet to 90 feet in depth, depending on the geology and location. This type of shoreline engineering would allow a structure to remain structurally stable, even if the bluff eroded or failed to the building line or beyond. Other retention devices consist of walls holding up the bluff, which are cantilevered or tied back into the bluff. A basement wall and footing setback 51 feet from the bluff edge, approximately 11'-0" below the adjacent grade, would not function as a shoreline protection device. If the bluff receded to a point near the basement, the basement would be in the same peril as the structure above, and removal could be required. Moreover, from a structural and geologic perspective, a basement lowers the surcharge from the building resulting in lower forces acting upon the bluff.

The construction of a basement is technically possible and does not violate any requirements of the City of Encinitas. For example, City Condition of Approval SPCL 02 states, "*Special – Shoring & Dewatering: If applicable, the developer shall design and have approved the shoring and construction dewatering systems necessary for the construction of the underground basement prior to issuance of any grading permit for the project. If dewatering is necessary, the appropriate permits shall be obtained from all applicable agencies."* Further, City Condition of Approval SPCL 038 states, "*Special – Dewatering: No permanent dewatering system shall be allowed for the underground basement. The underground basement shall be designed to withstand the hydrostatic pressure without any dewatering*."

The Staff Report incorrectly states that a basement cannot be removed safely in case of endangerment. This is inaccurate and is not supported. Like any other structure, a basement can be safely removed through careful demolition. Removing a standard concrete slab on grade foundation or a raised stem wall foundation would require the same process as removing a basement.

The City of Encinitas has a long history with the California Coastal Commission of processing and approving coastal bluff property construction with proposed basements. Such examples include both 824 and 828 Neptune. And for almost 30 years, the Coastal Commission has approved numerous blufftop projects in the City of Encinitas and has allowed the construction of basements. There is no data or support to justify denying the construction of a basement on a coastal bluff. Here, due to the steep topography to build a residence on the property, the integral garage will necessarily be defined as a "basement." Removing basements if the blufftop structure is endangered is technically and physically possible. Therefore, this should no longer be considered an issue.

Visual Resources

According to the Staff Report, "*The City did not ensure the project preserves public views or prevent a walling off effect from Neptune Avenue by establishing view corridors within the north and south side yards of the site or including conditions that restrict the height of landscaping and require any fencing/gate materials to have at least 75% of their surface open to light*."

Due to the steep natural upward slope from the street, no ocean views currently exist through the site. The middle of the site is approximately 13 feet higher than the street level. Please refer to Diagrams 1 and 2, which show both the north and south sides:

Diagram 1 – South Side Perspective:

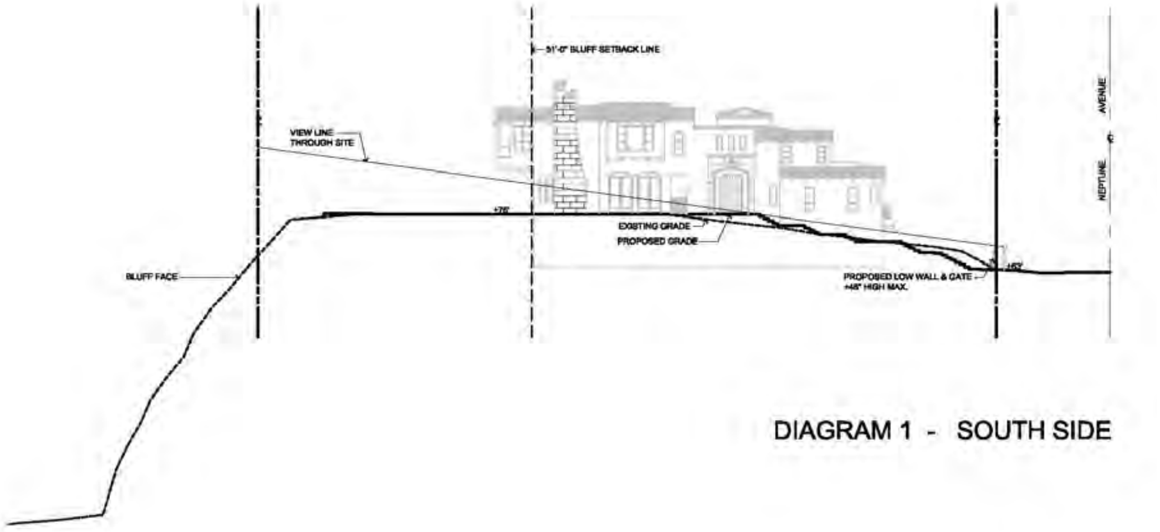
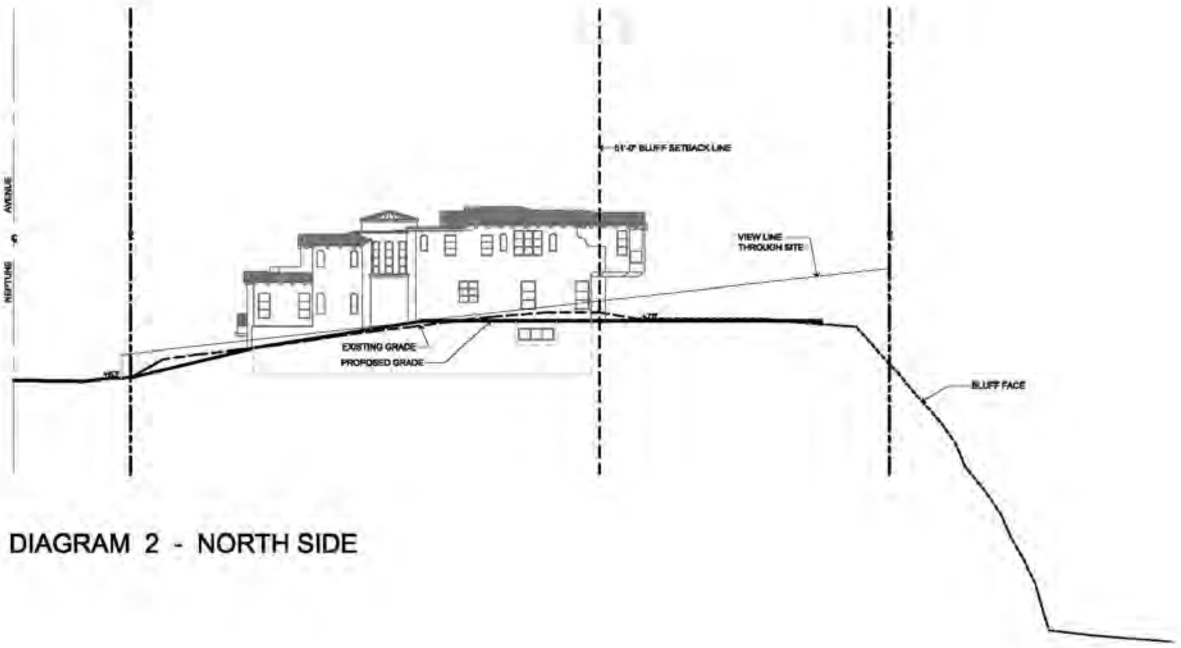


Diagram 2 – North Side Perspective:



As illustrated, there are only views of the sky high above the natural grade and ocean level. On the North side, there is a retention basin approximately 30” above the adjacent grade. On the South side, there is a wall and gate 48” above the adjacent grade. These structures will neither impair the site's blue-sky views nor create a “walled-off effect.” Nonetheless, Mr. Newman will agree to modify the gate and wall to be 75% open and modify the landscaping to limit its height to 36.” Therefore, this should no longer be considered an issue.

Conclusion

On behalf of Mr. Newman, we respectfully request that the Commission find no substantial issue. The coastal bluff setback is consistent with Coastal Bluff Overlay (EMC Section 30.24.020); the applicant is accepting of conditions of approval related to Future Shoreline Protection and Assumption of Risk; the basement is necessary, would not act as shoreline protection and can be safely removed; and the applicant will modify the gate and wall to be 75% open and modify the landscaping to limit its height to 36.”

We appreciate your time and consideration of this matter and will be available for questions at the hearing.

Sincerely,

AANNNESTAD ANDELIN & CORN, LLP

Anders Aannestad
Managing Partner

Exhibits:

1. GeoSoils Inc’s Report dated April 14, 2020
2. GeoSoils Inc’s Response Letter to the City of Encinitas Staff dated April 13, 2021
3. GeoSoils Inc’s Response Letter to Coastal Commission Staff dated March 22, 2022
4. GeoPacifica Review Letter to the City of Encinitas Staff dated June 9, 2022

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT, 216 NEPTUNE AVENUE
ENCINITAS, SAN DIEGO COUNTY, CALIFORNIA 92024
ASSESSOR'S PARCEL NUMBER (APN) 256-352-18-00

FOR

MR. WESLEY NEWMAN
2033 SAN ELIJO AVENUE, #131
CARDIFF, CALIFORNIA 92007

W.O. 7557-A-SC APRIL 14, 2020

GeoSoils, Inc.
Geotechnical • Geologic • Coastal • Environmental

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

April 14, 2020

W.O. 7557-A-SC

Mr. Wesley Newman
2033 San Elijo Avenue, #131
Cardiff, California 92007

Subject: Preliminary Geotechnical Investigation, Proposed Residential Development, 216 Neptune Avenue, Encinitas, San Diego County, California 92024, Assessor's Parcel Number (APN) 256-352-18-00

Dear Mr. Newman:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) has performed a preliminary geotechnical investigation of the subject site. The purpose of our study was to evaluate the onsite geologic and geomorphic conditions relative to the proposed single-family residential development at the subject property, and to provide preliminary geotechnical recommendations for site earthwork and the design of foundations, slab-on-grade floors, retaining walls, flatwork, and other improvements possibly 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:

- Our slope stability analyses indicate that the proposed residential structure would have acceptable static and seismic factors-of-safety (FOS) against deep-seated bluff instability, and 75 years of coastal erosion, if setback 40 feet from the coastal bluff edge.

- The project design civil engineer, Pasco Laret Suiter & Associates (PLSA), performed a photogrammetric analysis to determine the site specific historical erosion rate (PLSA, 2020), over the period 1932 to 2018 (86 years). They were able to demonstrate that the site specific historical erosion rate of the edge of bluff ranged between a low of 1.6 feet/86 years to a high of 7.8 feet/86 year. This is equivalent to a site specific historic rate of 0.0186 (low) to 0.0907 feet/year, and corresponds to a buff retreat rate ranging from about 1.4 to to 7 feet over 75 years. This site specific data is considered the best available science in this regard. More aggressive rates were weighed in our analysis, however, these rates were not indicated on the aerial photographs reviewed (as far back as 1932).

- 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 36 years (2020-2056), 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 assumed SLR will increase the bluff retreat rate by ⅓ the change in the rate of bluff retreat in the year 2095, for the 30-year period of 2020-2050, 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 (2079-2095), GSI has assumed that the bluff retreat rate will be that of the year 2095, 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 0.019-0.091 ft/yr from 2020-2050, 0.027-0.13 ft/yr for 2051-2080, and 0.043-0.207 ft/yr for 2081-2095, derived from site specific historical retreat rates (considered the best available science), being influenced by postulated SLR. The rates are discussed further herein. 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, the site is mantled by localized areas of undocumented fill and colluvium (topsoil). These surficial soil 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 exist at the toe of the bluff, also underlain by Torrey Sandstone.

- Earth materials considered unsuitable for the support of proposed settlement-sensitive improvements (i.e., residential structure, underground utilities, walls, hardscape, etc.) and/or planned engineered fills consist of undocumented artificial fill, colluvium (topsoil) 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. Based on the available data, the thickness of unsuitable earth materials in the area of proposed development are on the order of approximately 2½ feet, with localized areas that may be thicker/deeper. All unsuitable earth materials should be removed to expose unweathered old paralic deposits prior to fill placement or foundation construction, if not removed by default during earthwork for design grades.

- Laboratory testing, performed on a representative sample of the onsite soils, indicates an expansion index (E.I.) less than 5. This corresponds to a very low expansion potential. Thus, the tested onsite soils do not meet the criteria of 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 surficial soils, indicates that the soils are neutral with respect to soil acidity/alkalinity; are corrosive to exposed, buried metals when saturated; possess negligible (S0 per American Concrete Institute [ACI] 318-14) sulfate exposure to concrete; and contain 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, minimally considering exposure conditions S0, W1, and C1, per ACI 318-14 . Owing to the site's proximity to the Pacific Ocean, the effects of sea spray/fog 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 fills and colluvium, should conform to CAL-OSHA and/or OSHA requirements for Type "B" soil conditions (i.e., 1:1 [horizontal:vertical {h:v}] gradient) provided that perched 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.

- The regional groundwater table is considered nearly coincident with sea level. GSI did not encounter groundwater nor evidence of perched water in our hand-auger borings, nor in the boring on the adjoining property, to the explored depths, nor is it exiting the bluff face. A review of oblique aerial photographs (Appendix A), did not indicate groundwater perched on the top of the Torrey Sandstone exposed in the bluff, as far back as 1972. Thus, regional groundwater is not anticipated to significantly affect the proposed site development. Perched groundwater typically occurs along boundaries of contrasting permeability and density (i.e., fill/bedrock contacts, sandy/clayey fill lifts, etc.), and along geologic discontinuities (i.e., fractures, joints, etc.). The potential for perched water to manifest should be anticipated and disclosed to all interested/affected parties. Perched water manifestation is based on numerous factors including, but not limited to: site geologic conditions, rainfall, irrigation, broken or damaged wet utilities, etc.

- Our evaluation indicates there are no known active faults crossing the site. Thus, the potential for surface fault rupture to affect the existing and any future 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 and the dense nature of the old paralic deposits and Torrey Sandstone Formation, 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 any future development. The adverse effects of seismic shaking on the structure(s) will likely be wall cracks, some foundation/slab distress, and some seismic settlement. However, it is anticipated that the structure will be repairable in the event of the design seismic event. This potential should be disclosed to all interested/affected parties.

- 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 BMP consideration is not recommended.

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

- The proposed project 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).

- 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 residential structure 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.

- 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.

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PROFESSIONAL GEOLOGIST
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Distribution: (3) Addressee (3 wet signed)

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PRELIMINARY GEOTECHNICAL EVALUATION PROPOSED RESIDENTIAL DEVELOPMENT, 216 NEPTUNE AVENUE ENCINITAS, SAN DIEGO COUNTY, CALIFORNIA 92024 ASSESSOR'S PARCEL NUMBER (APN) 256-352-18-00

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 geotechnical report for the adjoining southerly property (see Appendix A).

2. Delineating the coastal bluff edge in the field.

3. Geologic site reconnaissance, mapping, and subsurface exploration with three (3) hand-auger borings to evaluate the near-surface soil and geologic conditions, and one (1) track-mounted hollow stem auger boring to evaluate the soils and geologic profile, and sample onsite earth materials (see Appendix B). The boring log for the adjacent 210 Neptune Avenue is also provided.

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 by GSI (2000), for the adjacent southerly property. The laboratory test results are provided in Appendix D.

6. Engineering and geologic analysis of data collected, including an evaluation 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 pan handle-shaped lot of undeveloped residential property located at 216 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 upper reaches of the subject site is 33.052809, -117.299814. The property is situated above an approximately 75- to 78-foot high coastal bluff slope, descending toward the Pacific Ocean. The pan handle portion of the lot is located within the bluff, and extends to the mean high tide line. The property is bounded by the Neptune Avenue right-of-way to the east, by the aforementioned coastal bluff to the west, and by existing residential development to the north and south. According to the 10-scale topographic survey provided by the project architect, Cohn + Associates ([Cohn], undated), site elevations across the property vary

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

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GeoSoils, Inc.

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SITE LOCATION MAP

Figure 1

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between approximately 4½ feet on the beach, to the bluff top at about 78 feet (North American Vertical Datum of 1988 [NAVD88]), for an overall relief of about 73½ feet.

Topographically, the upper central portion of the top of the site is relatively level to gently southwesterly sloping at gradients on the order of 12:1 (horizontal:vertical [h:v]) or flatter, increasing to the west on the bluff itself to as steep as about 0.65:1 to 2.2:1, and locally steeper and flatter. Proceeding east from the central portion of the top of the site, it flattens to ±8:1, descending to about 1:1 fronting Neptune Avenue. Existing improvements include a wooden fence just above the aforementioned steeper slope above Neptune Avenue. Vegetation is sparse and consists of native weeds and grasses, along with ice plant on the east portion of the property. Site drainage appears to be accommodated by sheet flow run-off directed uncontrolled toward the southwest and channeled to the east, where it is discharged into Neptune Avenue.

Proposed Development

Based on conversations with the owner, GSI understands that proposed residential development at the site consists preparing the site to receive a one- or two-story residential structure, perhaps with a basement, and associated retaining walls, with hardscape and perimeter wall improvements. Grading for the proposed development may require minor fills, near the easterly property line. Building loads are currently unknown but assumed to be typical for residential construction. Sanitary sewage disposal is anticipated to be tied into the municipal system.

ADJOINING GEOTECHNICAL STUDY

GSI performed a geotechnical investigation on the adjoining property to the south, in August of 2000. That evaluation included geologic mapping of exposed conditions, including bedding and joint/fracture attitudes; subsurface exploration consisting of the drilling of one relatively deep (about 47½ feet) exploratory hollow-stem auger boring 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. That boring, laboratory, and field work were reviewed and are included herein, and modified as appropriate.

SITE EXPLORATION

Surface observations and shallow subsurface explorations were performed on January 3, 2019, by a geologist from this firm. Near-surface soil and geologic conditions were explored with three (3) hand-auger borings. Subsequently, GSI also met with the City of Encinitas geotechnical consultant, Mr. Jim Knowlton, and mutually agreed to the location of the coastal bluff edge location, which has been surveyed. In addition, to further assess site soil strengths and geologic conditions, on March 9, 2020 a track-mounted hollow stem auger was advanced to a depth of 46 feet to further evaluate the site soil strengths and geologic profile. Representative undisturbed (in-situ) ring and bulk samples were obtained and transported to the laboratory for testing. The approximate locations of the geologic contacts, bluff top, and borings are shown on the Geotechnical Map (see Plate 1), which uses Cohn/PLSA (undated) as a base. A cross-section (X-X') depicting the subsurface conditions is provided as Plate 2. Logs of the borings 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 sea cliff; and an upper bluff slope generally ranging in inclination between about 20 and 80 degrees (measured from the horizontal). The bluff top or bluff edge is the boundary between the upper bluff and the relatively flat lying to gently sloping coastal terrace.

Offshore from the sea cliff 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 sea cliff, 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 sea cliff (the lower vertical and near-vertical section of the bluff) 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 cliff. 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. 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, our planet 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 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 (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 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 in a local area. 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

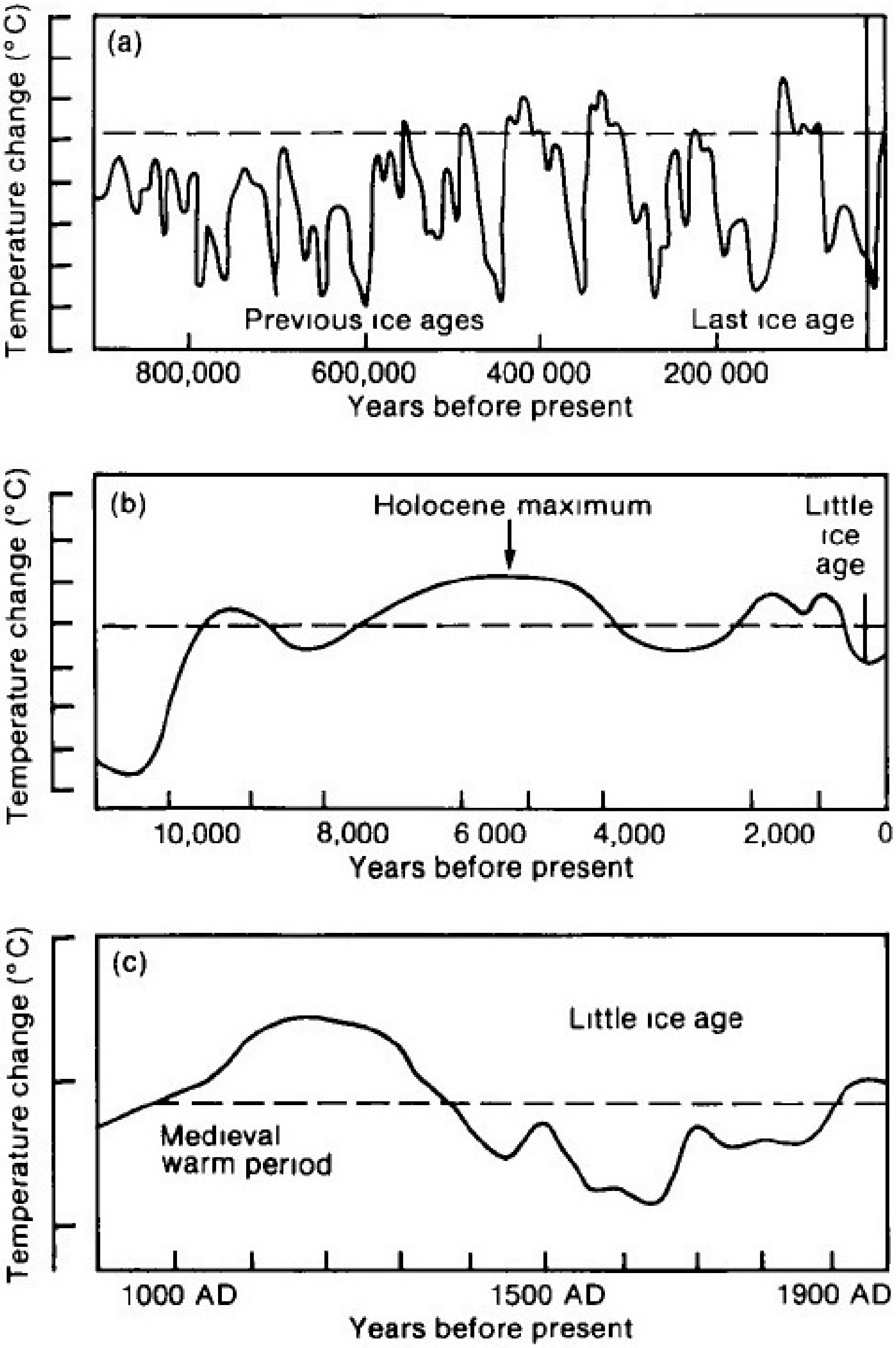


Figure 2 (from Figure 7.1 [IPCC, 1990]): Schematic diagrams of global temperature variations since the Pleistocene on three time scales (a) the last million years (b) the last ten thousand years and (c) the last thousand years. The dotted line nominally represents conditions near the beginning of the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

present inter-glacial warm period (they 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 was 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 a currently 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 Nurem, 2005; Nurem, et al., 2006, Nurem, 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 in the US west coast 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:

Sea-Level Change Curve Calculator

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"

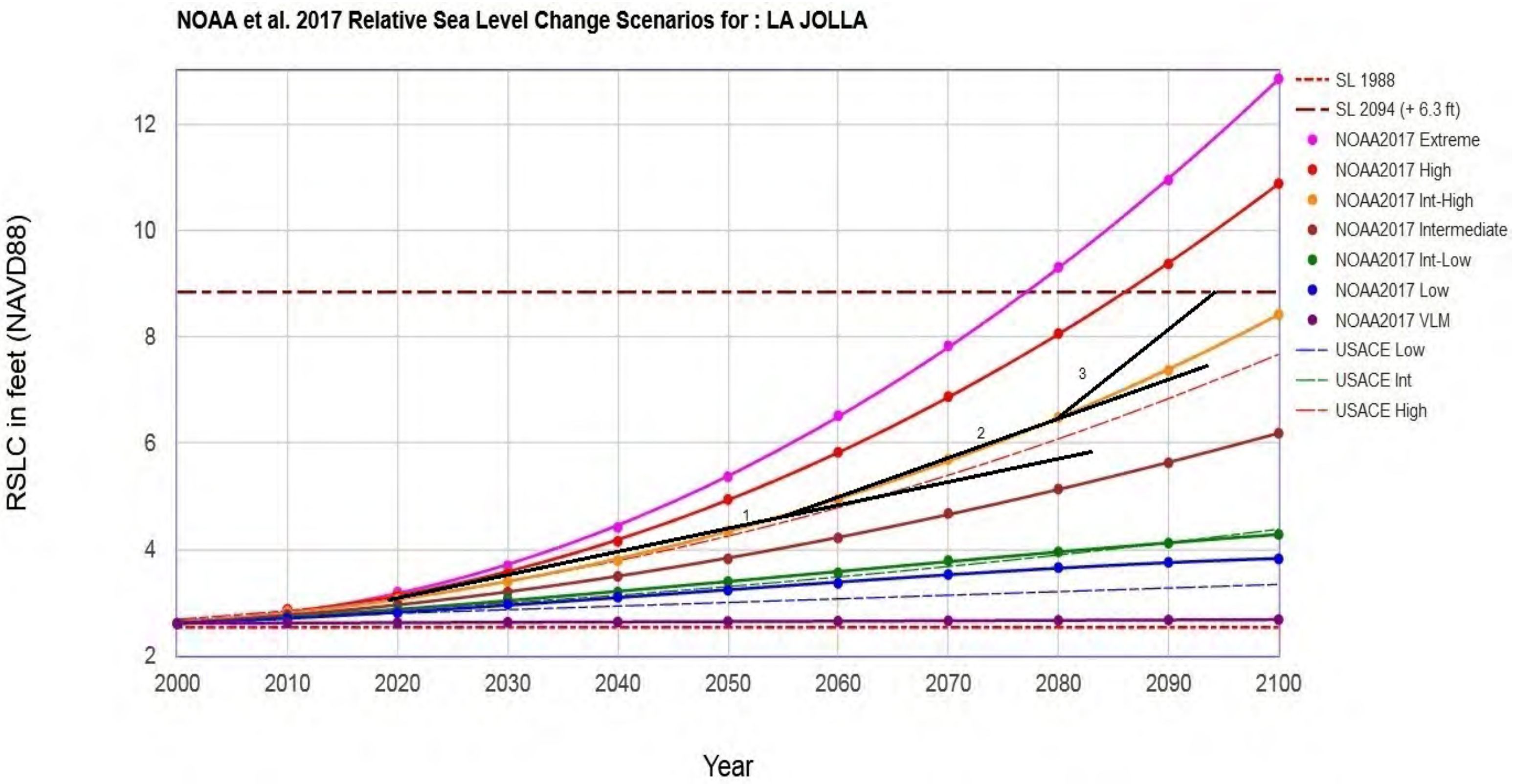


Figure 3 - Sea-Level Change Curve

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.

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 2020 to the year 2056; from 2057 to 2081, and from 2082 to 2095 (the end of the design life). These three linear portions are discussed further, later in the text.

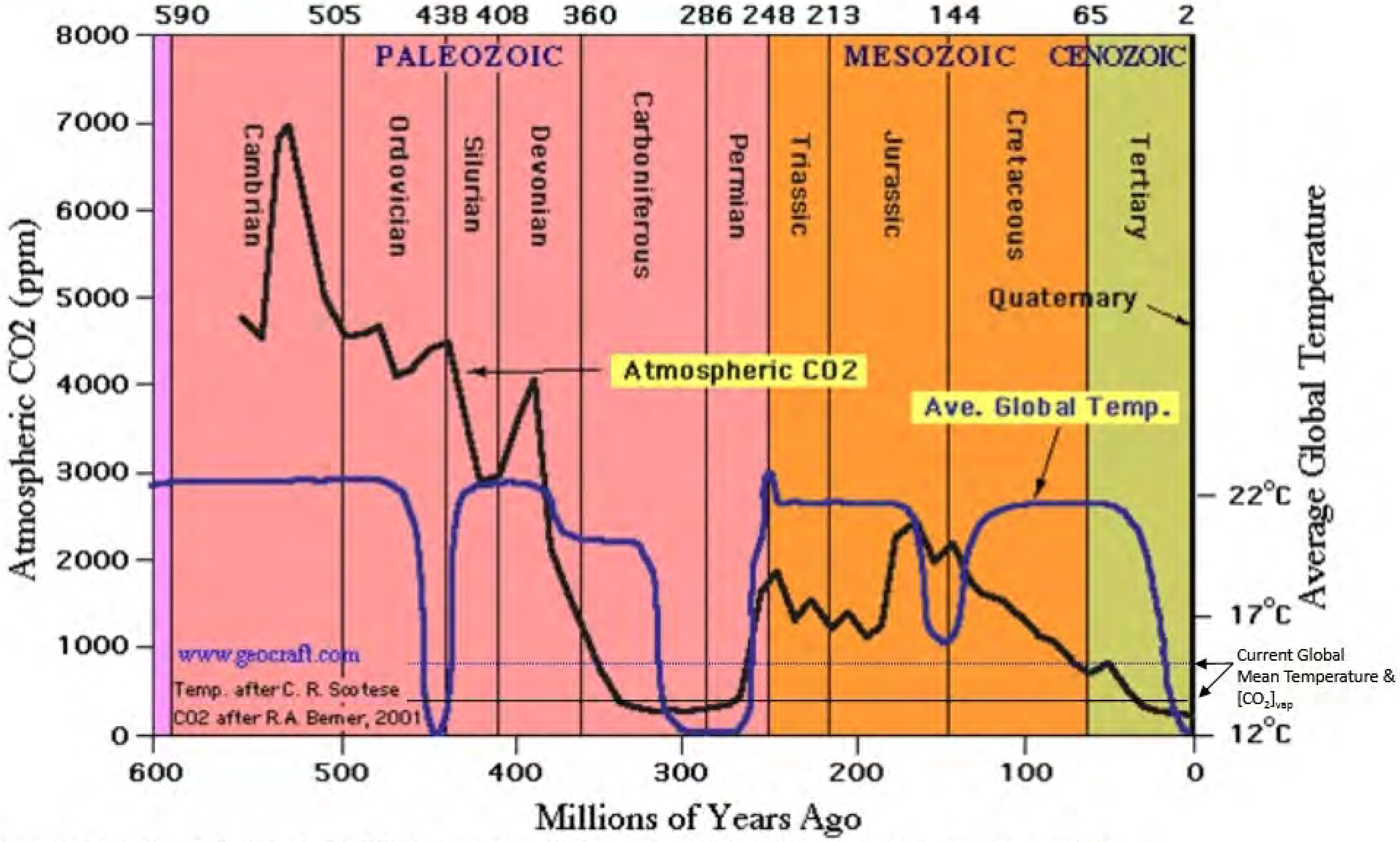
Computer climate models make an enormous range of assumptions and have not been able to accurately predict short-term observed climate changes. These models use assumptions that are manipulated, and parameters that are adjusted to produce a range of SLR scenarios. Whether all this tampering and adjusting 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, and further point out that the high risk scenario follows current greenhouse emissions tracking. CCC (2018) indicates that this range of SLR is the "best available science" in spite of the lack of scientific consensus. In fact, CO_2 has a 140 million year trend of decreasing atmospheric concentration (Berner, 2001; Wrightstone, 2017), to historic and current levels (±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 CO_2, which comprises a mere ±6 percent of all greenhouse gasses) 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 6.3 feet above current sea level at La Jolla (closest available projection in CCC [2018]).

Based upon the available information, the use of a 6.3-foot rise in sea level over the design life of the improvements to the property is conservative (0.5% probability that SLR exceeds this amount [CCC, 2018]). Due to the relatively hig elevation of the proposed development (approximately ±75 to ±78 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 6.3 feet over the 75-year life of the development (0.5% probability that SLR exceeds this amount, per CCC[2018]), from 2020 to 2095 (year 2095 extrapolated from 2090 to 2100 data).

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 sea cliff portion of coastal bluffs is exposed to direct wave attack along most of the coast. The waves erode the sea cliff 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 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 erosion and bluff instability.



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

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. At the subject site, there is evidence of backwearing near the toe of the bluff.

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, the sea cliff is 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 sea cliff, the "water-hammer" effect tends to cause hydraulic fracturing which exacerbates sea cliff 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 sea cliff 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 (Kuhn and Shepard, 1980). During our field work, nor during our review of oblique aerial photographs (see Appendix A), generally, GSI did not observe evidence of groundwater seepage near the toe of the bluff, nor at the overlying old paralic deposits/bedrock contact; however, occasionally there was a hint of nearby vegetation at the base of the cliff (seeking fresh water), and this was considered in our slope stablity 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 sea cliff fails by blockfall or rockfall, an over-steepened bluff face is left, with the debris at the toe of the sea cliff. 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 35 to 40 degrees range may indicate ages in the 75- to 100-year range. Steeper slopes indicate a younger age. Slopes angles at the site range from about 40 degrees (upper bluff [old paralic deposits]) to 65+ degrees (lower bluff [Torrey Sandstone]), 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 result in the creation of water paths that concentrate surface water runoff toward the bluff edge. In addition, drains, gutters, and downspouts 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.

Wave-induced marine erosion is characterized by wave notching at the bluff face resulting in failures originating from the bottom of the bluff upward. Based on site observations and our review of oblique aerial photographs available on the California Coastal Records Project website, there appears to be some notching along the toe of the bluff. Thus, the potential for toppling failures to occur in this area are higher than the portion of the bluff near the northwesterly property corner.

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 sea cliff. 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 7.70 cm/yr (3.03 in/yr) with a standard deviation of 2.31 cm/yr (0.91 in/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 PLSA [2020]), did not support the aggressive Benumoff and Griggs retreat rates.

The project design civil engineer, Pasco Laret Suiter & Associates (PLSA), performed a photogrammetric analysis to determine the site specific historical erosion rate (PLSA, 2020), over the period 1932 to 2018 (86 years). They were able to demonstrate that the site specific historical erosion rate of the edge of bluff ranged between a low of 1.6 feet/86 years to a high of 7.8 feet/86 year. This is equivalent to a site specific historic rate of 0.0186 (low) to 0.0907 (high) feet/year, and corresponds to a buff retreat rate ranging from about 1.4 to 7 feet over 75 years. This site specific data is considered the best available science in this regard.

FUTURE LONG-TERM BLUFF RETREAT RATE

Lately, the CCC has also been utilizing the online application CoSMos to bolster their claims of increased bluff erosion under SLR. The 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. GSI points out that the CoSMoS website contains a disclaimer stating that, ***"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."***

Neither the Hapke and Reid (2007) nor the Hapke, et al. (2006) reports are intended to override comprehensive, detailed site-specific analysis of cliff retreat and annualized retreat rate. In addition, the Hapke and Reid (2007) study explicitly reports a retreat rate uncertainty of 0.2 m/yr (0.656 ft/yr), which is an order of magnitude greater than the bluff retreat rate we have calculated for the Newman property. CoSMoS 3.0 provides detailed predictions (meter-scale) 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 a past rate to predict the future rate, if the past rate is incorrect, then the future rate intuitively would be incorrect, 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.

> ***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 Mr. Patrick Limber of the USGS. In their correspondence, Mr. 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 at the Newman property using 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. The output of the CoSMoS modeling is provided as Figure 5. As shown in Figure 5, the CoSMoS output does not correctly identify the location of the bluff edge. This is because the program is not accurate and explicitly is not intended to be used for site-specific analysis. The use of a very low SLR estimate in the CoSMoS application should produce a future bluff edge location that is nearly identical to its 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 18.5 feet of bluff retreat with 25 cm SLR. This suggests a retreat of less than 18.5 feet by the year 2095, or a rate of approximately 0.25 ft/yr. As previously stated, our site-specific evaluation of long-term bluff retreat by physical surveys (Benumof & Griggs, 1999), was estimated to be 0.328 ft/yr from 1932 to 1999, more than the CoSMos modeling of 0.25 ft/yr for the next 81years (until the year 2100), with 25 cm (0.82 ft) of SLR. This future retreat rate estimate is not reconcilable with the hypothesis that SLR will greatly affect bluff retreat, given that it is essentially the same as the regional Benumof & Griggs historic rate for this reach. This does, however, gives credence to the reasonable conclusion that future SLR should have little, if any, impact on the site.

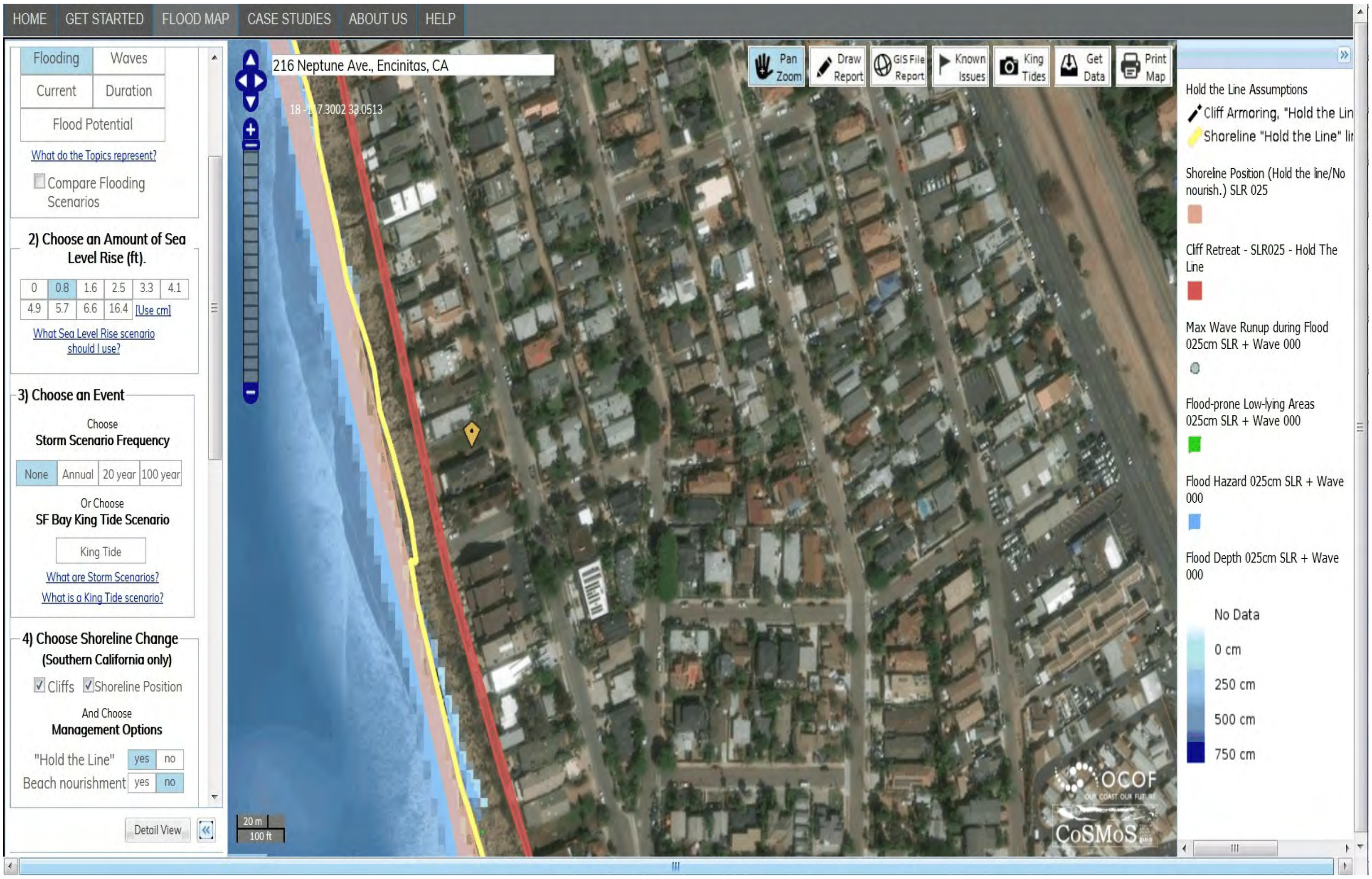


Figure 5. Our Coast Our Future (OCOF) CoSMoS output showing the extent of regional coastal bluff retreat in the year 2100, assuming 25 cm (0.82 ft) of SLR.

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 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); 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 Plate1).

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 (“SCAPE”) 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. 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) include eroding shorelines with negative feedbacks, such as high earth material strengths or a protective beach that reduce 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) include 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.

Coastal Bluff Lithology

The lithology of the onsite coastal bluff likely provides the greatest dampening effect on marine erosion. As shown on Plate 1, wave attack will still be focused on the more resistant Torrey Sandstone formation rather than the more erodible old paralic deposits over the design life of the proposed residential structure, 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 (i.e., induration/cementation or hardness), 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 design life of the proposed residential structure. 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 design 6.3 feet of SLR will have occurred). Given the closeness to the BB:0 line, m= 0.333 (or ⅓) appears appropriate for this site.

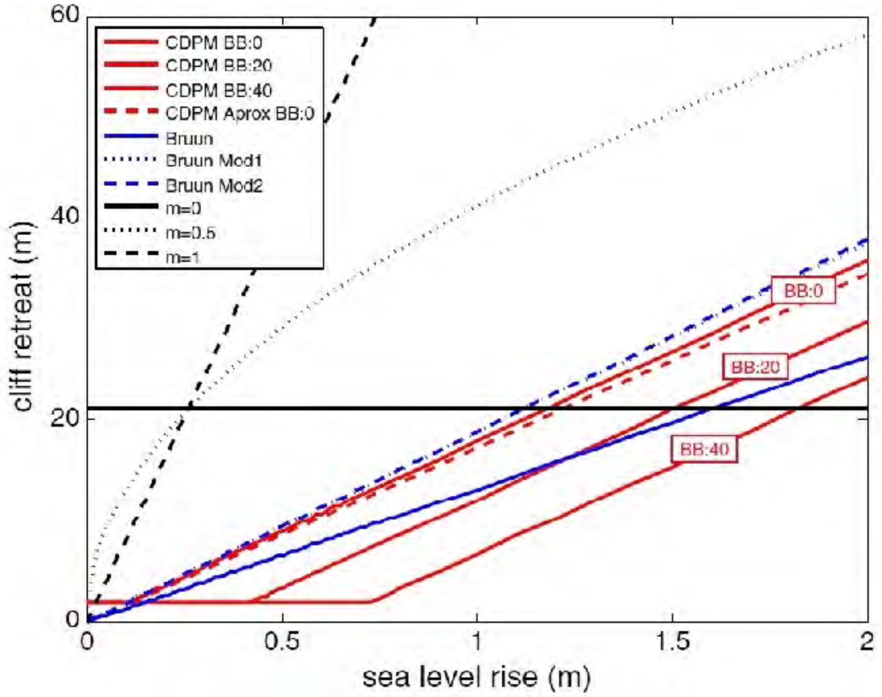


Fig. 12. Comparison of the conditionally decoupled profile model (CDPM) with 0, 20, and 40 m beach buffers (BB) and original Bruun, modified Bruun (Bruun Mod1 and Mod2), no feedback ($m = 0$), approximate SCAPE ($m = 0.5$), and linear extrapolation ($m = 1$). Exponent m models are based on historical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

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

Sediment Contributions from the Onsite and Nearby Coastal Bluffs

Sieve analysis tests performed on samples of the typical coastal bluff earth materials indicates that the 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.

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 (PLSA, 2020), is between 0.019 and 0.091 ft/yr = R_1.
2. Avg SLR rate over 87 years (1932 to 2019), based on NOAA (Gloss Station Handbook Scripps Pier, La Jolla) is 2.148 mm/yr = 0.085 inch/yr x 1 ft/12 in = 0.007083 ft/yr = S1
3. Future SLR rate (2095), under *medium-high risk aversion scenario* = 6.3 ft/75 yrs = 0.084 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 2056). We have utilized the endpoints of the range of 0.019 ft/yr and 0.091 ft/yr for this time interval. The erosion rate should marginally increase for the following 25 years (2057-2081), and we have reasonably added ⅓ of the change in erosion rate in 2095, to the initial erosion rate. During the more asymptotic SLR end of the 75-year design life (2082-2095), 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 2095, under *medium-high risk aversion scenario (0.5% Probability)*,

$$R_2 = R_1\ (S_2/S_1)^m$$
$$R_2 = (0.019\text{ ft/yr})\ (0.084\text{ ft/yr/[}0.007083\text{ ft/yr]})^{1/3}$$
$$R_2 = (0.019)\ (11.86)^{1/3}$$
$$R_2 = (0.019)(2.28) = 0.043\text{ ft/yr in the year 2095.}$$

Based on the above, the retreat rate will change from 0.019 to 0.043 ft/yr, and the difference between the 75-year commencement and end of the design life, Δ= 0.024 ft/yr, from 2020 to 2095.

FUTURE BLUFF RETREAT BASED ON SLR CURVE INCREMENTS			
APPLICABLE DATES	**BLUFF RETREAT RATE (FT/YR)**	**DURATION (YEARS)**	**BLUFF RETREAT (FEET)**
2020-2056 (0.019) SLR rate	0.019	37	0.70
2057-2081 (0.019 + ⅓[0.024]= 0.027) increase in SLR rate	0.027	25	0.68
2082-2095 (Calculated SLR rate in 2095)	0.043	13	0.56
Totals		**75**	**1.94**

As shown above, the onsite coastal bluff may experience approximately 2 feet of retreat over the 75-year design life of the proposed residential structure. 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 2095, based on the ±2 feet of bluff retreat, with an assumed SLR of 6.3 feet over that interval.

High Site Specific Rate

At year 2095, assuming the high site specific rate applied under *medium-high risk aversion scenario (0.5% Probability)*,

$$R_2 = R_1\ (S_2/S_1)^m$$
$$R_2 = (0.091\text{ ft/yr})\ (0.084\text{ ft/yr/[}0.007083\text{ ft/yr]})^{1/3}$$
$$R_2 = (0.091)\ (11.86)^{1/3}$$
$$R_2 = (0.091)(2.28) = 0.207\text{ ft/yr in the year 2095.}$$

Based on the above, the retreat rate will change from 0.091 to 0.207 ft/yr, and the difference between the 75-year commencement and end of the design life, Δ= 0.116 ft/yr, from 2020 to 2095.

FUTURE BLUFF RETREAT BASED ON SLR CURVE INCREMENTS			
APPLICABLE DATES	**BLUFF RETREAT RATE (FT/YR)**	**DURATION (YEARS)**	**BLUFF RETREAT (FEET)**
2020-2056 (0.091) SLR rate	0.091	37	3.37
2057-2081 (0.091 + ⅓[0.116]= 0.039) increase in SLR rate	0.130	25	3.25
2082-2095 (Calculated SLR rate in 2095)	0.207	13	2.69
Totals		**75**	**9.31**

As shown above, using the high rate, the onsite coastal bluff may experience approximately 10 feet of retreat over the 75-year design life of the proposed residential structure. 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 2095, based on the ±10 feet of bluff retreat, with an assumed SLR of 6.3 feet over that interval. We note that regardless of the site specific erosion rate utilized, when added to the static FOS line from the bluff edge, these rates plot well inside of the City of Encinitas 40-foot bluff-edge setback zone.

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 sedimentation occurred 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 sea cliff; a depositional characteristic more commonly associated with the Torrey Sandstone. The presence of the Torrey Sandstone in the sea cliff 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 site geologic units encountered during our subsurface investigation and site reconnaissance included localized areas of undocumented artificial fill, discontinuous Quaternary-age colluvium (topsoil), Quaternary-age old paralic deposits (previously termed “terrace deposits on older geologic maps), underlain by Tertiary sedimentary bedrock belonging to the Torrey Sandstone Formation at depth. Transient beach deposits occur at the base of the bluff, also are 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 is shown on Plate 1.

Beach Deposits (Map Symbol - Qb)

A transient shingle beach composed of sand with cobbles at shallow depth, exists at the base of the bluff. The beach deposits will not be encountered in the vicinity of the proposed development.

Quaternary-age Colluvium (Mapped Symbol - Qcol)

Quaternary-age colluvium was encountered at the surface in Hand-Auger Borings B-1, B-2, and B-3. The colluvium consisted of medium to dark brown, silty sand. The colluvium was generally slightly moist to moist, and very loose, to dense, was cohesionless, and non-uniform. As observed in the aforementioned Hand-Auger Borings, the total thickness of the colluvium ranged between 1 and 2½ feet. Colluvium is considered unsuitable for the support of the proposed settlement-sensitive improvements and/or new planned fills in its existing state.

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

Quaternary-age old paralic deposits were encountered beneath the surficial colluvium all in Hand-Auger Borings advanced at the site, and in Boring B-1, from GSI (2000), as well as supplemental HSA Boring B-4. These deposits were also observed in the coastal bluff exposure above approximate elevation for 34 feet. As observed, a weathered zone was developed upon the old paralic deposits in the Hand-Auger Borings and deeper Boring B-1, on the order of 1½ to 2 feet thick, based on the available subsurface data. Where directly observed, the weathered old paralic deposits generally consisted of yellowish brown to light to medium brown, silty sandstone. Unweathered old paralic deposits were encountered in Boring B-1 and B-4. As is typical in the vicinity, reddish brown to reddish yellow, and yellowish brown very fine- to fine-grained sandstone comprise the weakly iron oxide cemented upper fluff. The unweathered old paralic deposits were generally damp to moist, and medium dense to dense. Weathered old paralic deposits are considered unsuitable for the support of settlement-sensitive improvements and/or new planned fill in their existing state. Unweathered old paralic deposits are considered suitable bearing materials.

Tertiary Torrey Sandstone Formation (Map Symbol - Tt)

Based on geologic mapping of the coastal bluff (Eisenberg and Abbott, 1985) and our review of Kennedy and Tan (2007, sedimentary bedrock belonging to the Tertiary Torrey Sandstone Formation unconformably underlies the old paralic deposits at approximate elevation of 34 feet. This formation is exposed in the lower portions of the coastal bluff (i.e., sea cliff). Based on our observations, the Torrey Sandstone consists of a broadly planar, cross-bedded, yellow and light gray to gray brown, slightly silty, fine to coarse grained sandstone, with localized concretions. The sandstone my be characterized as, interbedded layers of yellow and gray fine sand with nodules, laminations, and iron-oxide staining. Regionally, this formation is described as an arkosic, subangular, moderately well indurated sandstone (Kennedy, 1975). The Torrey Sandstone is believed to have been formed 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

Regionally, the old paralic deposits generally contain thick, nearly sub-horizontal beds. The Torrey Sandstone Formation is generally planar cross-bedded, however, was thickly bedded here. Regional dip is approximately 5 degrees to the northwest. Several northerly trending joints were also observed in the Torrey Sandstone Formation on the property to the south. Joints were not observed onsite, however, are generally parallel or slightly oblique to the trend of the coastal bluff. Some notching was observed near the bluff toe.

FAULTING AND REGIONAL SEISMICITY

Local and Regional Faults

Our review and field observations indicates that there are no known active faults crossing this 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 active and potentially active faults. 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 3.1 miles (5.0 kilometers). Portions of the Rose Canyon fault have demonstrated movement in the Holocene Epoch (i.e., last 11,000 years); and therefore, are considered active and located in an Alquist-Priolo Earthquake Fault Zone (California Geological Survey [CGS], 2018). Cao, et al. (2003) indicate that Rose Canyon fault is an “B” fault with a slip rate of 1.5 (±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 active faults crossing the site that have exhibited activity in the last 11,700 years (Jennings and Bryant, 2010; CGS, 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 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 2018 was 0.56 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, 2019a), 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 (2019), was utilized to aid with design. The short spectral response utilizes a period of 0.2 seconds.

2019 CBC SEISMIC DESIGN PARAMETERS

PARAMETER	VALUE	VALUE per ASCE 7-16	2019 CBC or REFERENCE
Risk Category	II	-	Table 1604.5
Site Class	D	-	Section 1613.2.2/Chap. 20 ASCE 7-16 (p. 203-204)

2019 CBC SEISMIC DESIGN PARAMETERS			
PARAMETER	**VALUE**	**VALUE per ASCE 7-16**	**2019 CBC or REFERENCE**
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)	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)
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. $S_{D1} \geq 0.2 => 0.8 \geq 0.2$, per Section 11.6 site is in Risk Category D

GENERAL SEISMIC PARAMETERS	
PARAMETER	**VALUE**
Distance to Seismic Source (Rose Canyon fault)	3.1 mi (5.0 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, 2019a) and regular maintenance and repair following locally significant seismic events (i.e., M_w5.5) will likely be necessary.

GROUNDWATER

GSI did not encounter groundwater nor evidence of perched water in our hand-auger borings, deeper boring, nor in the boring on the adjoining property, to the explored depths, nor is it exiting the bluff face. Although moisture content was elevated at the contact between the bedrock and the paralic deposits, perched groundwater was not noted at this location. In addition, a review of oblique aerial photographs (Appendix A), did not indicate groundwater perched on the top of the Torrey Sandstone exposed in the bluff, as far back as 1972; however, indicated an intermittent hint of nearby vegetation (seeking fresh water), at the bluff toe. Accordingly, this groundwater condition was modeled in our slope stability analysis.

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.

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 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 adjoining un-mitigated properties may influence improvements at the perimeter of the site. Special setbacks and/or foundations may be utilized if significant structures/improvements are placed close to the perimeter of the site. 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 and densification is relatively low owing to the dense, nature of the old paralic deposits and the underlying Torrey Sandstone Formation. In addition, the recommendations for remedial earthwork and foundations would further reduce any significant liquefaction/densification potential. Some seismic densification of the adjoining un-mitigated site(s) may adversely influence planned improvements at the perimeter of the site. However, given the remedial earthwork and foundation recommendations provided herein, the potential for the planned building to be affected by significant seismic densification, or liquefaction of offsite soils, may be considered low.

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 sea cliff, have typically experienced past toppling failure events due to the presence of nearly bluff-parallel joints and notching along the bluff toe. GSI believes that the overlying old paralic deposits in the middle and upper portions of the coastal bluff are most susceptible to 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

Tsunamis 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. However, 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, colluvium, and Quaternary old paralic deposits with standard excavation equipment 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.

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 as well as those found in the GSI (2000) boring, advanced on the adjacent southerly property. Our slope stability analysis also included the shear strength parameters (saturated unit weights, cohesion, friction angle) assigned to the old paralic deposits and Torrey Sandstone routinely utilized in similar studies in the immediate site vicinity. 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). This program calculates the factor of safety for specified circles or searches for the circular, block, or irregular slip surface having the minimum factor of safety using the Modified Bishop, Simplified Janbu, and General Limit Equilibrium (GLE) methods (Spencer or Morgenstern-Price Methods). Our analyses incorporated the limit-equilibrium approach as modeled in the Modified Bishop and GLE (Spencer's) method. Additional information regrading 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') was prepared for the analysis. Geologic Cross Section X-X' incorporated the anticipated location for the proposed residence. The geologic cross-section is provided as Plate 2. The location of Geologic Cross Section X-X' is shown on Plate 1. Geologic Cross Section X-X' was selected for the analysis as it represents the most critical slope, owing to the steepness of the bluff along this line of section.

We modeled the regional water table at near the maximum sea level at high tide. GSI also applied uniform loads to model building foundations and the interior slab-on-grade loads in our analysis. 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 a failure through the old paralic deposits and gross bluff failure (see Plates E-1 and E-2 in Appendix E). The criteria for bluff setback in Encinitas is the greater of 40 feet, or FOS ≥ 1.5, or 75 year retreat rate. Thus, the proposed residential structure setback would be governed by the 40 foot prescriptive bluff setback.

Lately, as part of the review process for a coastal development permit, the City has been admonishing that "*The applicant should be made aware that the Coastal Commission Staff may require a setback that is a cumulative setback of the combination of the Factor of Safety setback and the erosion rate setback as part of their review of the project.*" This would ostensibly maintain a cumulative FOS ≥ 1.5 for the 75-year design life of the project, thus providing confidence that bluff stabilization would not be necessary for the property. However, this assumption is unreasonably conservative. Typically bluff stabilization is allowed when the FOS ≤ 1.2 (CCC, 2014) intercepts the foundation of the primary structure. To that end, the FOS ≥ 1.2 (see Plate E-3), and the cumulative FOS ≥ 1.2 + the 75-year erosion rate setback is also shown on Plate 1, assuring that bluff stabilization would not be necessary for the property during the design life.

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.

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 GI (2005) shear strength tests are also presented in Appendix D.

Classification

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

Expansion Potential

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

SAMPLE LOCATION AND DEPTH (FT)	SOIL TYPE	EXPANSION INDEX	EXPANSION POTENTIAL*
B-2 @ 0-4	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 Borings B-2, and B-4 were evaluated in general accordance with ASTM D 422. Based on this testing, these earth materials are 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 performed as well as previously performed on several undisturbed samples, from the site and lot to the south, in general accordance with ASTM test method D-3080. Test results are presented on the following table.

SAMPLE LOCATION AND DEPTH (FT)	PRIMARY		RESIDUAL	
	COHESION (PSF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)	FRICTION ANGLE (DEGREES)
2020 B-4 @7½ (Qop)	74	38.7	72	29.7
2020 B-4 @ 15 (Qop)	90	40.5	74	32.2
2020B-4 @ 25 (Qop)	229	41.0	27	33.0
2020 B-4 @ 32½ (Qop)	175	39.3	64	32.0
2020 B-4 @ 45-feet (Tt)	306	35.1	121	35.1
B-1 @ 5-6' (Qop)	-	-	190	33
B-1 @ 40-41' (Qop)	-	-	496	33
Base of Bluff (Tt)	-	-	1,000	37
TP-103 @ 1¼	250	27	150	27

Soil pH, Saturated Resistivity, Soluble Sulfates and Soluble Chlorides

GSI conducted testing on a sample of the site earth materials, collected from Boring B-2, for an evaluation of general soil corrosivity and soluble sulfates, and soluble chlorides. Testing was performed in general accordance with California Test Methods

(CTMs) 417, 422, and 532 (643). Test results are presented in Appendix D and the following table:

SAMPLE LOCATION AND DEPTH (FT)	pH	SATURATED RESISTIVITY (ohm-cm)	SOLUBLE SULFATES (% by weight)	SOLUBLE CHLORIDES (ppm)
B-2 @ 0-4	6.69	1,500	0.019	16

Corrosion Summary

Laboratory testing indicates that tested sample of the onsite soils is neutral with respect to soil acidity/alkalinity; is corrosive to exposed, buried metals when saturated; presents a very low sulfate exposure to concrete (i.e., Exposure Class S0 per American Concrete Institute 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, plumbing/mechanical engineer(s), and/or structural engineer, minimally considering Exposure Classes S0, W1, and C1, per ACI 318-14. Please note that the site is located in close proximity to the Pacific Ocean (i.e., ocean spray/fog), considered a corrosive environment. Thus, corrosive effects from this condition should be considered in the project design and construction.

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.
- 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. Slope stability analysis indicate that the proposed residential structure with a prescriptive bluff setback of 40 feet, will have code-compliant factors-of-safety against upper bluff and gross bluff failures. In addition, the aforementioned setback distance should provide sufficient protection from coastal bluff retreat over the 75-year design life of the proposed residential structure.

4. All undocumented fill, colluvium, 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., residential structure, 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.

   Based on the available subsurface data, potentially compressible earth materials, within the areas of proposed development will likely extend to depths of approximately 2½ 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 serviceable use both during and following construction.

5. Laboratory testing indicates that the expansion index of 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, 2019a). 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 sulfate, and chloride testing, GSI concludes that the onsite soils are neutral with respect to soil acidity/alkalinity; are corrosive to exposed, buried metals when saturated; possesses negligible sulfate exposure to concrete (Exposure Class S0 per ACI 318-14); and have 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, minimally considering Exposure Classes S0, W1, and C1, per ACI 318-14. Owing to the site's proximity to the Pacific Ocean, the effects of sea spray/fog 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 existing artificial fill and colluvium should be constructed in accordance with Cal/OSHA guidelines for

Type “C” soils (i.e., 1½:1 [h:v] gradient), provided groundwater or running sands are not 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.

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 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, 2019a), 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.

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 fills, colluvium, and weathered old paralic deposits, these materials should be removed to expose unweathered old paralic deposits and then be reused as engineered fill in areas proposed for settlement-sensitive improvements or areas to receive planned fills. Based on the available subsurface data, potentially compressible earth materials, within the areas of proposed development will likely extend to depths of approximately 2½ feet to 4 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 serviceable use both during and following construction.

Fill Placement

1. Subsequent to ground preparation, fill materials should be 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).

2. Should the removal of potentially compressible soils, within the building footprint, result in the need for fill materials to achieve pad grade, the fill materials should be 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).

3. Fill materials should be cleansed of major vegetation and debris prior to placement.

4. Any import materials should be observed and determined suitable by the soils engineer prior to placement on the site. Import material (if any) for a fill cap should be very low expansive (E.I. less than 20 with a plasticity index less than 15). Foundation designs may be altered if import materials have a greater expansion value than the onsite materials encountered in this investigation.

Earth Material Transitions in Building Pad Area

Should an earth material transition condition be encountered in the building pad area, fill materials placed therein, should be moisture conditioned as previously recommended, laid down 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), following the removal of potentially compressible soils. Additional compactive effort may be necessary by the earthwork contractor to obtain the recommended relative compaction. The intent of the above recommendations is to provide uniform support of the slab-on-grade floor. Foundations for the residence should extend through any fill or colluvium and be founded into the underlying old paralic deposits.

Temporary Slopes

On a preliminary basis, temporary excavations greater than 4 feet, but less than 20 feet in overall height, completed into existing fills and colluvium, 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. Temporary excavations greater than 4 feet, but less than 20 feet in overall height, completed into old paralic deposits (weathered and unweathered), 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.

SHORING DESIGN

Shoring of Excavations

The proposed project would typically include planned and remedial excavations in close proximity to adjacent property, improvements, and the coastal bluff setback zone. Thus, shoring of excavations will likely be necessary in some areas of the site. We recommend that excavations be retained either by a cantilever or braced shoring system deriving passive support from cast-in-place soldier piers and sheeting or wood 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 caving conditions and the proximity of neighboring property. 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.

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 stockpiles should not be stored or operated 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.

Lateral Pressure

The active and passive pressures to be utilized for temporary and permanent shoring designs are provided in Figures 7 and 8, respectively. 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.

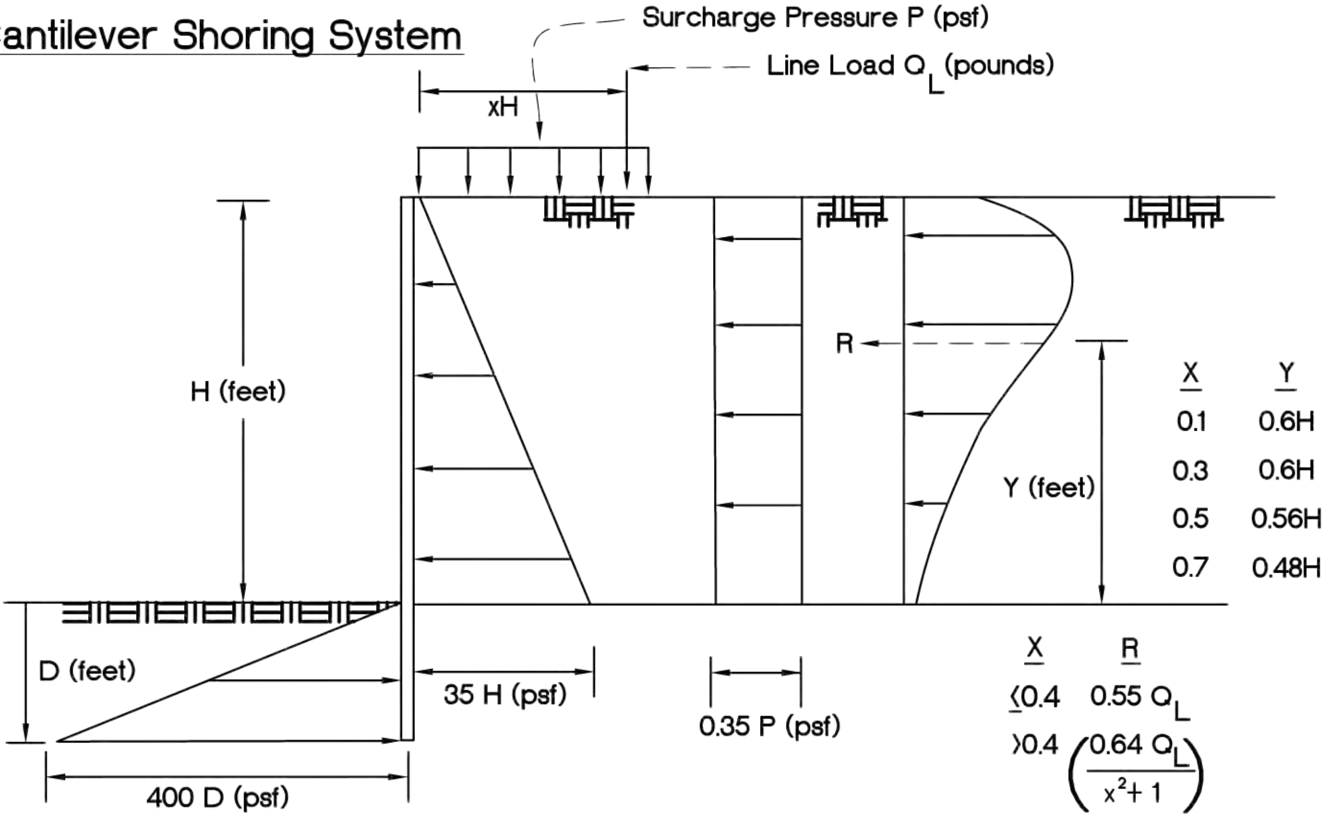
The recommendations for shoring design assume excavation walls up to about 15 feet high. A braced system may allow for deeper excavation. However, this would be at the sole discretion of the shoring designer. An empirical equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. This does not include other superimposed loading conditions such as traffic, structures, seismic events, or adverse geologic conditions. For permanent shoring walls greater than 4 feet in height, a seismic increment of 15H (uniform pressure [psf/ft]) may be considered 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 surcharges (if any) should be minimally applied as 100 psf per lineal foot in the upper 5 feet of wall(s) if traffic is within “H” of the back of walls, where “H” equals the height of the retained soils.

Construction Monitoring/Excavation Observation (All Excavations)

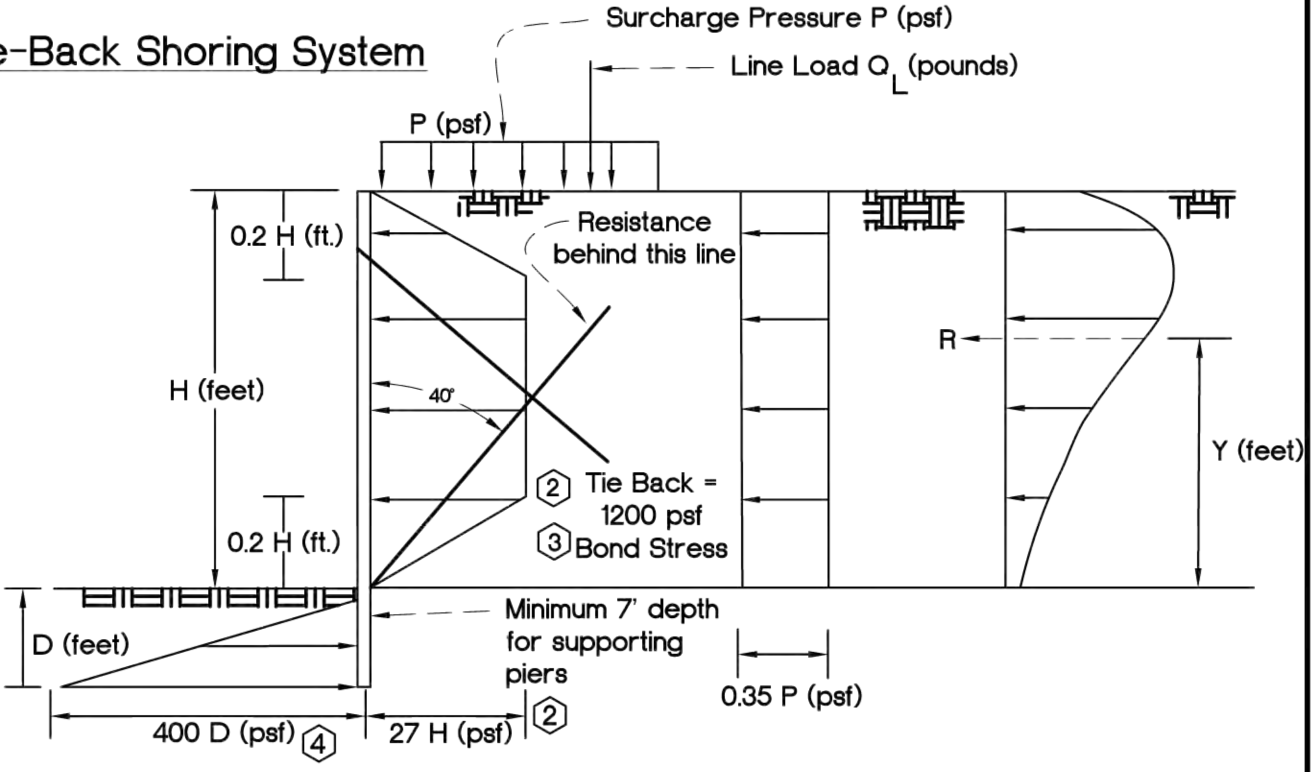
When excavations are made adjacent to an existing structure (i.e., utility, road or building) there is a risk of some damage to that structure even if a well designed system of excavation and/or shoring, is planned and installed. We recommend, therefore, that a systematic program of observations be made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

Cantilever Shoring System



Tie-Back Shoring System



NOTES

① Include groundwater effects below groundwater level.

② Include water effects below groundwater level.

③ Grouted length greater than 7 feet; field test anchor strength.

④ Neglect passive pressure below base of excavation to a depth of one pier diameter.

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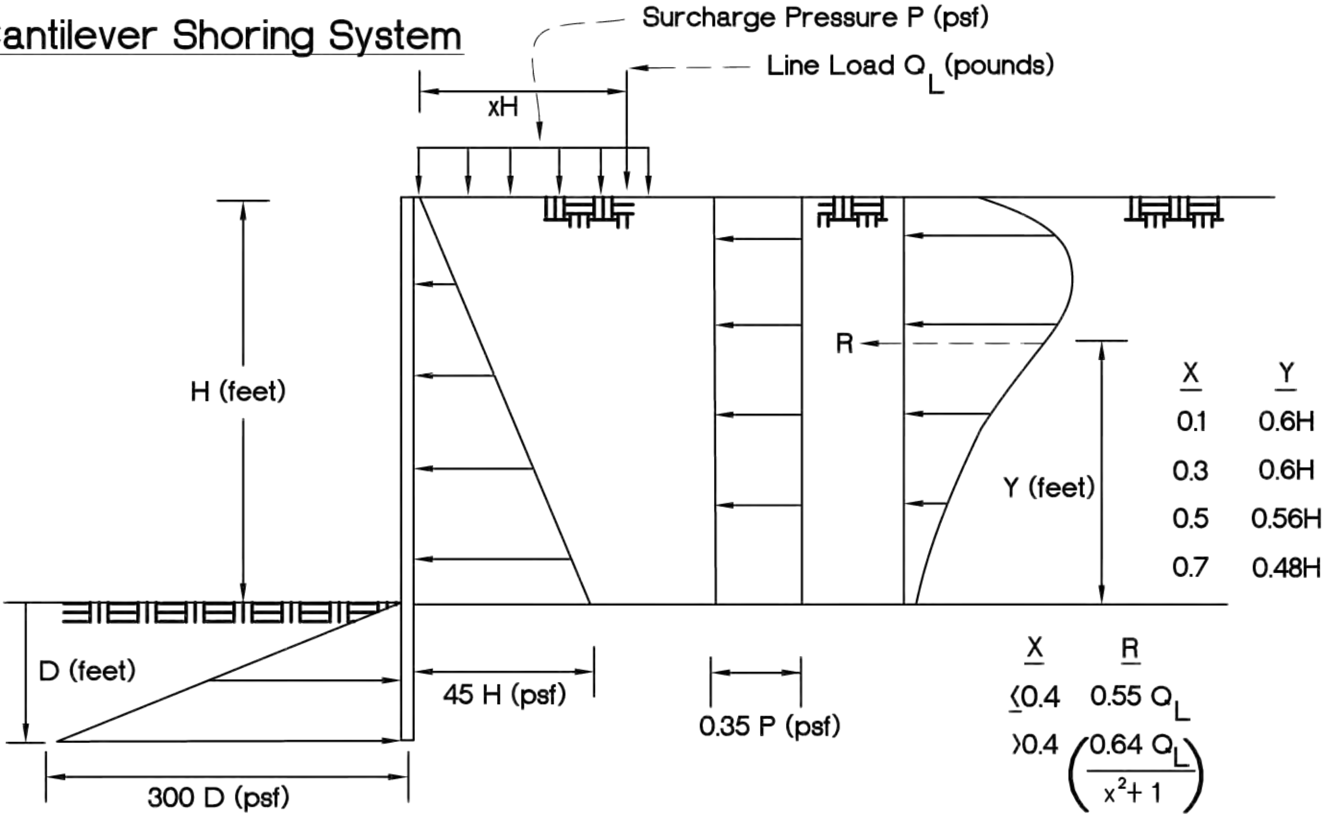
ORANGE CO.
SAN DIEGO CO.

LATERAL EARTH PRESSURES FOR TEMPORARY SHORING SYSTEMS

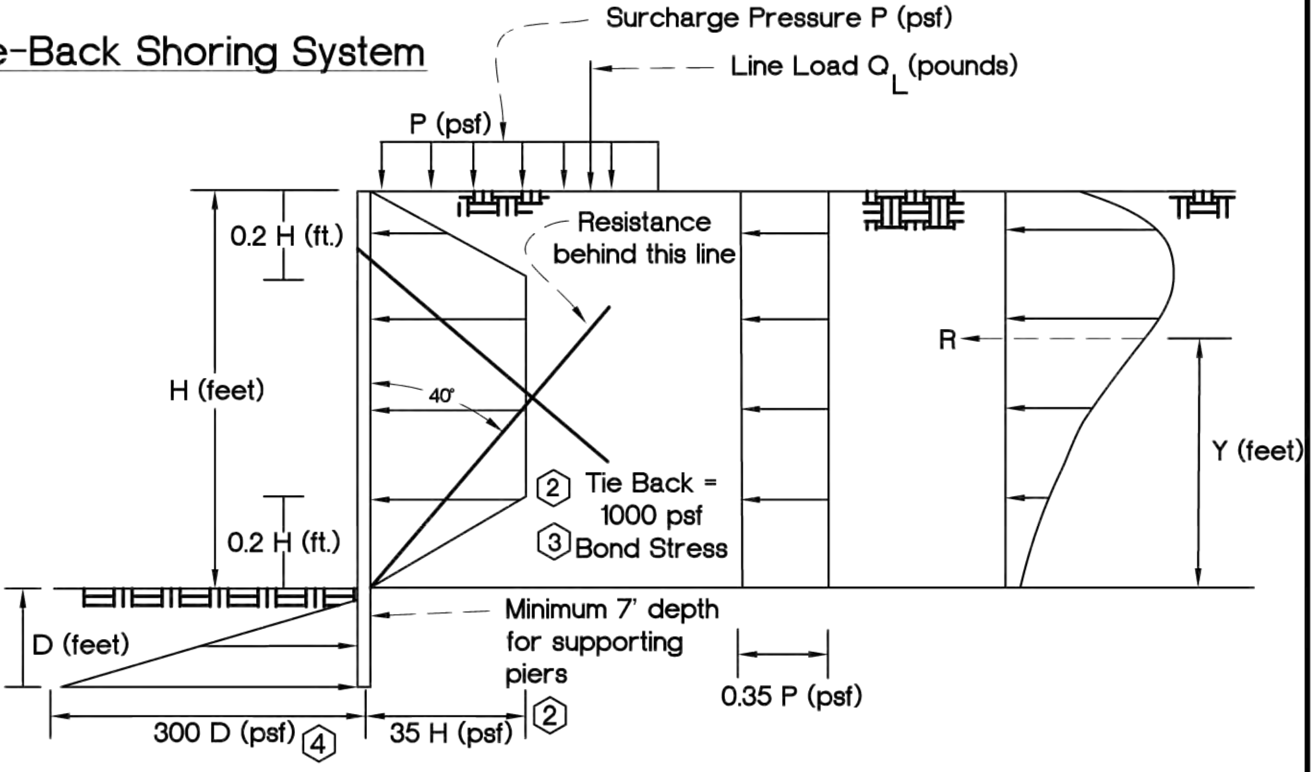
Figure 7

W.O. 7557-A-SC	DATE 04/20	SCALE None

Cantilever Shoring System



Tie-Back Shoring System



NOTES

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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 PERMANENT SHORING SYSTEMS

Figure 8

W.O. 7557-A-SC	DATE 04/20	SCALE None

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.

PRELIMINARY RECOMMENDATIONS - FOUNDATIONS

General

The following recommendations for foundation design and construction are considered preliminary and are based on assumed loading conditions and current expansion index (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 proposed residential structure will be underlain by at least 7 feet of very low expansive soils (E.I. of 20 or less and Plasticity Index [P.I.] of 14 or less).

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

1. The foundation system for the proposed residence 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 unweathered old paralic deposits or compacted fill (but not both simultaneously). This value may be increased by 20 percent for each additional 12 inches in depth 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.

3. Passive earth pressure 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.

Foundation Settlement

Shallow spread footings founded into unweathered old paralic deposits per the recommendations contained herein should be minimally designed to accommodate a total settlement of 1½ inches and a differential settlement of at least ¾-inch in a 40-foot span (angular distortion = 1/640).

Footing Setbacks

Footings for the proposed residential structure should maintain a minimum 40-foot horizontal setback from the bluff edge. Footings adjacent to any other site slope 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 non-detrimentally expansive (i.e., E.I. of 20 or less and P.I. of 14 or less). Recommendations for foundations underlain by at least 7 feet of non-detrimentally expansive soil conditions are presented herein.

Recommendations by the project's design-structural engineer or architect, which may exceed the soils 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 or compacted fill (but not both simultaneously). 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. 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 main 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. Concrete slab-on-grade floors 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 percent minimum relative compaction. This material 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 the concrete floor slab, 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 the foundation or slabs. 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 vapor transmission will occur in new slab-on-grade floors as a result of chemical reactions taking place within the curing concrete. 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:

- Concrete slabs, including garages, should be thicker than 5 inches.

- 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.).

- 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 project includes the construction of basement retaining walls. It is possible that the project may also include site retaining walls. Recommendations for the design and construction of conventional 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 soils (typically Class 2 permeable filter material or Class 3 aggregate base) or native onsite materials with an expansion index up to 20 are used to backfill any retaining wall. The type of backfill (i.e., select or native), should be specified by the wall designer, and clearly shown on the 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]).

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. 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 unweathered old paralic deposits.

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 120 pcf and 125 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 15 feet high. Design parameters for walls less than 3 feet in height may be superceded by regional standard design. 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 in the upper 5 feet of backfill for light truck and cars traffic. 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 residence, 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 in the event of failure, GSI recommends that the walls be evaluated for seismic surcharge in general accordance with 2019 CBC requirements. The retaining walls in this category should maintain an overturning Factor-of-Safety (FOS) of approximately 1.25 when the seismic surcharge (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° - $\phi/2$ 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

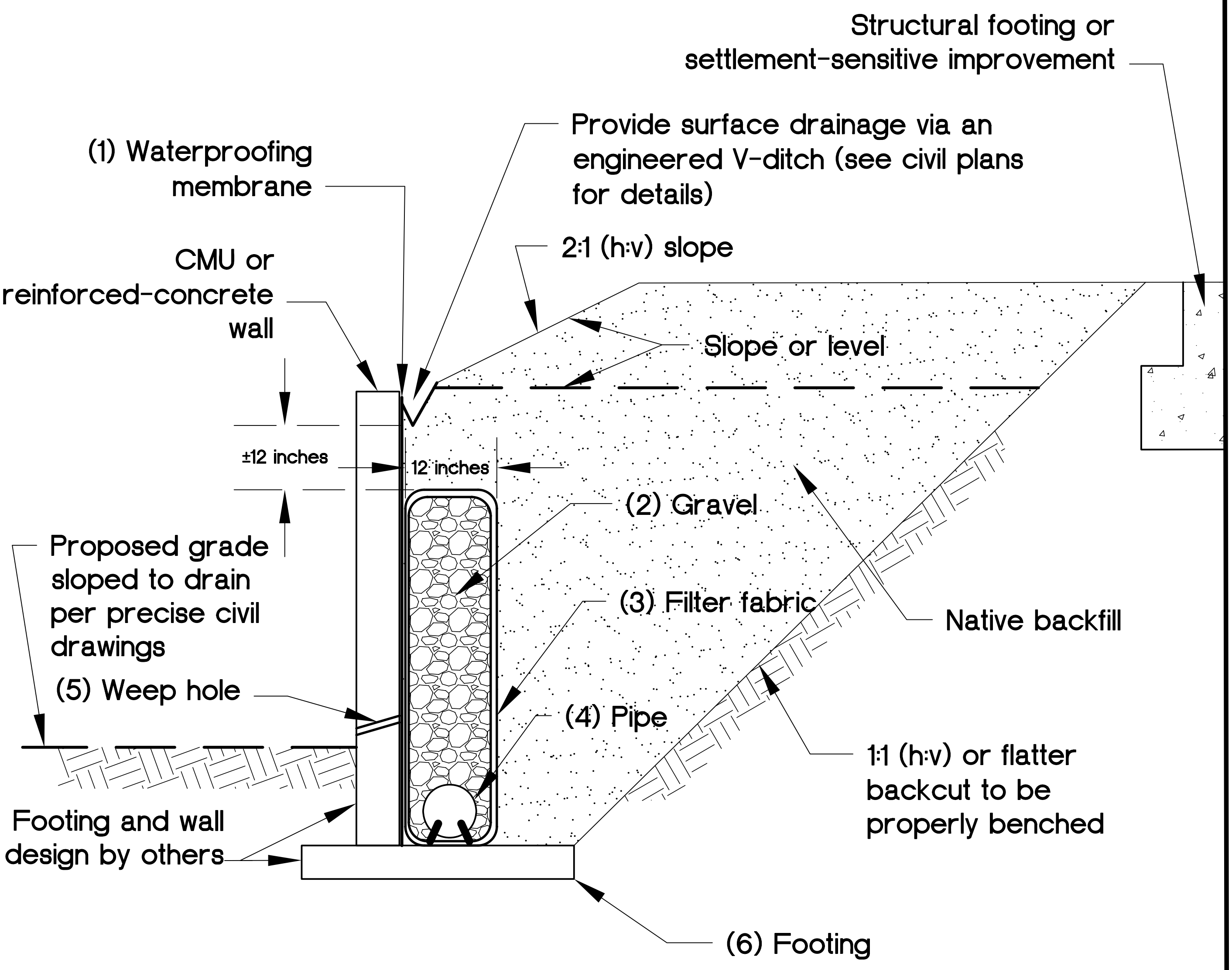
a_h = Probabilistic horizontal site acceleration with a percentage of "g"

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 must be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. A backdrain system is considered necessary 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



(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.

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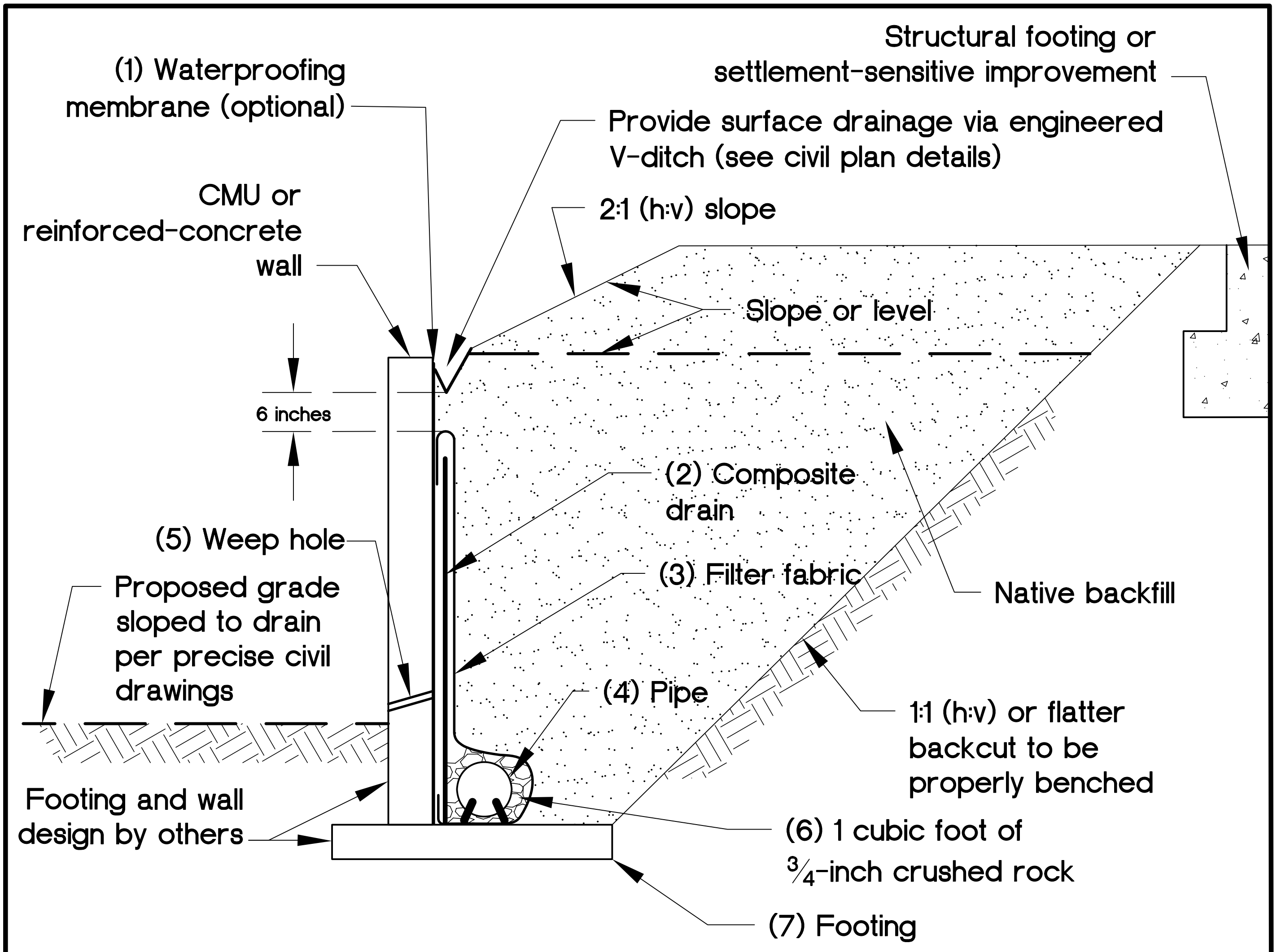
RETAINING WALL DETAIL — ALTERNATIVE A

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.

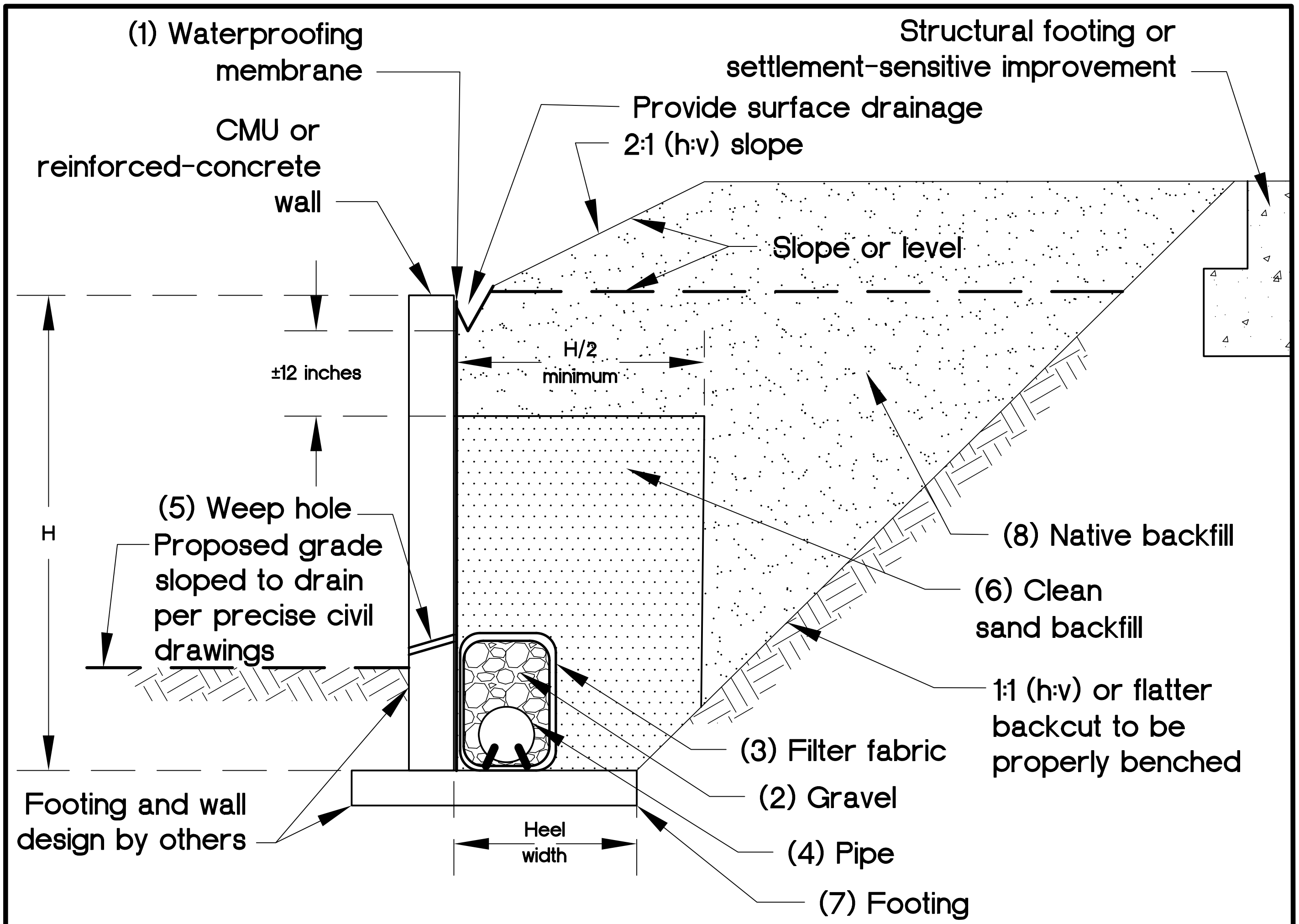
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(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, $^3/_4$ to $1^1/_2$ inch.

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

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

(5) Weep hole: 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. $\geq$35 then all sand requirements also may not be required and will be reviewed by the geotechnical consultant.

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

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 1.1 to 1.2 times the soil's optimum moisture content, placed in relatively thin lifts, and compacted to at least 90 percent of the laboratory standard (ASTM D 1557).

Outlets should consist of a 4-inch diameter solid PVC or ABS pipe spaced no greater than ±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. ≤ 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

Site 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, the civil 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.

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 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.

Passive Resistance: 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.

Allowable Axial Capacity:

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 should 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), 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 lots should provide a minimum of 1 to 2 percent fall to the street, 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 paralic deposits will occur near the surface at the conclusion of grading.

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 slope stability concerns.

- Infiltration of storm water 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.

- 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. 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 each 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.

4. 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 (i.e., sand, pea-gravel, etc.), or vapor barriers (i.e., 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

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.

File:Mgr: c;\SoilClassif.wpd

PLATE B-1

GeoSoils, Inc.

BORING LOG

PROJECT: NEWMAN
216 Neptune, Encinitas

W.O. 7557-A4-SC *BORING* B-1 *SHEET* 1 *OF* 1

DATE EXCAVATED 1-3-19 *LOGGED BY:* MJS *APPROX. ELEV.:* 69' MSL

SAMPLE METHOD: Hand Auger

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
	Bulk	Undisturbed	Blows/Ft.					
0	■			SM				**COLLUVIUM:** @ 0' SILTY SAND, medium to dark brown, slightly moist, loose; cohesionless.
5	■			SM				**QUATERNARY OLDER PARALIC DEPOSITS:** @ 2½' SILTY SAND, medium brown, dry to slightly moist, medium dense; @ 4' SILTY SAND, yellowish brown to light brown, dry to slightly moist, medium dense.

Total Depth = 6'
No Groundwater/Caving Encountered
Backfilled 1-3-2019

10
15
20
25
30

Standard Penetration Test
Undisturbed, Ring Sample
Groundwater
Seepage

GeoSoils, Inc.

BORING LOG

PROJECT: NEWMAN
216 Neptune, Encinitas

W.O. 7557-A4-SC *BORING* B-2 *SHEET* 1 *OF* 1

DATE EXCAVATED 1-3-19 *LOGGED BY:* MJS *APPROX. ELEV.:* 77' MSL

SAMPLE METHOD: Hand Auger

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0				SM				**COLLUVIUM:** @ 0' SITLY SAND, dark brown, moist, very loose to very dense.
	■			SM				**QUATERNARY OLDER PARALIC DEPOSITS:** @ 1' SILTY SAND, yellowish brown to light brown, dry to slightly moist, loose to medium dense.

Total Depth = 6'
No Groundwater/Caving Encountered
Backfilled 1-3-2019

◪ *Standard Penetration Test*
⌶ *Undisturbed, Ring Sample*
▼ *Groundwater*
⚲ *Seepage*

GeoSoils, Inc.

BORING LOG

PROJECT: NEWMAN
216 Neptune, Encinitas

W.O. 7557-A4-SC *BORING* B-3 *SHEET* 1 *OF* 1

DATE EXCAVATED 1-3-19 *LOGGED BY:* MJS *APPROX. ELEV.:* 76' MSL

SAMPLE METHOD: Hand Auger

Depth (ft.)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/Ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
0	■			SM				**COLLUVIUM:** @ 0' SILTY SAND, dark brown, moist, very loose to loose.
	■			SM				**QUATERNARY OLDER PARALIC DEPOSITS:** @ 2½' SILTY SAND, yellowish brown, dry to moist, medium dense.
5								
								Total Depth = 6' No Groundwater/Caving Encountered Backfilled 1-3-2019
10								
15								
20								
25								
30								

◪ *Standard Penetration Test*
⌶ *Undisturbed, Ring Sample*
▼ *Groundwater*
♀ *Seepage*

GeoSoils, Inc.

PLATE B-4

GeoSoils, Inc.

BORING LOG

PROJECT: NEWMAN
216 Neptune, Encinitas

W.O. 7557-A4-SC *BORING* B-4 *SHEET* 1 *OF* 2

DATE EXCAVATED 3-9-20 *LOGGED BY:* TMP *APPROX. ELEV.:* 78' MSL

SAMPLE METHOD: 6" Hollow Stem Auger, 140 lb. Hammer, 30-inch Drop

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Material Description
	Bulk	Undisturbed	Blows/Ft.					
0	■			SP				**QUATERNARY OLD PARALIC DEPOSITS (Qop 6- 7):** @ 0' SAND, red to light brown, slightly moist, medium dense; fine to medium grained, cohesionless.
			20		108.9	5.1	26.2	@ 2.5' As per 0', damp.
5		⌶	16		105.0	5.4	24.6	@ 5' SAND, light brown to red brown, damp, medium dense.
		⌶	22		104.9	5.3	24.4	@ 7.5' As per 5'.
10		⌶	30		108.8	8.2	41.7	@ 10' As per 5', silty, moist.
		⌶	36		111.5	7.0	38.4	@ 12.5' As per 10', dense.
15		⌶	38	SM	104.7	5.5	25.3	@ 15' SILTY SAND, dark yellow brown, damp, dense.
		⌶	45	SP	105.8	5.4	25.3	@ 17.5' SAND, medium yellow brown, damp, dense; fine to coarse grained, partially cemented.
20		⌶	50		105.5	4.6	21.6	@ 20' As per 17.5'.
		⌶	44		107.3	4.1	19.8	@ 22.5' As per 20'.
25		⌶	44		108.3	4.1	20.7	@ 25' As per 22.5', fine to medium grained.
		⌶	72		105.6	3.5	16.3	@ 27.5' As per 25', fine to coarse grained.
30		⌶	63		106.8	3.0	14.4	@ 30' As per 27.5'.
		⌶	57		106.1	3.4	16.2	@ 32.5' As per 27.5'.

◩ *Standard Penetration Test*
⌶ *Undisturbed, Ring Sample*
▽ *Groundwater*
♀ *Seepage*

GeoSoils, Inc.

BORING LOG

PROJECT: NEWMAN
216 Neptune, Encinitas

W.O. 7557-A4-SC *BORING* B-4 *SHEET* 2 *OF* 2

DATE EXCAVATED 3-9-20 *LOGGED BY:* TMP *APPROX. ELEV.:* 78' MSL

SAMPLE METHOD: 6" Hollow Stem Auger, 140 lb. Hammer, 30-inch Drop

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	**Material Description**
	Bulk	Undisturbed	Blows/Ft.					
35		⌶	57		109.5	3.7	19.1	@ 35' As per 27.5'.
		⌶	44	SM	111.3	10.9	59.5	@ 37.5' SILTY SAND, dark reddish brown, moist, dense; some clay partially cemented, basal section
40		⌶	44		113.9	11.5	67.3	@ 40' As per 37.5', fine to coarse grained, occasional pebble to 2 mm.
		⌶	50-5½"		95.5	9.0	32.6	**TERTIARY TORREY SANDSTONE FORMATION (Tt):** @ 43' SANDSTONE, light olive green/dark reddish yellow, slightly moist, very dense; occasional iron oxidation staining, weathered.
45			50-4"		91.4	10.1	32.9	@ 45' As per 43', partially cemented.
								Total Depth = 46' No Caving or Groundwater Encountered Backfilled with bentonite, as per DEH guidelines, 3-9-20
50								
55								
60								
65								

◪ *Standard Penetration Test*
⌶ *Undisturbed, Ring Sample*
▽ *Groundwater*
♀ *Seepage*

GeoSoils, Inc.

BORING LOG

W.O. 2915-A-SC

PROJECT: JAY REFOLD
210 Neptune Avenue

BORING B-1 *SHEET* 1 *OF* 2

DATE EXCAVATED 7-27-00

SAMPLE METHOD: 140 lb Hammer with 30" drop

▦ *Standard Penetration Test*

▨ *Undisturbed, Ring Sample*

≋ *Water Seepage into hole*

Depth (ft.)	Sample		Blows/ft.	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Saturation (%)	Description of Material
	Bulk	Undis-turbed						
0	■			SM				**TERRACE DEPOSITS** @ 0', SILTY SAND, light to medium red brown, dry to damp, loose; cohesionless.
5		▨	13		111.3	3.1		@ 5', SILTY SAND, medium reddish brown, damp, medium dense; cohesionless.
10		▦	9			3.8		@ 10', SILTY SAND, medium reddish brown, damp, loose; cohesionless.
15								
20		▨	29		101.9	6.2		@ 20', SILTY SAND, medium brownish red, damp, medium dense; cohesionless.
25								

BORING LOG

GeoSoils, Inc.

W.O. 2915-A-SC

PROJECT: JAY REFOLD
210 Neptune Avenue

BORING B-1 *SHEET* 2 *OF* 2

DATE EXCAVATED 7-27-00

SAMPLE METHOD: 140 lb Hammer with 30" drop

▩ *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
		▩	60	SM		5.5		@ 30', SILTY SAND, medium reddish brown, damp, very dense; cohesionless.
35								
40		▨	55	SM	119.2	10.5		**TORREY SANDSTONE** @ 40', SILTY SANDSTONE, light grayish brown, moist, dense.
45								
		▩	50/3"			6.6		@ 46', SILTY SANDSTONE, light grayish brown, moist, very dense.
								Total Depth = 47 1/2' No groundwater encountered Backfilled 7-27-00
50								
55								

APPENDIX C

SEISMICITY DATA

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

                      DETERMINISTIC ESTIMATION OF
                PEAK ACCELERATION FROM DIGITIZED FAULTS


JOB NUMBER: 7557
                                                     DATE: 01-08-2019

JOB NAME: Newman

CALCULATION NAME: Test Run Analysis

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


SITE COORDINATES:
   SITE LATITUDE:  33.0528
   SITE LONGITUDE:  117.2998

SEARCH RADIUS:   62.4  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:   1
   Basement Depth:  .10 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
```

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                                        ---------------
                                        EQFAULT SUMMARY
                                        ---------------



                                 -----------------------------
                                 DETERMINISTIC SITE PARAMETERS
                                 -----------------------------

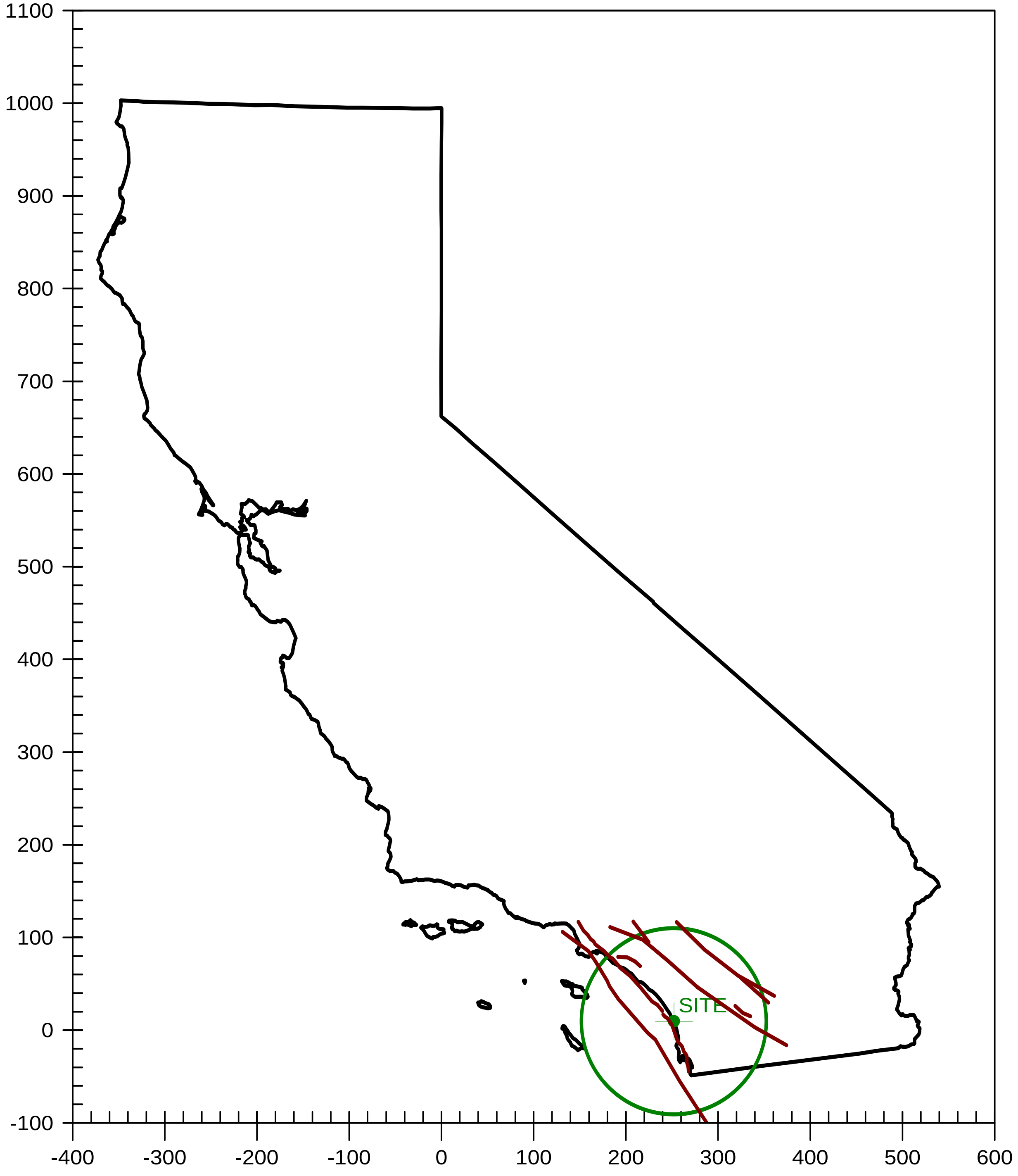
Page  1
-------------------------------------------------------------------------------
                                |              |ESTIMATED MAX. EARTHQUAKE EVENT
                                | APPROXIMATE  |-------------------------------
          ABBREVIATED           |   DISTANCE   | MAXIMUM  |   PEAK   |EST. SITE
          FAULT  NAME           |   mi   (km)  |EARTHQUAKE|   SITE   |INTENSITY
                                |              | MAG.(Mw) | ACCEL. g |MOD.MERC.
================================|==============|==========|==========|=========
ROSE CANYON                     |   3.1(   5.0)|   7.2    |   0.741  |   XI
NEWPORT-INGLEWOOD (Offshore)    |  10.5(  16.9)|   7.1    |   0.374  |   IX
CORONADO BANK                   |  17.6(  28.4)|   7.6    |   0.321  |   IX
ELSINORE (JULIAN)               |  28.0(  45.0)|   7.1    |   0.147  |  VIII
ELSINORE (TEMECULA)             |  28.0(  45.0)|   6.8    |   0.120  |   VII
PALOS VERDES                    |  40.4(  65.0)|   7.3    |   0.116  |   VII
ELSINORE (GLEN IVY)             |  40.9(  65.8)|   6.8    |   0.081  |   VII
SAN JOAQUIN HILLS               |  42.1(  67.8)|   6.6    |   0.097  |   VII
EARTHQUAKE VALLEY               |  42.6(  68.5)|   6.5    |   0.063  |   VI
SAN JACINTO-ANZA                |  50.8(  81.7)|   7.2    |   0.085  |   VII
SAN JACINTO-SAN JACINTO VALLEY  |  52.4(  84.4)|   6.9    |   0.067  |   VI
NEWPORT-INGLEWOOD (L.A.Basin)   |  52.7(  84.8)|   7.1    |   0.076  |   VII
SAN JACINTO-COYOTE CREEK        |  53.7(  86.5)|   6.6    |   0.053  |   VI
ELSINORE (COYOTE MOUNTAIN)      |  54.7(  88.0)|   6.8    |   0.060  |   VI
CHINO-CENTRAL AVE. (Elsinore)   |  55.2(  88.8)|   6.7    |   0.078  |   VII
WHITTIER                        |  59.1(  95.1)|   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 3.1 MILES (5.0 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.7410 g
```

CALIFORNIA FAULT MAP

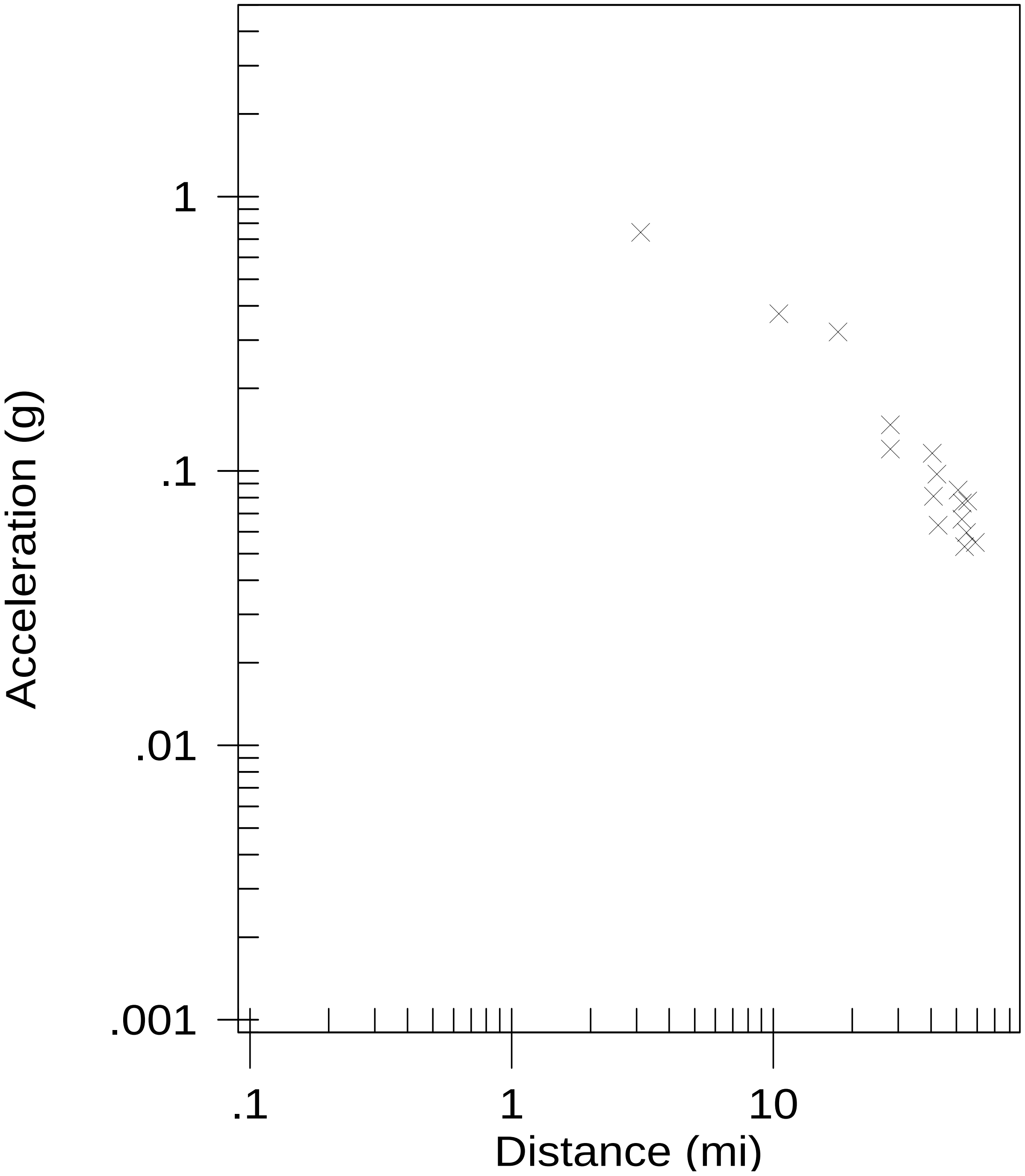
Newman



W.O. 7557-A-SC
PLATE C-3

MAXIMUM EARTHQUAKES

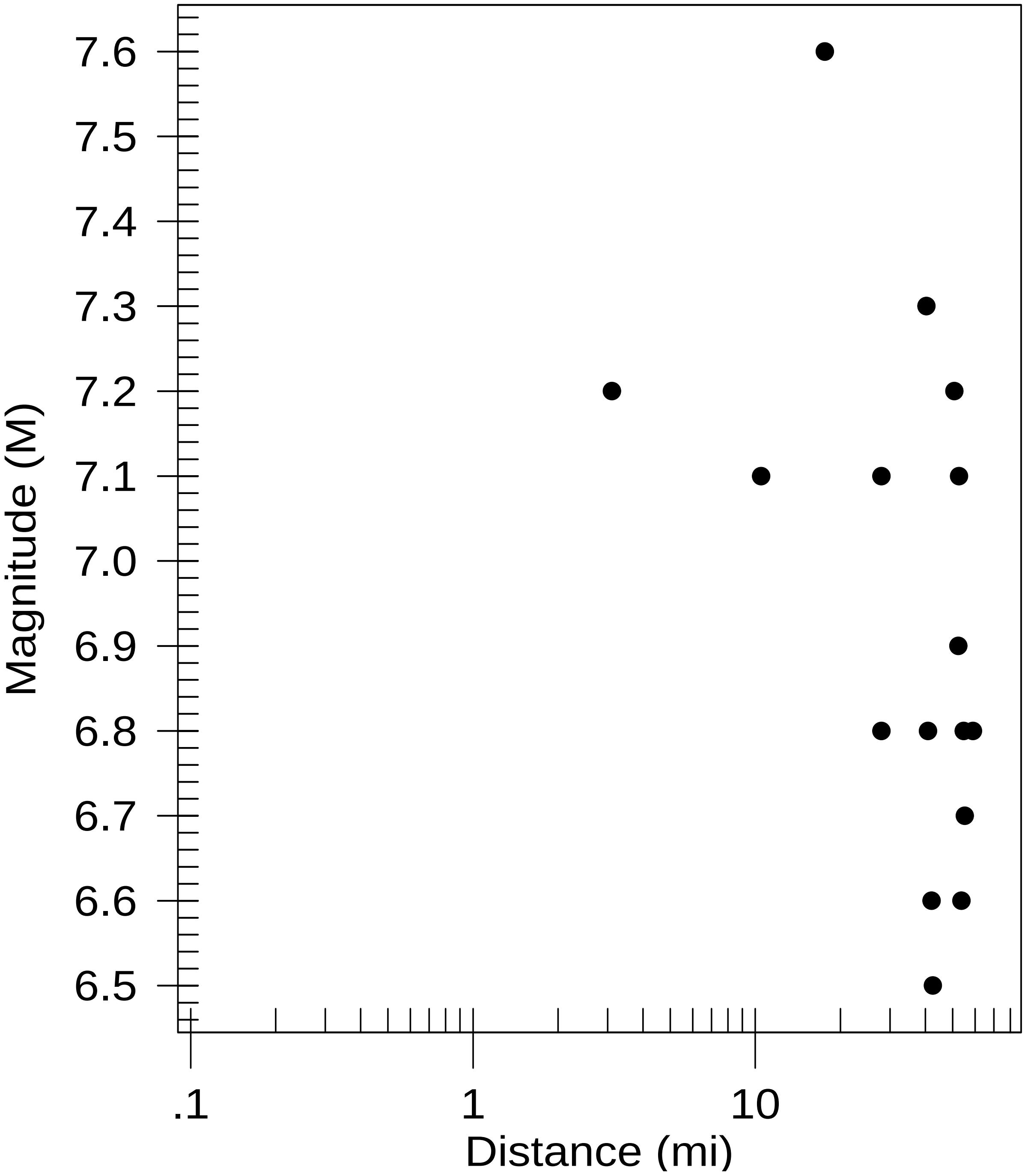
Newman



W.O. 7557-A-SC
PLATE C-4

EARTHQUAKE MAGNITUDES & DISTANCES

Newman



W.O. 7557-A-SC
PLATE C-5

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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: 7557
                                                     DATE: 01-08-2019

JOB NAME: Newman

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT


SITE COORDINATES:
   SITE LATITUDE:  33.0528
   SITE LONGITUDE:  117.2998

SEARCH DATES:
           START DATE:   1800
           END DATE:   2019

SEARCH RADIUS:
           62.4 mi
           100.4 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:   1  Depth Source:  A
   Basement Depth:  .01 km      Campbell SSR:  0     Campbell SHR:  0
   COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km):  3.0
```

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                              -------------------------
                              EARTHQUAKE SEARCH RESULTS
                              -------------------------

Page  1
-------------------------------------------------------------------------------
    |       |        |          |  TIME   |     |     | SITE  |SITE|  APPROX.
FILE|  LAT. |  LONG. |   DATE   |  (UTC)  |DEPTH|QUAKE|  ACC. | MM |  DISTANCE
CODE| NORTH |  WEST  |          | H M Sec | (km)| MAG.|   g   |INT.|  mi  [km]
----+-------+--------+----------+---------+-----+-----+-------+----+------------
DMG |33.0000|117.3000|11/22/1800|2130 0.0|  0.0| 6.50| 0.555 |  X |  3.6(  5.9)
MGI |33.0000|117.0000|09/21/1856| 730 0.0|  0.0| 5.00| 0.062 | VI | 17.7( 28.5)
MGI |32.8000|117.1000|05/25/1803| 0 0 0.0|  0.0| 5.00| 0.052 | VI | 20.9( 33.7)
DMG |32.7000|117.2000|05/27/1862|20 0 0.0|  0.0| 5.90| 0.075 | VII| 25.0( 40.3)
T-A |32.6700|117.1700|12/00/1856| 0 0 0.0|  0.0| 5.00| 0.040 |  V | 27.5( 44.2)
T-A |32.6700|117.1700|10/21/1862| 0 0 0.0|  0.0| 5.00| 0.040 |  V | 27.5( 44.2)
T-A |32.6700|117.1700|05/24/1865| 0 0 0.0|  0.0| 5.00| 0.040 |  V | 27.5( 44.2)
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 | 33.8( 54.4)
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( 67.1)
DMG |33.7000|117.4000|04/11/1910| 757 0.0|  0.0| 5.00| 0.024 | IV | 45.1( 72.5)
DMG |33.7000|117.4000|05/15/1910|1547 0.0|  0.0| 6.00| 0.043 | VI | 45.1( 72.5)
DMG |33.7000|117.4000|05/13/1910| 620 0.0|  0.0| 5.00| 0.024 | IV | 45.1( 72.5)
DMG |33.6990|117.5110|05/31/1938| 83455.4| 10.0| 5.50| 0.031 |  V | 46.2( 74.4)
DMG |33.7100|116.9250|09/23/1963|144152.6| 16.5| 5.00| 0.021 | IV | 50.3( 80.9)
DMG |33.0000|116.4330|06/04/1940|1035 8.3|  0.0| 5.10| 0.022 | IV | 50.3( 81.0)
DMG |33.7500|117.0000|04/21/1918|223225.0|  0.0| 6.80| 0.064 | VI | 51.1( 82.3)
DMG |33.7500|117.0000|06/06/1918|2232 0.0|  0.0| 5.00| 0.021 | IV | 51.1( 82.3)
GSG |33.4200|116.4890|07/07/2010|235333.5| 14.0| 5.50| 0.027 |  V | 53.2( 85.7)
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 | 53.4( 86.0)
DMG |33.8000|117.0000|12/25/1899|1225 0.0|  0.0| 6.40| 0.046 | VI | 54.4( 87.5)
MGI |33.8000|117.6000|04/22/1918|2115 0.0|  0.0| 5.00| 0.019 | IV | 54.4( 87.6)
DMG |33.6170|117.9670|03/11/1933| 154 7.8|  0.0| 6.30| 0.043 | VI | 54.8( 88.1)
PAS |33.5010|116.5130|02/25/1980|104738.5| 13.6| 5.50| 0.026 |  V | 55.0( 88.4)
GSP |33.5080|116.5140|10/31/2001|075616.6| 15.0| 5.10| 0.020 | IV | 55.2( 88.8)
DMG |33.5000|116.5000|09/30/1916| 211 0.0|  0.0| 5.00| 0.019 | IV | 55.5( 89.4)
GSP |33.4315|116.4427|06/10/2016|080438.7| 12.3| 5.19| 0.021 | IV | 56.0( 90.1)
T-A |32.2500|117.5000|01/13/1877|20 0 0.0|  0.0| 5.00| 0.019 | IV | 56.6( 91.1)
DMG |33.6170|118.0170|03/14/1933|19 150.0|  0.0| 5.10| 0.020 | IV | 56.8( 91.4)
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 | 58.8( 94.6)
DMG |33.6830|118.0500|03/11/1933| 658 3.0|  0.0| 5.50| 0.023 | IV | 61.4( 98.7)
GSP |32.3290|117.9170|06/15/2004|222848.2| 10.0| 5.30| 0.020 | IV | 61.5( 99.0)

*******************************************************************************
-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 3.6 MILES (5.9 km) AWAY.

LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 6.8

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.555 g
```

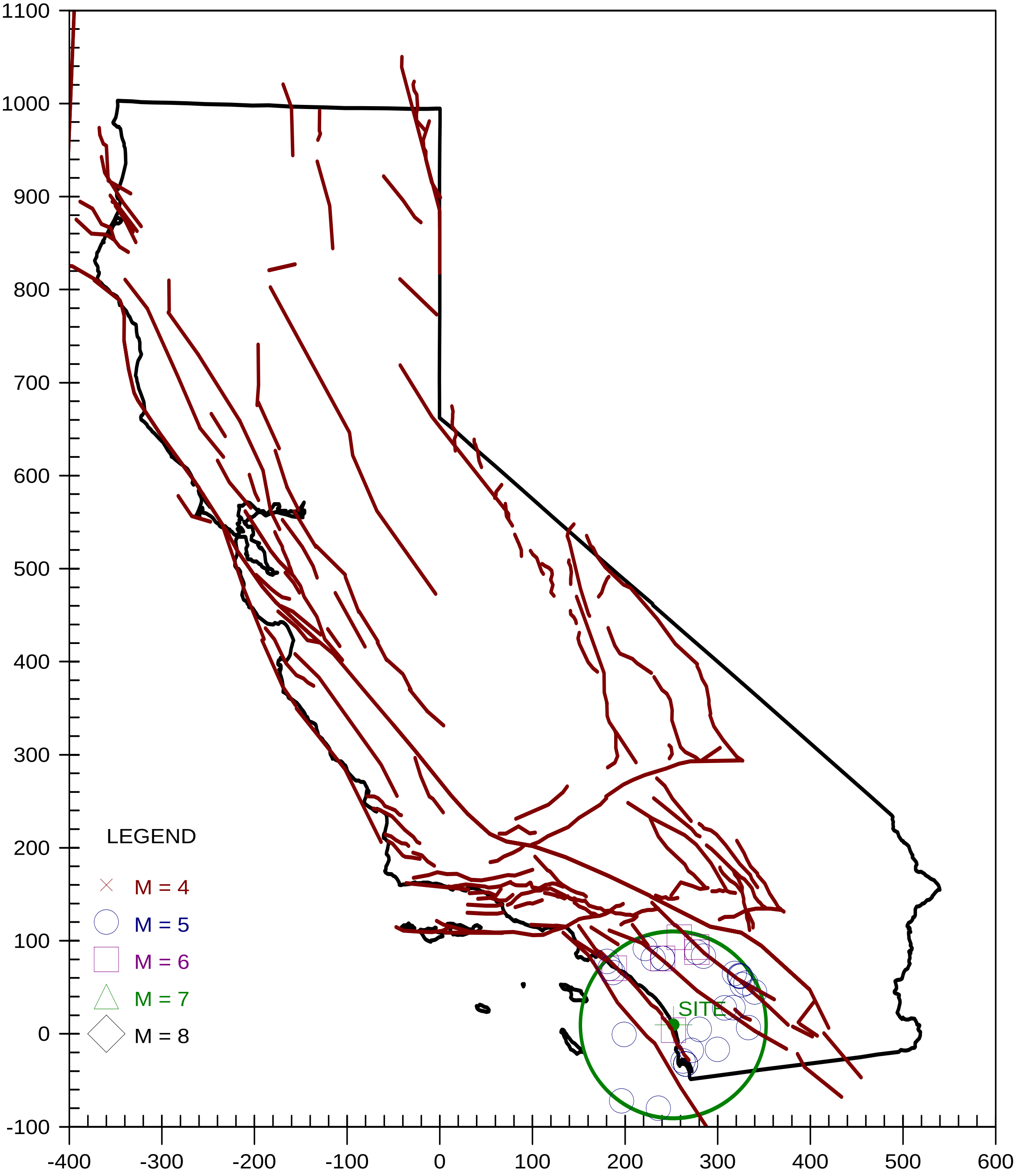
```
COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
  a-value=  0.924
  b-value=  0.392
  beta-value=  0.904


------------------------------------
TABLE OF MAGNITUDES AND EXCEEDANCES:
------------------------------------

  Earthquake | Number of Times | Cumulative
   Magnitude |     Exceeded    | 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

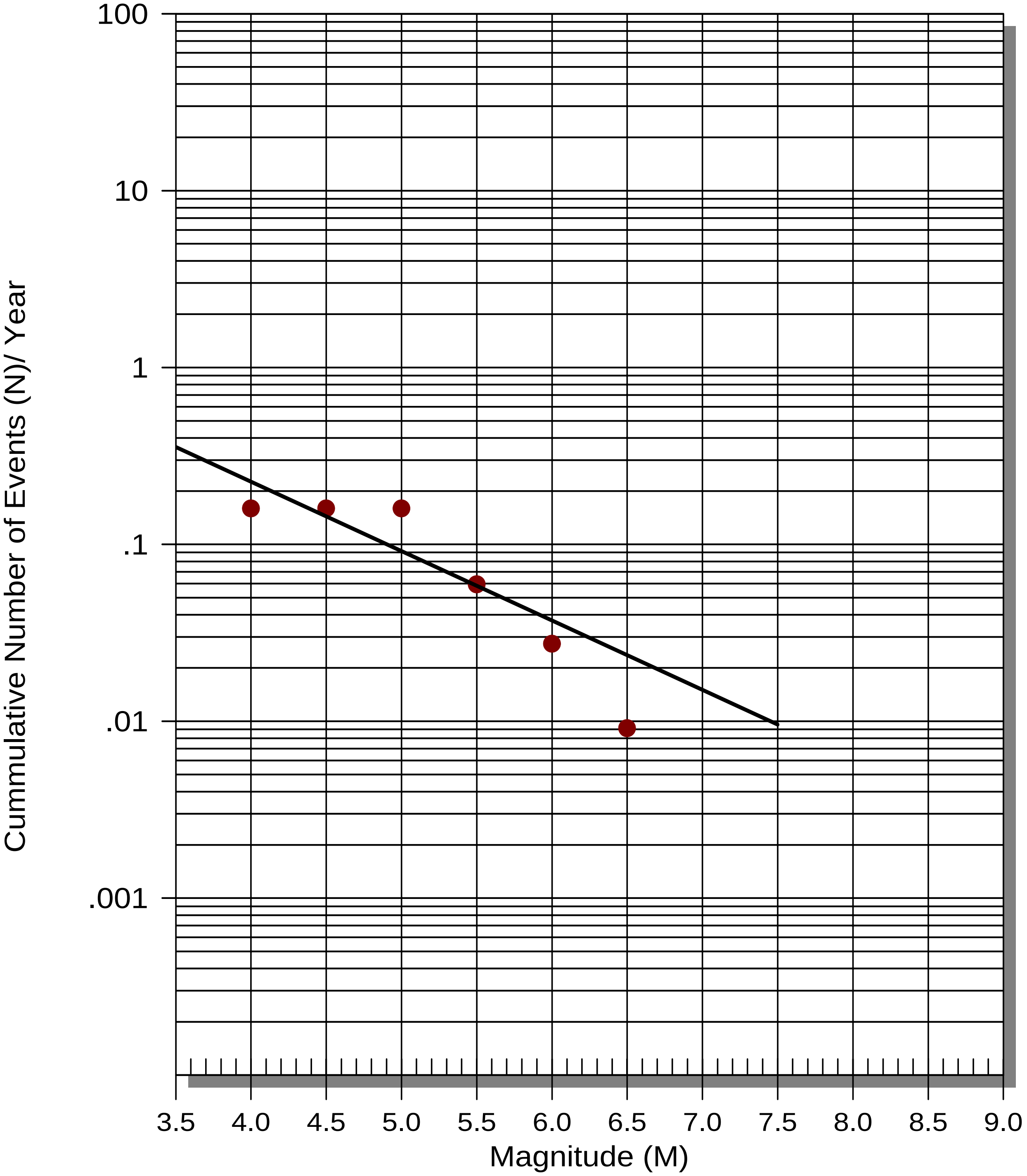
Newman



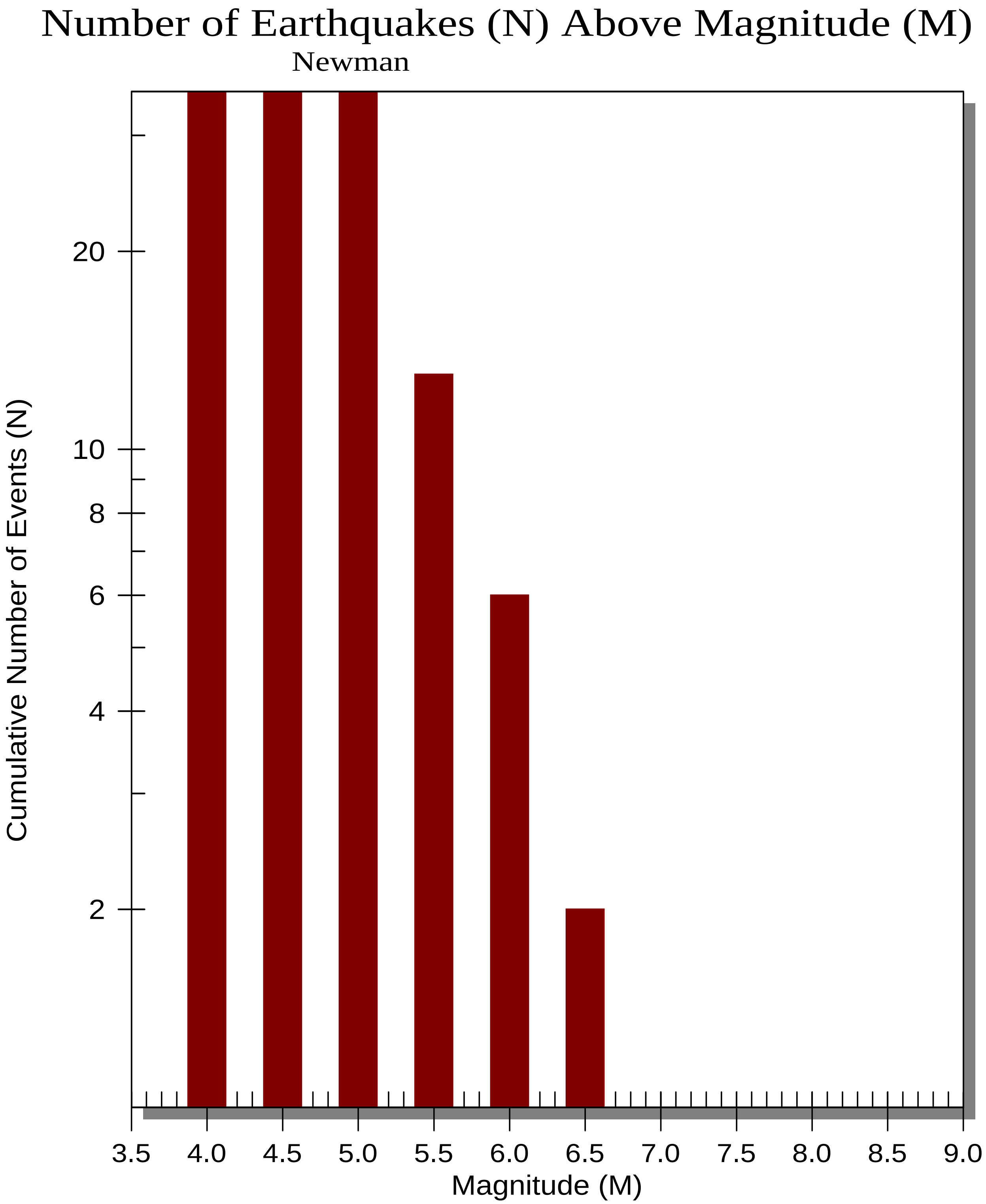
W.O. 7557-A-SC
PLATE C-9

EARTHQUAKE RECURRENCE CURVE

Newman



W.O. 7557-A-SC
PLATE C-10

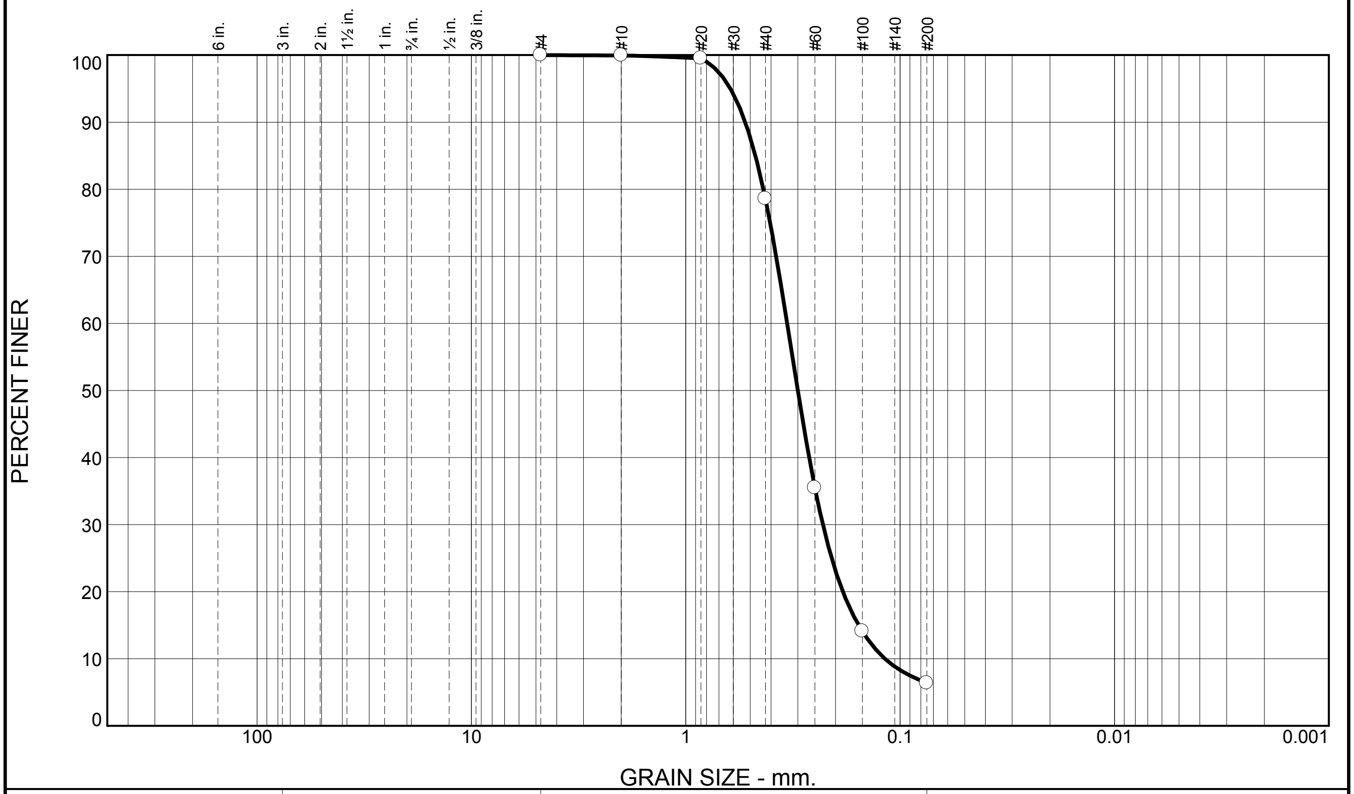


W.O. 7557-A-SC
PLATE C-11

APPENDIX D

LABORATORY DATA INCLUDING
GSI (2000) SHEAR STRENGTH DATA

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.1	21.3	72.2	6.4	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.9		
#20	99.5		
#40	78.6		
#60	35.5		
#100	14.1		
#200	6.4		

* (no specification provided)

Soil Description

Light Brown Sand w/Silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.5277 D_{85}= 0.4732 D_{60}= 0.3367

D_{50}= 0.3002 D_{30}= 0.2295 D_{15}= 0.1560

D_{10}= 0.1178 C_u= 2.86 C_c= 1.33

Classification

USCS= SP-SM AASHTO=

Remarks

Source of Sample: B-2 **Depth:** 1-4
Sample Number: B-2

Date: 1-4-18

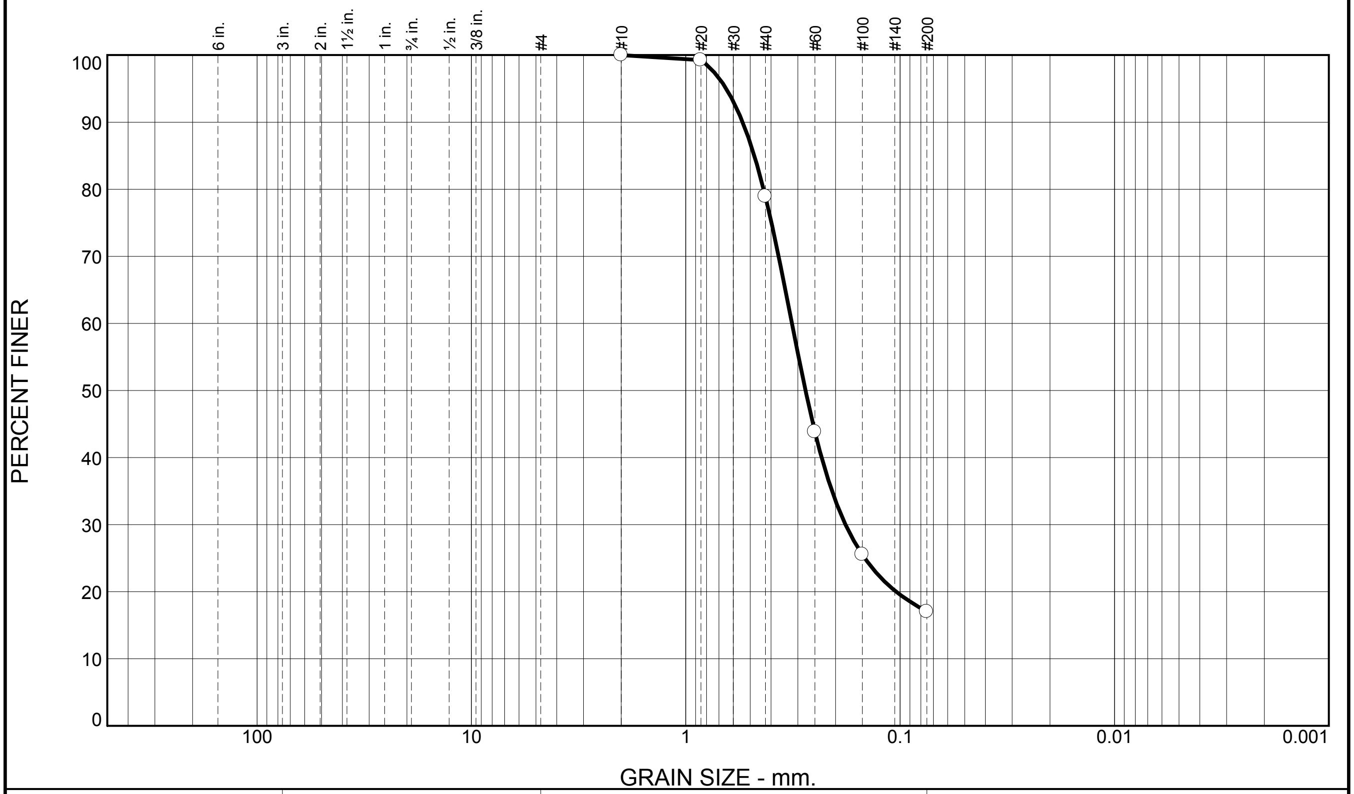
GeoSoils, Inc.

Client: Newman
Project: 216 Neptune

Project No: 7557-A-SC **Figure** D-1

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

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.0	21.1	61.9	17.0	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#10	100.0		
#20	99.2		
#40	78.9		
#60	43.8		
#100	25.5		
#200	17.0		

* (no specification provided)

Soil Description

Dark Yellowish Brown Silty Sand

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.5427 D_{85}= 0.4788 D_{60}= 0.3198

D_{50}= 0.2765 D_{30}= 0.1797 D_{15}=

D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

Source of Sample: B-4 **Depth:** 15.0
Sample Number: B-4

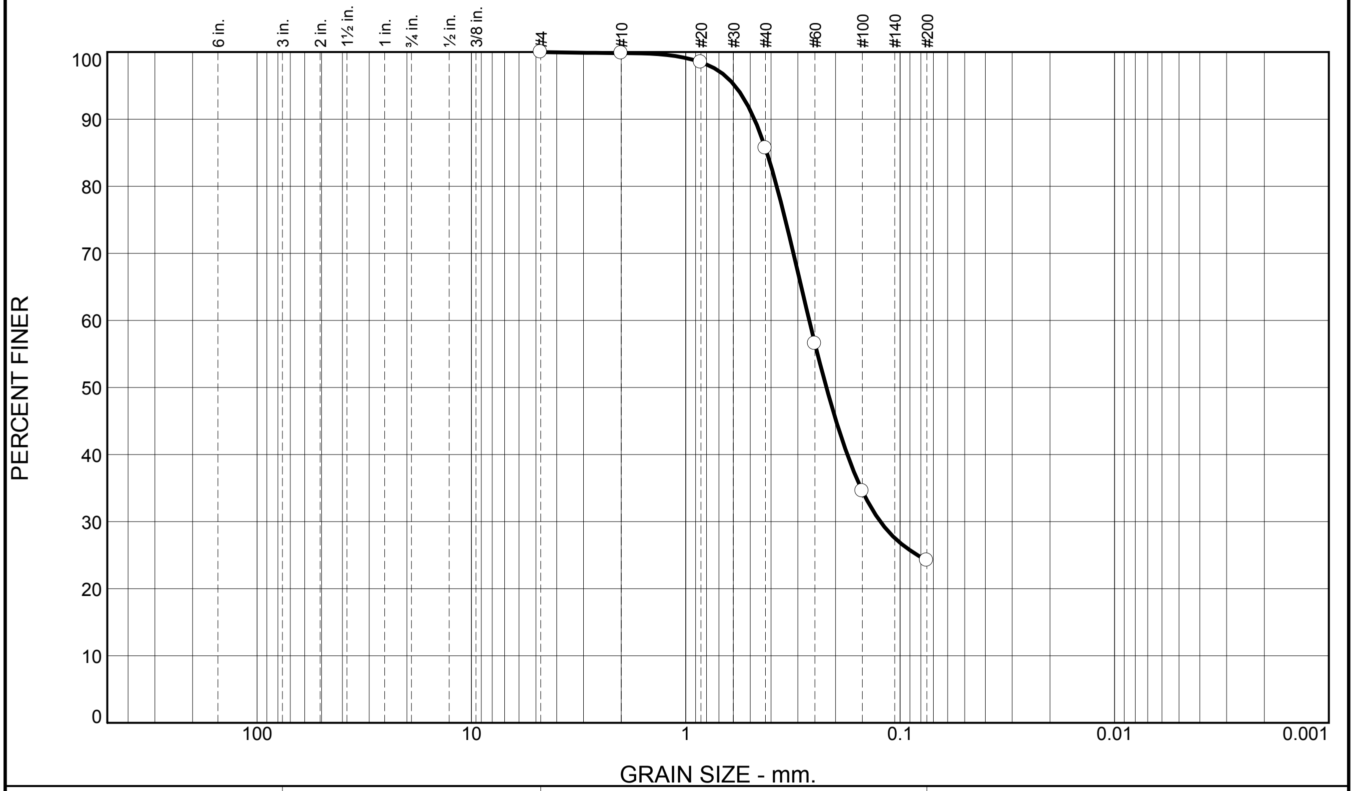
Date: 3-23-20

GeoSoils, Inc.

Client: Newman
Project: 216 Neptune
Project No: 7557-A-SC **Figure** D-2

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

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	0.2	14.1	61.5	24.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.8		
#20	98.5		
#40	85.7		
#60	56.5		
#100	34.6		
#200	24.2		

* (no specification provided)

Soil Description

Dark Brown Silty Sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 0.4782 D_{85}= 0.4182 D_{60}= 0.2657

D_{50}= 0.2209 D_{30}= 0.1238 D_{15}=

D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

Source of Sample: B-4 **Depth:** 37.5

Sample Number: B-4

Date: 3-23-20

GeoSoils, Inc.

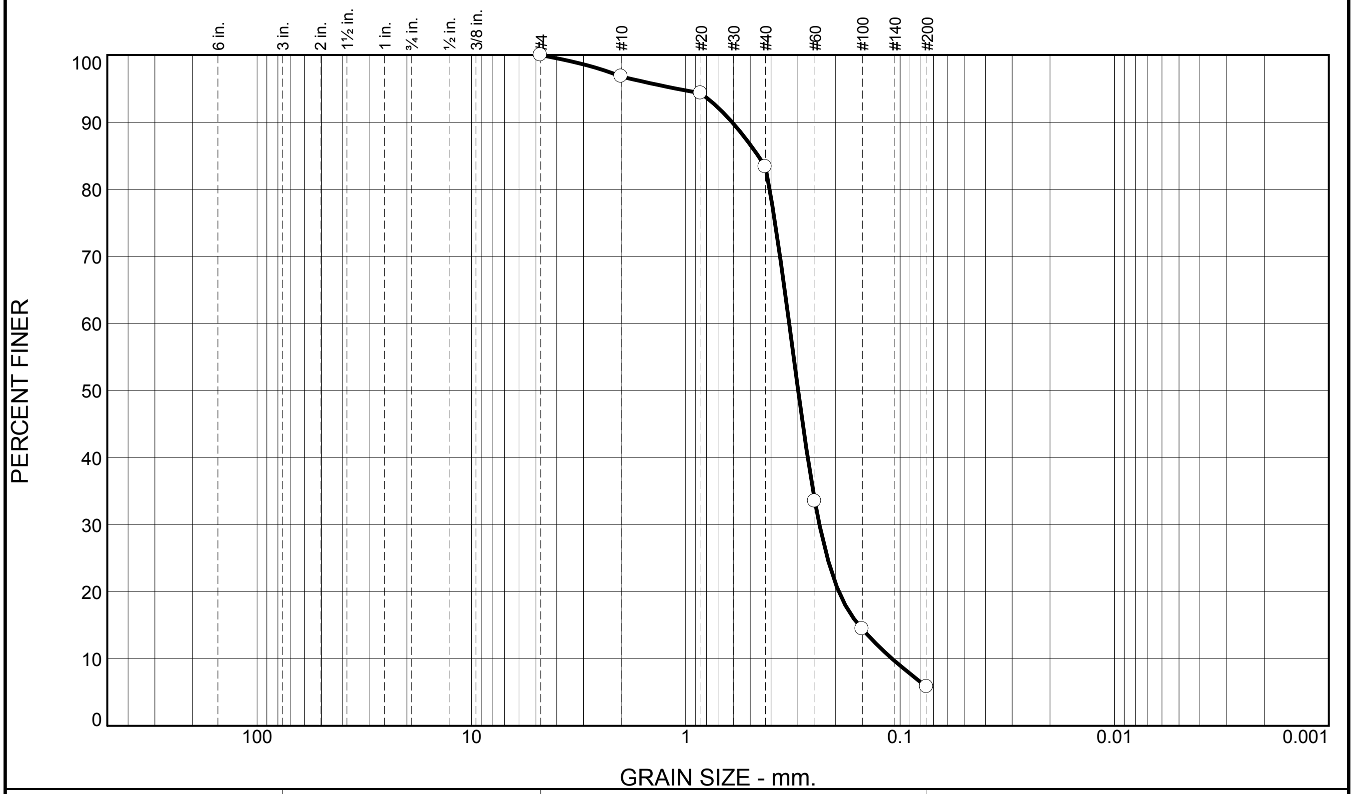
Client: Newman

Project: 216 Neptune

Project No: 7557-A-SC **Figure** D-3

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

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	0.0	3.2	13.5	77.5	5.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	96.8		
#20	94.3		
#40	83.3		
#60	33.5		
#100	14.4		
#200	5.8		

* (no specification provided)

Soil Description

Dark Yellowish Brown Sand w/Silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 0.6078	D_{85}= 0.4599	D_{60}= 0.3295
D_{50}= 0.2993	D_{30}= 0.2381	D_{15}= 0.1554
D_{10}= 0.1090	C_u= 3.02	C_c= 1.58

Classification

USCS= SP-SM AASHTO=

Remarks

Source of Sample: B-4 **Depth:** 45.0
Sample Number: B-4

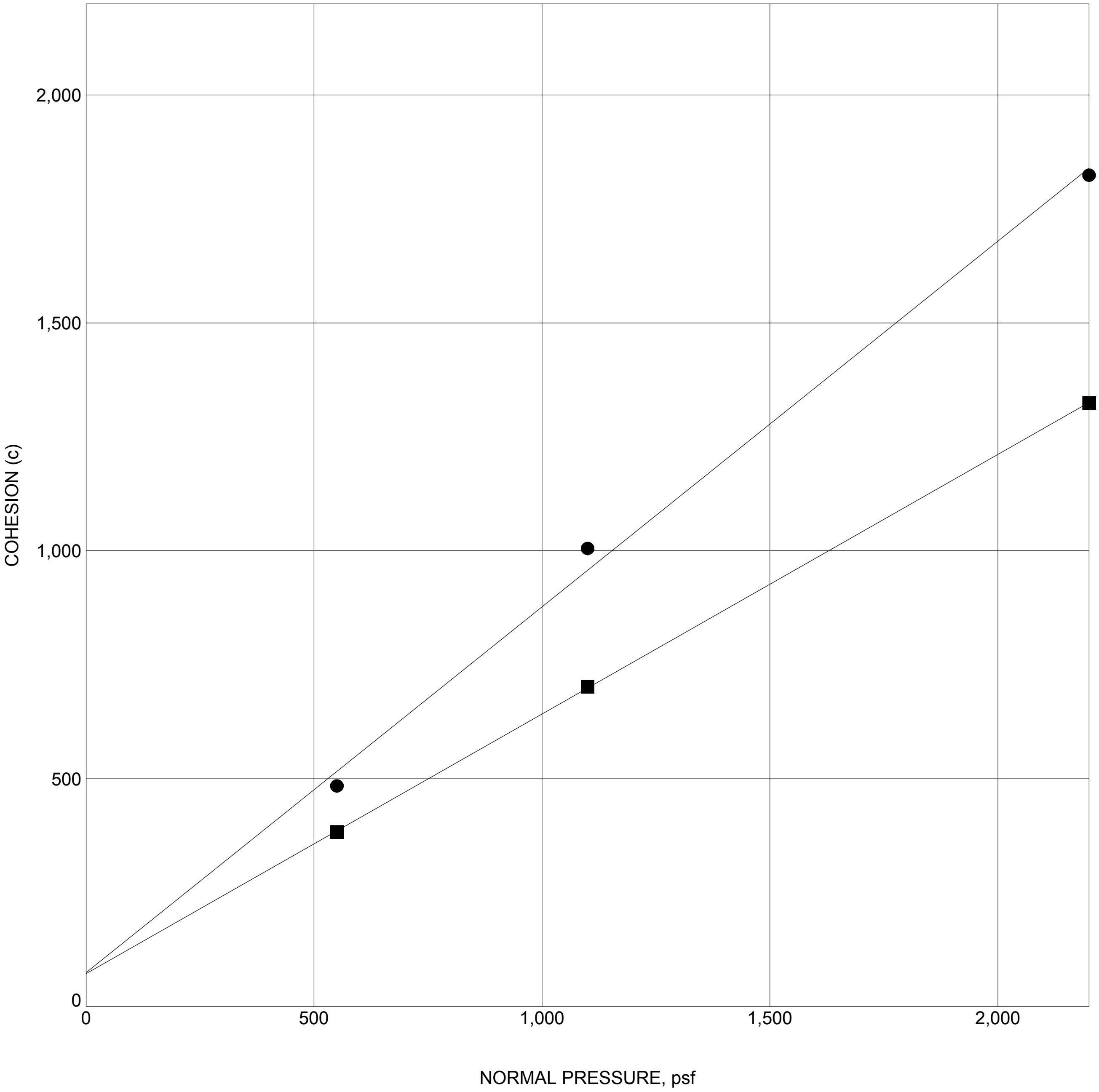
Date: 3-23-20

GeoSoils, Inc.

Client: Newman
Project: 216 Neptune

Project No: 7557-A-SC **Figure** D-4

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



	Sample	Depth/El.	Primary/Residual Shear	Sample Type	γ_d	MC%	c	ϕ
●	B-4	7.5	Primary Shear	Natural	103.1	19.2	75	39
■	B-4	7.5	Residual Shear	Natural	103.1	19.2	72	30

Note: Sample Innundated prior to testing

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92010
Telephone: 760-438-3155
Fax: 760-931-0915

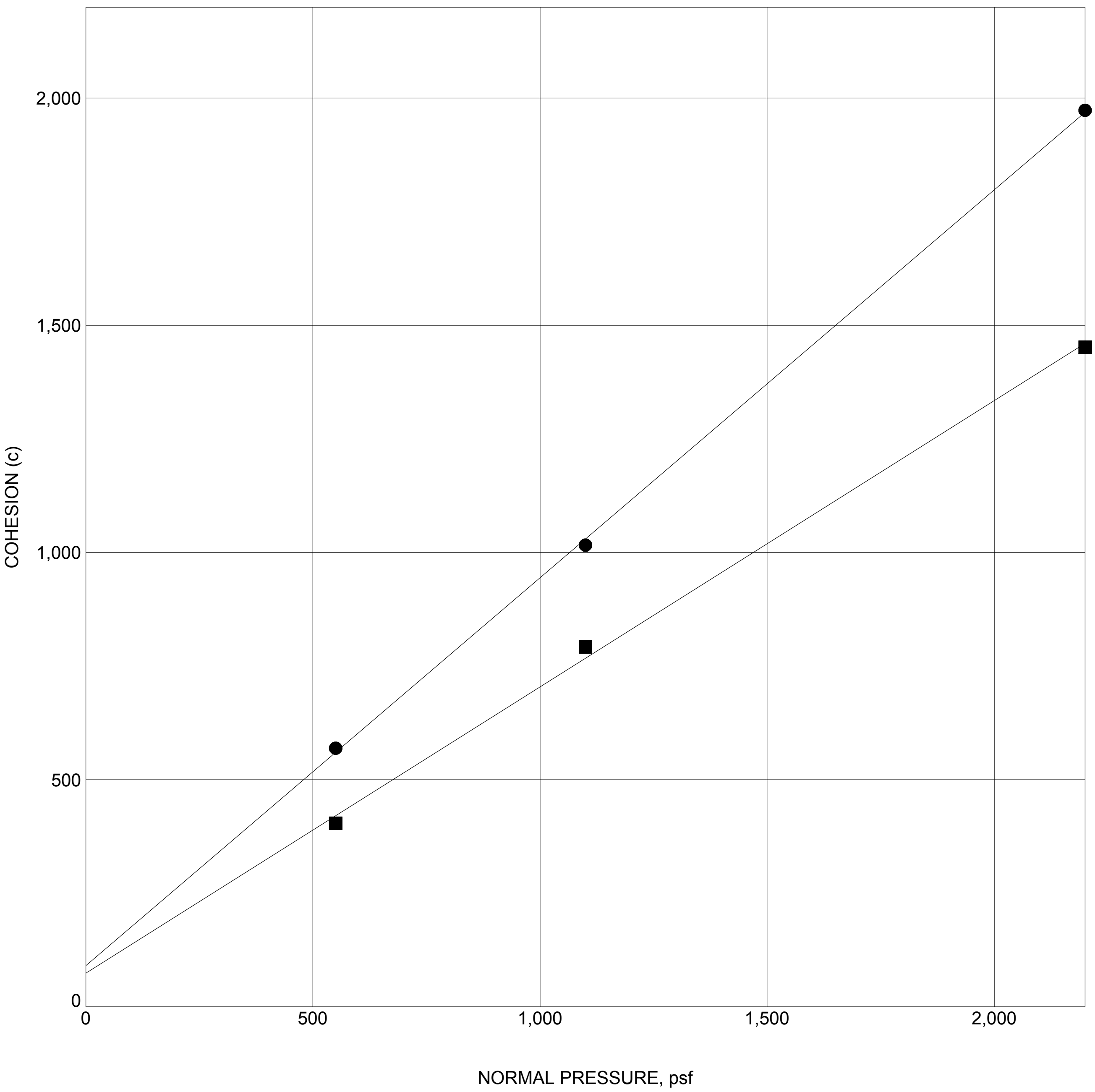
DIRECT SHEAR TEST

Project: Newman

Number: 7557-A-SC

Date: April 2020

Figure: D - 5



Sample	Depth/El.	Primary/Residual Shear	Sample Type	γ_d	MC%	c	ϕ
● B-4	15.0	Primary Shear	Natural	110.0	14.8	91	40
■ B-4	15.0	Residual Shear	Natural	110.0	14.8	74	32

Note: Sample Innundated prior to testing

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92010
Telephone: 760-438-3155
Fax: 760-931-0915

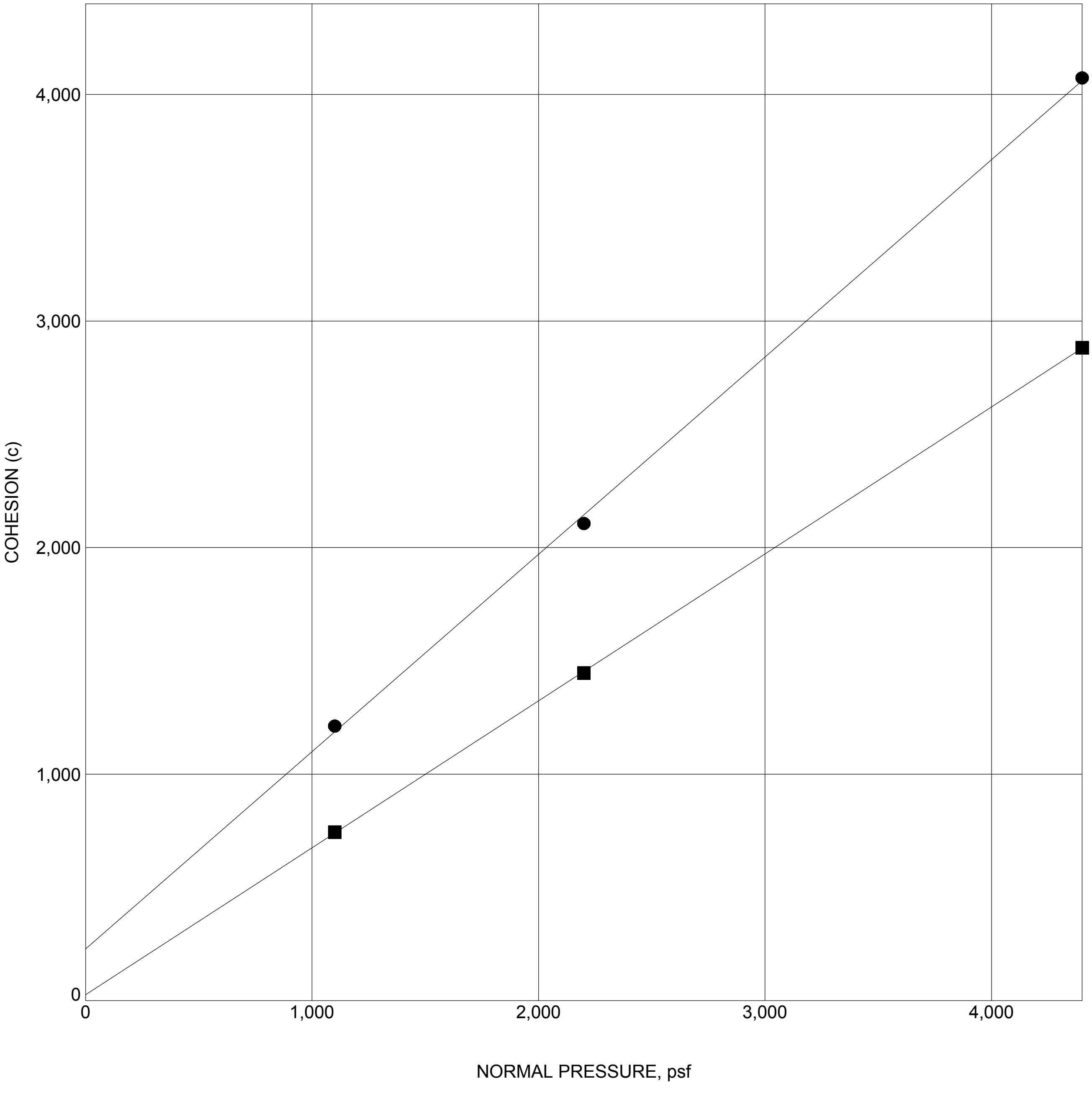
DIRECT SHEAR TEST

Project: Newman

Number: 7557-A-SC

Date: April 2020

Figure: D - 6



	Sample	Depth/El.	Primary/Residual Shear	Sample Type	γ_d	MC%	c	ϕ
●	B-4	25.0	Primary Shear	Natural	106.2	18.4	229	41
■	B-4	25.0	Residual Shear	Natural	106.2	18.4	26	33

Note: Sample Innundated prior to testing

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92010
Telephone: 760-438-3155
Fax: 760-931-0915

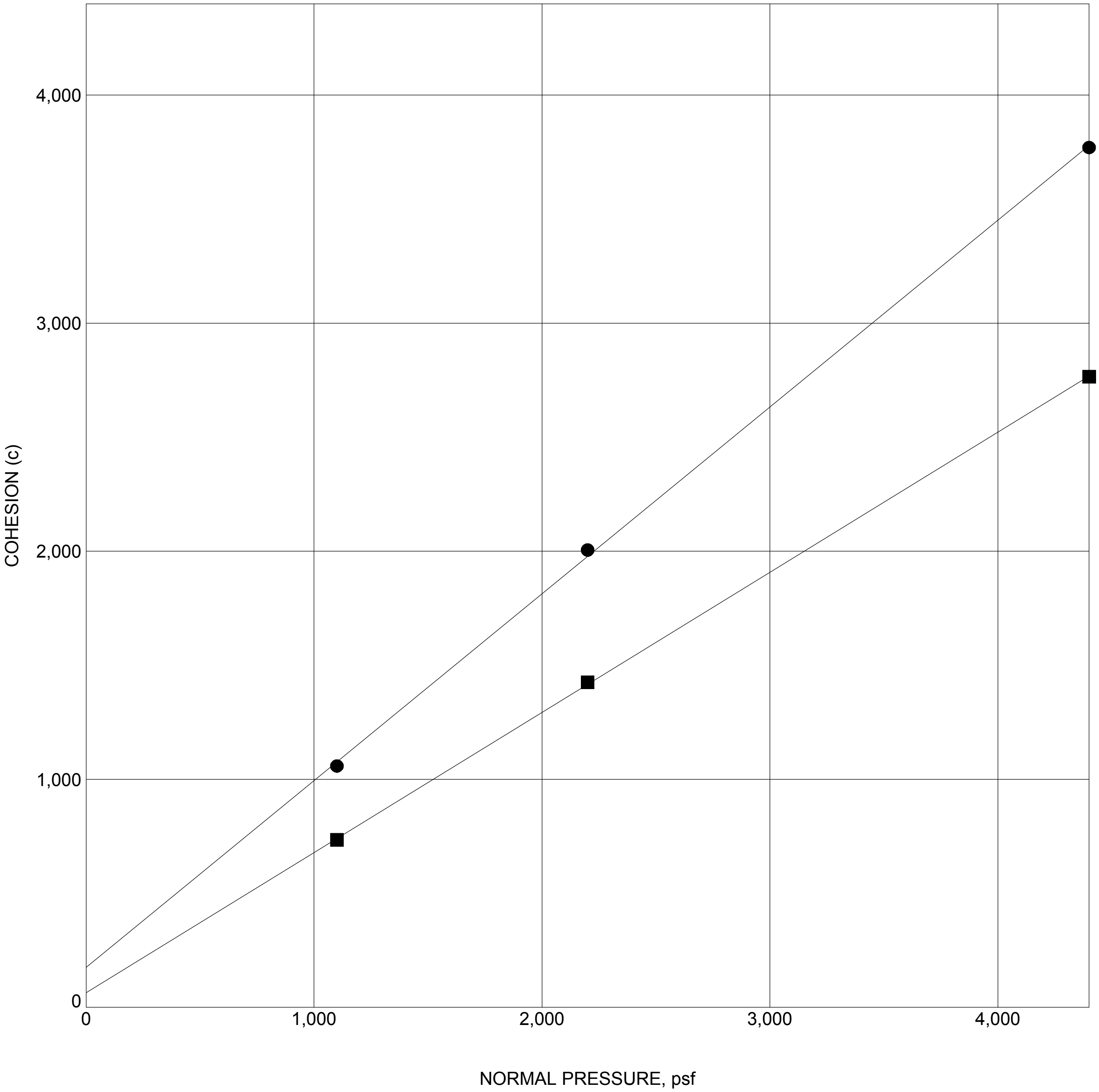
DIRECT SHEAR TEST

Project: Newman

Number: 7557-A-SC

Date: April 2020

Figure: D - 7



	Sample	Depth/El.	Primary/Residual Shear	Sample Type	γ_d	MC%	c	ϕ
●	B-4	32.5	Primary Shear	Natural	104.0	18.9	176	39
■	B-4	32.5	Residual Shear	Natural	104.0	18.9	64	32

Note: Sample Innundated prior to testing

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92010
Telephone: 760-438-3155
Fax: 760-931-0915

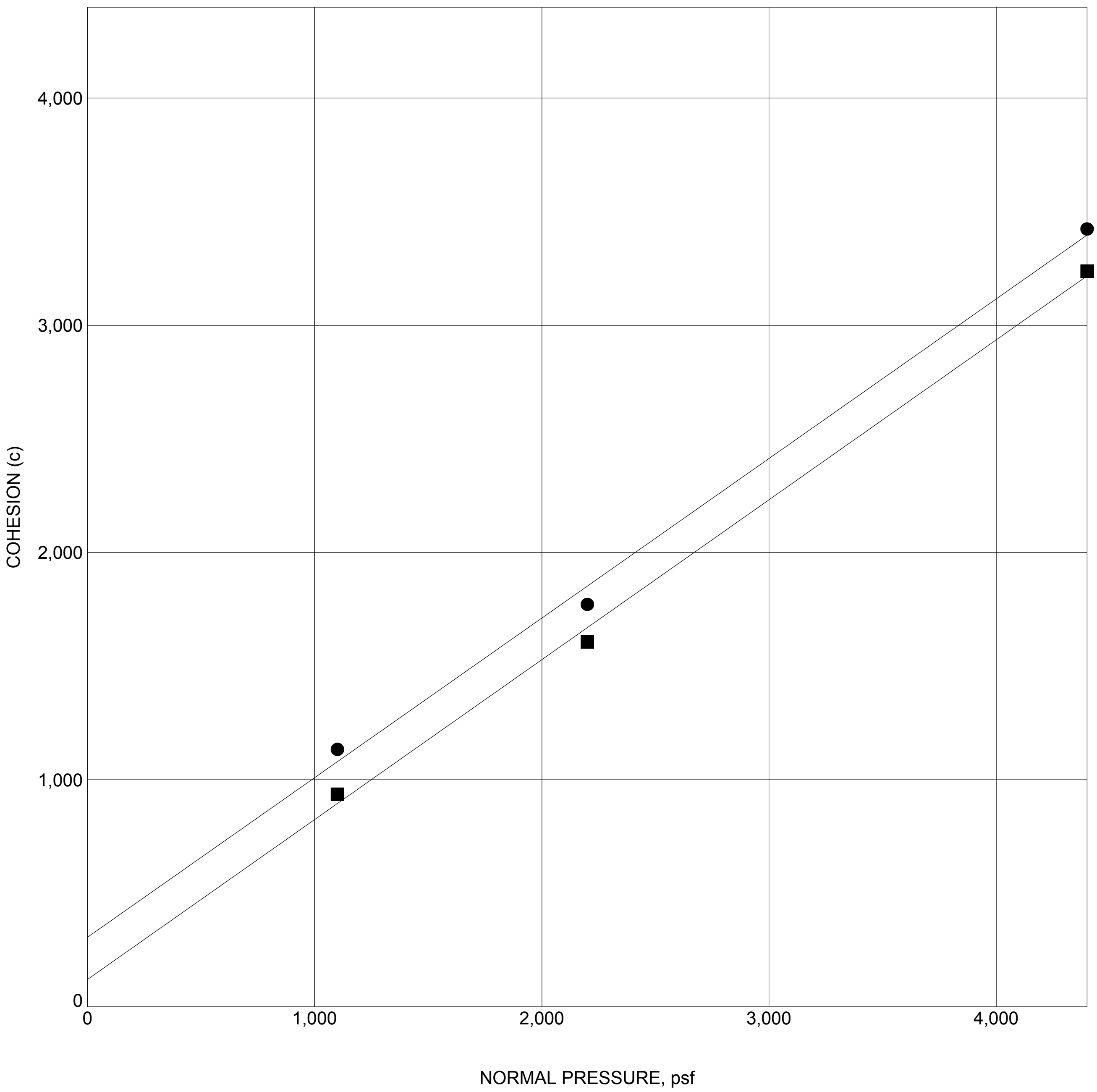
DIRECT SHEAR TEST

Project: Newman

Number: 7557-A-SC

Date: April 2020

Figure: D - 8



	Sample	Depth/El.	Primary/Residual Shear	Sample Type	γ_d	MC%	c	ϕ
●	B-4	45.0	Primary Shear	Natural	91.9	25.8	307	35
■	B-4	45.0	Residual Shear	Natural	91.9	25.8	121	35

Note: Sample Innundated prior to testing

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92010
Telephone: 760-438-3155
Fax: 760-931-0915

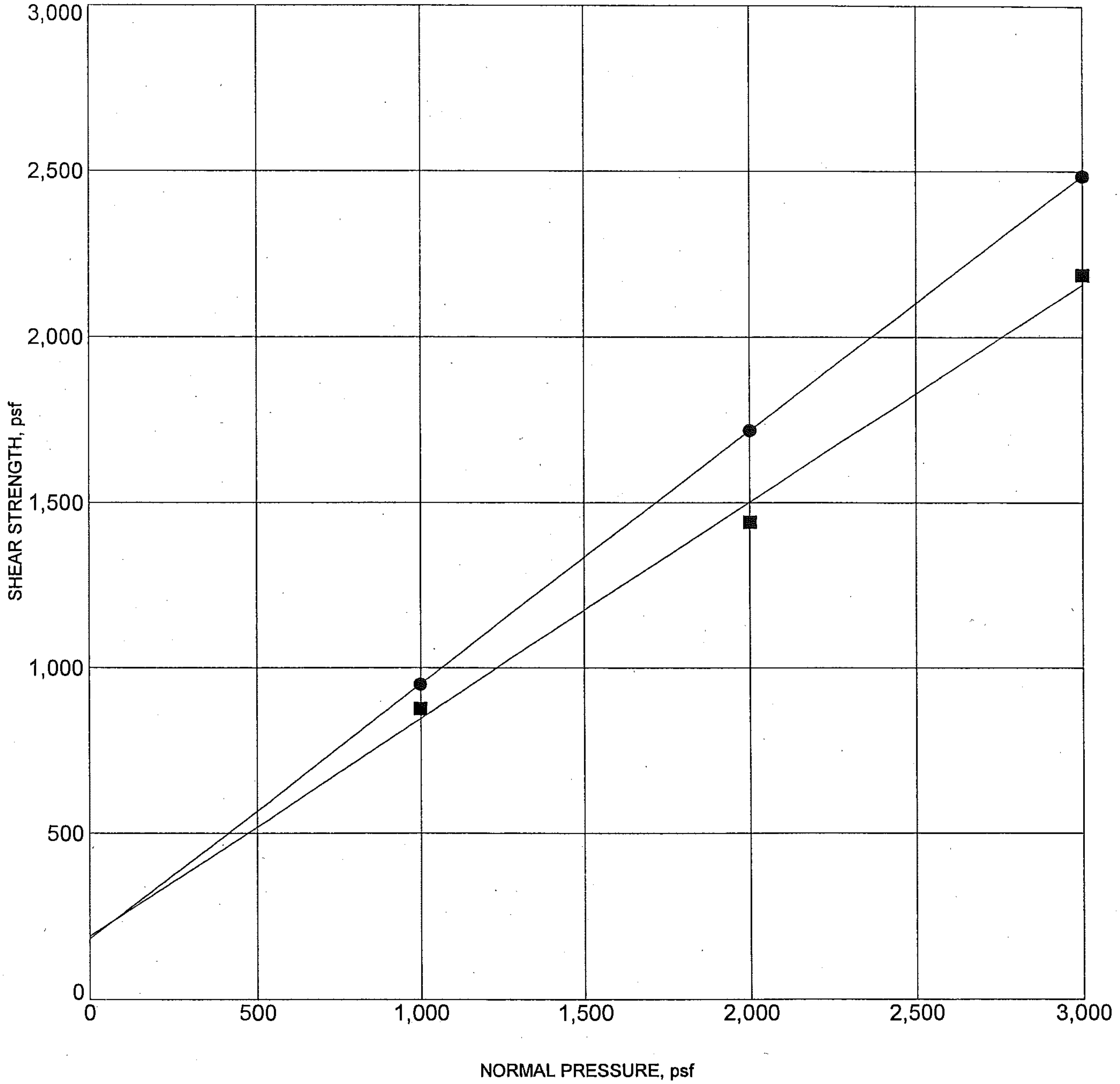
DIRECT SHEAR TEST

Project: Newman

Number: 7557-A-SC

Date: April 2020

Figure: D - 9



	Sample	Depth/El.	Classification	γ_d	MC%	c	ϕ
●	**B-1**	**5.0**	**Silty Sand**		**3**	**183**	**38**
■	**B-1**	**5.0**			**3**	**190**	**33**

US DIRECT SHEAR 2915.GPJ US LAB.GDT 8/3/00

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

DIRECT SHEAR TEST

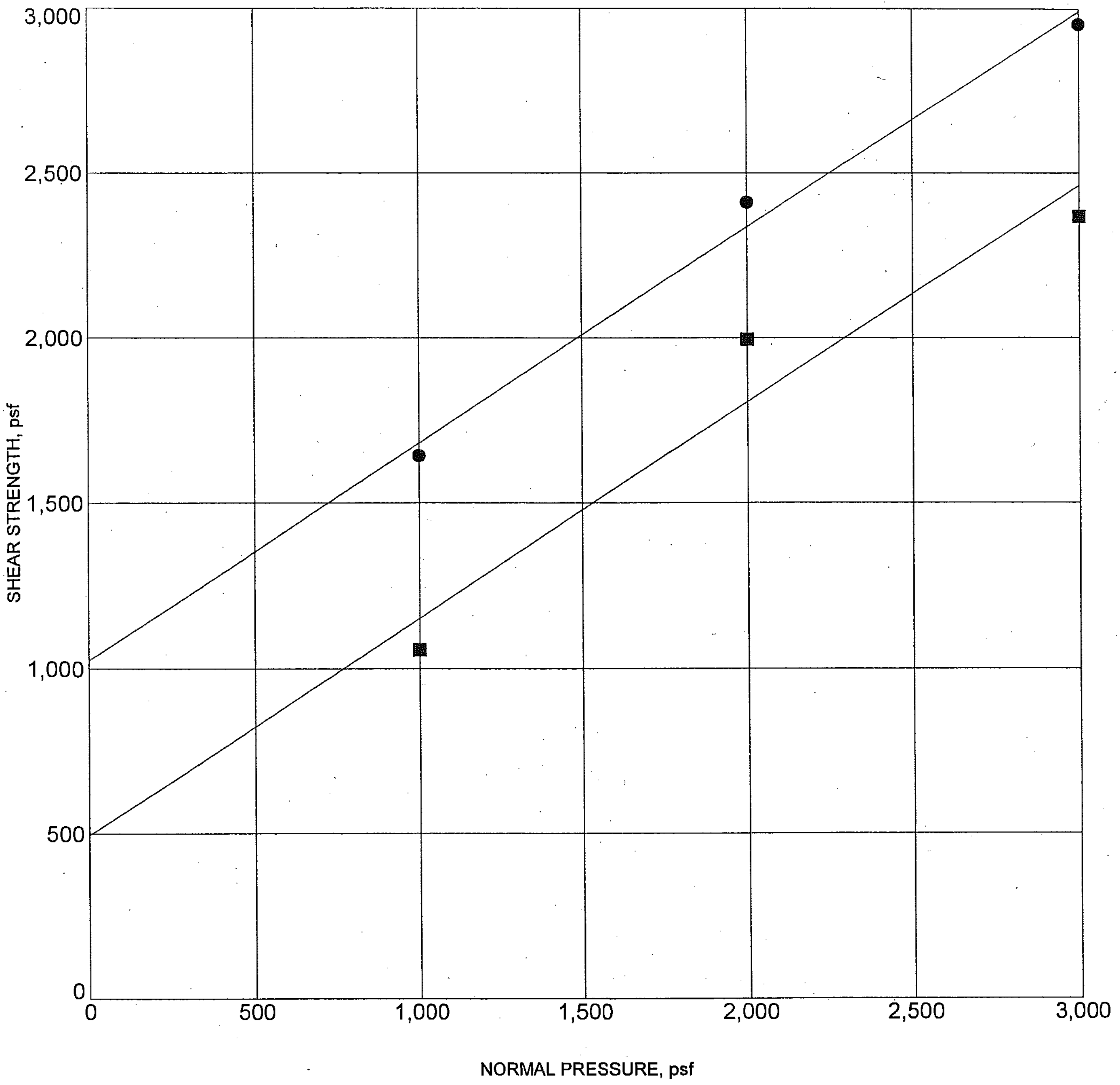
Project: REFOLD

Number: 2915-A-SC

Date: August 2000

Figure: - 1

PLATE D-10



	Sample	Depth/El.	Classification	γ_d	MC%	c	ϕ
●	**B-1**	**40.0**	**Clayey Sand**		**5**	**1025**	**33**
■	**B-1**	**40.0**			**5**	**496**	**33**

US DIRECT SHEAR 2915.GPJ US LAB.GDT 8/3/00

GeoSoils, Inc.
5741 Palmer Way
Carlsbad, CA 92008
Telephone: (760) 438-3155
Fax: (760) 931-0915

DIRECT SHEAR TEST

Project: REFOLD

Number: 2915-A-SC

Date: August 2000

Figure: - 1

SUMMARY OF LABORATORY TEST DATA

GeoSoils, Inc.
5741 Palmer Way, Ste D
Carlsbad CA 92010
W.O.: 7557-A-SC
Project: Newman

QCI Project No.: 19-029-001a
Date: January 10, 2019
Summarized by: MA

Corrosivity Test Results

Sample ID	Sample Depth (ft)	pH CT-532 (643)	Chloride CT-422 (ppm)	Sulfate CT-417 % By Weight	Resistivity CT-532 (643) (ohm-cm)
B-2	1-4’	6.69	16	0.019	1500

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. We used an anisotropic angular range between 5 and -5 degrees for the Torrey Sandstone, owing to its cross-bedded nature.

4. Pseudo-static earthquake loading. A seismic coefficient (*k)* of 0.15 and a peak horizontal ground acceleration of 0.618 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	125.0	0.0	30.0
Quaternary Colluvium (Qcol)	100.0	105.0	100.0	27.0
Quaternary Old Paralic Deposits (Qop)	116.0	125.0	175.0	32.0
Tertiary Torrey Sandstone - Parallel Bed (Tt)	132.0	135.0	700.00	32.0
Tertiary Torrey Sandstone - Cross-Bed (Tt)	132.0	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 0.618g 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, ±5,000 trial surfaces were analyzed for each section for either static or pseudo-static analyses.

Results of Slope Stability Calculations

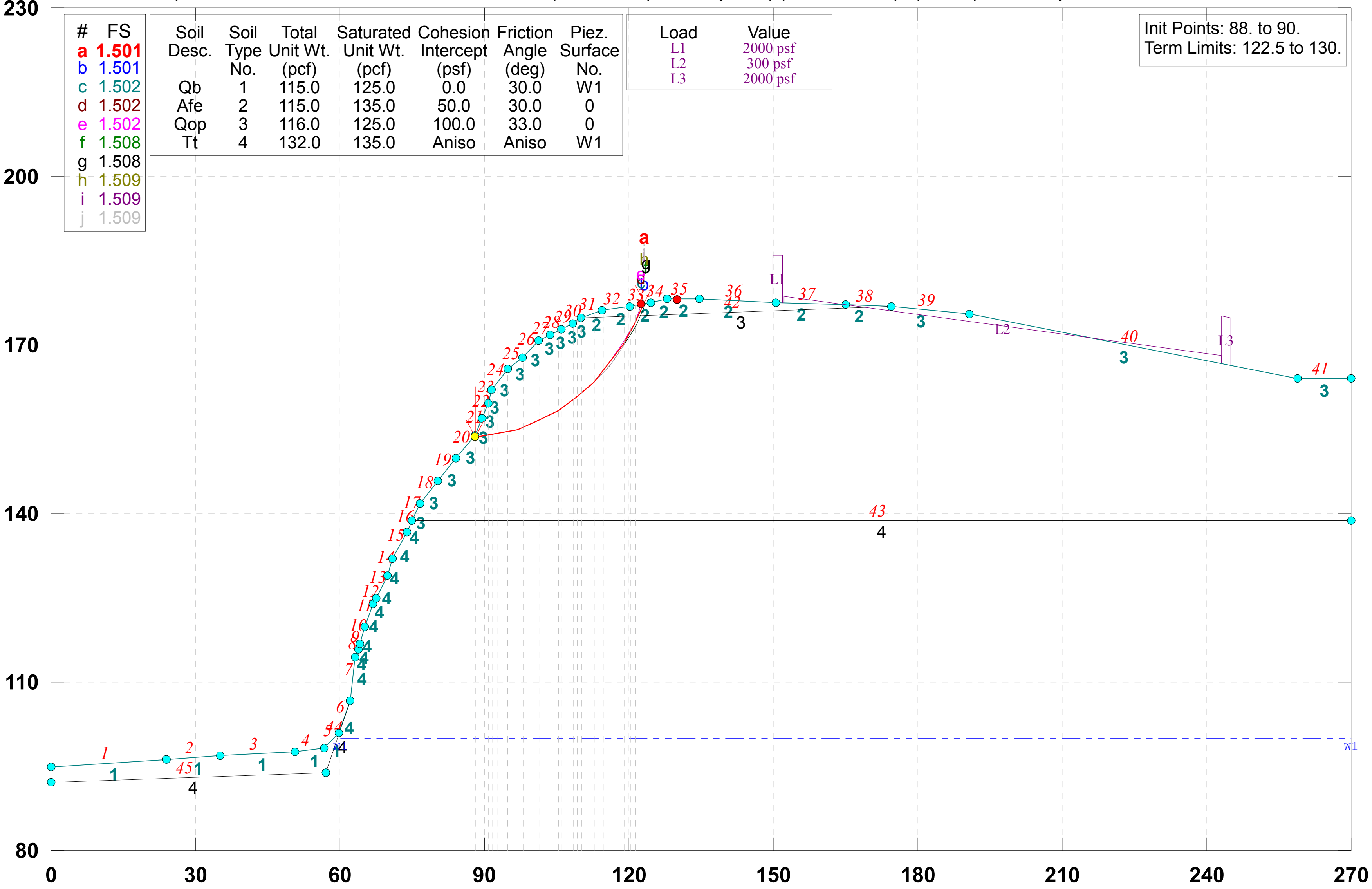
Table E-2 provides a summary of the results of our stability analyses along Geologic Cross Section X-X' (see Plates 1 and 2). Computer printouts from the GSTABL7 program are also included herein.

TABLE E-2 - SUMMARY OF SLOPE STABILITY ANALYSES

LOCATION	FACTOR-OF-SAFETY (FOS) EXISTING SLOPE CONDITION		METHOD	COMMENTS
	STATIC	SEISMIC		
Section X-X' Failure Through Qop	1.501 (See Plate E-1)	1.153 (See Plate E-8)	GLE (Spencer's)	Adequate Static and Seismic FOS
Section X-X' Failure Through Qop	$\geq$1.229 (See Plate E-14)	-	Modified Bishops	Postulates where hypothetical Bluff Stabilization will be needed

7557 NEWMAN, 216 NEPTUNE X-X' Qop Failure Static

x:\shared\word perfect data\carlsbad\7500\7557 newman, 216 neptune\a4 slope stability\x-x' qop failure bishop spencer.pl2 Run By: GeoSoils, Inc. 4/7/2020 11:58AM



#	FS
a	1.501
b	1.501
c	1.502
d	1.502
e	1.502
f	1.508
g	1.508
h	1.509
i	1.509
j	1.509

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Qb	1	115.0	125.0	0.0	30.0	W1
Afe	2	115.0	135.0	50.0	30.0	0
Qop	3	116.0	125.0	100.0	33.0	0
Tt	4	132.0	135.0	Aniso	Aniso	W1

Load	Value
L1	2000 psf
L2	300 psf
L3	2000 psf

Init Points: 88. to 90.
Term Limits: 122.5 to 130.

GSTABL7 v.2 FSmin=1.501

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

W.O. 7557-A-SC
PLATE E-1

```
                         ***  GSTABL7  ***
            ** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **
        ** Original Version 1.0, January 1996; Current Ver. 2.005.3, Feb. 2013 **
               (All Rights Reserved-Unauthorized Use Prohibited)
    *********************************************************************************
                       SLOPE STABILITY ANALYSIS SYSTEM
           Modified Bishop, Simplified Janbu, or GLE Method of Slices.
           (Includes Spencer & Morgenstern-Price Type Analysis)
           Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
           Nonlinear Undrained Shear Strength, Curved Phi Envelope,
           Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
           Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
    *********************************************************************************
    Analysis Run Date:        4/7/2020
    Time of Run:              11:58AM
    Run By:                   GeoSoils, Inc.
    Input Data Filename:      X:\shared\Word Perfect Data\CARLSBAD\7500\7557 Newman, 216 N
eptune\A4 Slope Stability\x-x' qop failure  bishop spencer.in
    Output Filename:          X:\shared\Word Perfect Data\CARLSBAD\7500\7557 Newman, 216 N
eptune\A4 Slope Stability\x-x' qop failure  bishop spencer.OUT
    Unit System:              English
    Plotted Output Filename:  X:\shared\Word Perfect Data\CARLSBAD\7500\7557 Newman, 216 N
eptune\A4 Slope Stability\x-x' qop failure  bishop spencer.PLT
    PROBLEM DESCRIPTION:  7557 NEWMAN, 216 NEPTUNE
                          X-X' Qop Failure Static
    BOUNDARY COORDINATES
       41 Top   Boundaries
       45 Total Boundaries
    Boundary     X-Left     Y-Left    X-Right    Y-Right    Soil Type
       No.        (ft)       (ft)       (ft)       (ft)     Below Bnd
        1          0.00      95.00      24.00      96.20        1
        2         24.00      96.20      35.15      96.90        1
        3         35.15      96.90      50.62      97.68        1
        4         50.62      97.68      56.60      98.16        1
        5         56.60      98.16      59.61     100.80        1
        6         59.61     100.80      62.10     106.50        4
        7         62.10     106.50      63.00     114.36        4
        8         63.00     114.36      63.80     115.80        4
        9         63.80     115.80      64.00     116.85        4
       10         64.00     116.85      65.10     119.80        4
       11         65.10     119.80      66.80     123.80        4
       12         66.80     123.80      67.50     124.76        4
       13         67.50     124.76      69.80     128.80        4
       14         69.80     128.80      71.00     131.93        4
       15         71.00     131.93      74.00     136.80        4
       16         74.00     136.80      74.90     138.83        4
       17         74.90     138.83      76.75     141.90        3
       18         76.75     141.90      80.49     145.76        3
       19         80.49     145.76      84.19     149.85        3
       20         84.19     149.85      88.08     153.77        3
       21         88.08     153.77      89.37     156.81        3
       22         89.37     156.81      90.67     159.81        3
       23         90.67     159.81      91.50     161.88        3
       24         91.50     161.88      94.72     165.77        3
       25         94.72     165.77      97.93     167.78        3
       26         97.93     167.78     101.30     170.64        3
       27        101.30     170.64     103.62     171.67        3
       28        103.62     171.67     105.93     172.75        3
       29        105.93     172.75     108.30     173.72        3
       30        108.30     173.72     109.95     174.84        3
       31        109.95     174.84     114.58     176.22        2
       32        114.58     176.22     120.10     176.82        2
       33        120.10     176.82     124.53     177.70        2
       34        124.53     177.70     127.85     178.12        2
       35        127.85     178.12     134.71     178.08        2
       36        134.71     178.08     150.49     177.53        2
       37        150.49     177.53     164.88     177.14        2
       38        164.88     177.14     174.53     176.85        2
       39        174.53     176.85     190.80     175.66        3
       40        190.80     175.66     259.00     164.00        3
```

```
    41         259.00     164.00     270.00     164.00         3
    42         109.95     174.84     174.53     176.85         3
    43          74.90     138.83     270.00     138.83         4
    44          57.00      94.00      62.10     106.50         4
    45           0.00      92.00      57.00      94.00         4
 User Specified Y-Origin =        80.00(ft)
 Default X-Plus Value = 0.00(ft)
 Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  4 Type(s) of Soil
 Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
 Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
  No.  (pcf)    (pcf)     (psf)     (deg)    Param.   (psf)     No.
   1   115.0    125.0       0.0     30.0    0.00       0.0      1
   2   115.0    135.0      50.0     30.0    0.00       0.0      0
   3   116.0    125.0     100.0     33.0    0.00       0.0      0
   4   132.0    135.0    1000.0     37.0    0.00       0.0      1
ANISOTROPIC STRENGTH PARAMETERS
     1 soil type(s)
 Soil Type  4 Is Anisotropic
 Number Of Direction Ranges Specified =  4
 Direction    Counterclockwise     Cohesion     Friction
   Range       Direction Limit    Intercept       Angle
    No.             (deg)           (psf)        (deg)
     1              -90.0          1000.00         37.00
     2               -5.0          1000.00         37.00
     3                5.0           900.00         37.00
     4               90.0          1000.00         37.00
 ANISOTROPIC SOIL NOTES:
    (1) An input value of 0.01 for C and/or Phi will cause Aniso
        C and/or Phi to be ignored in that range.
    (2) An input value of 0.02 for Phi will set both Phi and
        C equal to zero, with no water weight in the tension crack.
    (3) An input value of 0.03 for Phi will set both Phi and
        C equal to zero, with water weight in the tension crack.
 1 PIEZOMETRIC SURFACE(S) SPECIFIED
 Unit Weight of Water =  62.40 (pcf)
 Piezometric Surface No.  1 Specified by  2 Coordinate Points
 Pore Pressure Inclination Factor =  0.50
   Point      X-Water     Y-Water
    No.         (ft)        (ft)
     1         60.00      100.00
     2        270.00      100.00
BOUNDARY LOAD(S)
     3 Load(s) Specified
 Load        X-Left      X-Right     Intensity     Deflection
  No.         (ft)         (ft)        (psf)          (deg)
   1        150.00       152.00       2000.0          0.0
   2        152.10       242.90        300.0          0.0
   3        243.00       245.00       2000.0          0.0
 NOTE - Intensity Is Specified As A Uniformly Distributed
        Force Acting On A Horizontally Projected Surface.
 A Critical Failure Surface Searching Method, Using A Random
 Technique For Generating Circular Surfaces, Has Been Specified.
 4999 Trial Surfaces Have Been Generated.
 4999 Surface(s) Initiate(s) From Each Of     1 Points Equally Spaced
 Along The Ground Surface Between  X =  88.00(ft)
                             and  X =  90.00(ft)
 Each Surface Terminates Between   X = 122.50(ft)
                            and   X = 130.00(ft)
 Unless Further Limitations Were Imposed, The Minimum Elevation
 At Which A Surface Extends Is  Y =      0.00(ft)
  4.50(ft) Line Segments Define Each Trial Failure Surface.
 Restrictions Have Been Imposed Upon The Angle Of Initiation.
 The Angle Has Been Restricted Between The Angles Of -10.0
 And   6.0 deg.
 Following Are Displayed The Ten Most Critical Of The Trial
       Failure Surfaces Evaluated. They Are
       Ordered - Most Critical First.
       * * Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2) * *
```

```
          Selected ki function = Bi-linear
          Selected Lambda Coefficient =  1.00
          Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
          (if applicable) have been applied to the slice base(s)
          on which they intersect.
          Specified Tension Crack Water Force Factor =   0.000
          Total Number of Trial Surfaces Attempted =  4999
          Number of Trial Surfaces With Valid FS = 4999
          Statistical Data On All Valid FS Values:
             FS Max =   2.006   FS Min =   1.501   FS Ave =   1.814
             Standard Deviation =    0.126   Coefficient of Variation =    6.97 %
                     ((Modified Bishop FS for Critical Surface =  1.504))
          Failure Surface Specified By 11 Coordinate Points
            Point      X-Surf      Y-Surf
             No.        (ft)        (ft)
              1         88.000      153.689
              2         92.477      154.141
              3         96.877      155.086
              4        101.145      156.512
              5        105.228      158.403
              6        109.077      160.734
              7        112.644      163.478
              8        115.885      166.600
              9        118.760      170.062
             10        121.233      173.821
             11        123.060      177.408
          Circle Center At X =    86.178 ; Y =   194.194 ; and Radius =    40.546
          ***  FOS =     1.501   Theta (ki=1.0) =    28.67  ***
                                Lambda =   0.547
               Individual data on the    24  slices
                        Water  Water     Tie     Tie     Earthquake
                        Force  Force    Force   Force       Force   Surcharge
 Slice  Width   Weight   Top    Bot     Norm     Tan     Hor     Ver    Load
  No.    (ft)    (lbs)  (lbs)  (lbs)    (lbs)   (lbs)   (lbs)   (lbs)   (lbs)
   1      0.1       0.3     0.0      0.0        0.       0.      0.0      0.0       0.0
   2      1.3     228.6     0.0      0.0        0.       0.      0.0      0.0       0.0
   3      1.3     666.1     0.0      0.0        0.       0.      0.0      0.0       0.0
   4      0.8     659.0     0.0      0.0        0.       0.      0.0      0.0       0.0
   5      1.0     949.9     0.0      0.0        0.       0.      0.0      0.0       0.0
   6      2.2    2610.3     0.0      0.0        0.       0.      0.0      0.0       0.0
   7      2.2    2900.3     0.0      0.0        0.       0.      0.0      0.0       0.0
   8      1.1    1488.8     0.0      0.0        0.       0.      0.0      0.0       0.0
   9      3.2    4911.3     0.0      0.0        0.       0.      0.0      0.0       0.0
  10      0.2     252.3     0.0      0.0        0.       0.      0.0      0.0       0.0
  11      2.3    3776.9     0.0      0.0        0.       0.      0.0      0.0       0.0
  12      1.6    2615.1     0.0      0.0        0.       0.      0.0      0.0       0.0
  13      0.7    1136.9     0.0      0.0        0.       0.      0.0      0.0       0.0
  14      2.4    3763.5     0.0      0.0        0.       0.      0.0      0.0       0.0
  15      0.8    1216.0     0.0      0.0        0.       0.      0.0      0.0       0.0
  16      0.9    1363.9     0.0      0.0        0.       0.      0.0      0.0       0.0
  17      2.7    3999.4     0.0      0.0        0.       0.      0.0      0.0       0.0
  18      1.9    2585.2     0.0      0.0        0.       0.      0.0      0.0       0.0
  19      1.3    1560.5     0.0      0.0        0.       0.      0.0      0.0       0.0
  20      2.9    2726.2     0.0      0.0        0.       0.      0.0      0.0       0.0
  21      1.3     878.8     0.0      0.0        0.       0.      0.0      0.0       0.0
  22      1.1     520.4     0.0      0.0        0.       0.      0.0      0.0       0.0
  23      0.7     212.3     0.0      0.0        0.       0.      0.0      0.0       0.0
  24      1.1     126.7     0.0      0.0        0.       0.      0.0      0.0       0.0
          Failure Surface Specified By 11 Coordinate Points
            Point      X-Surf      Y-Surf
             No.        (ft)        (ft)
              1         88.000      153.689
              2         92.477      154.141
              3         96.877      155.086
              4        101.145      156.512
              5        105.228      158.403
              6        109.077      160.734
              7        112.644      163.478
              8        115.885      166.600
              9        118.760      170.062
```

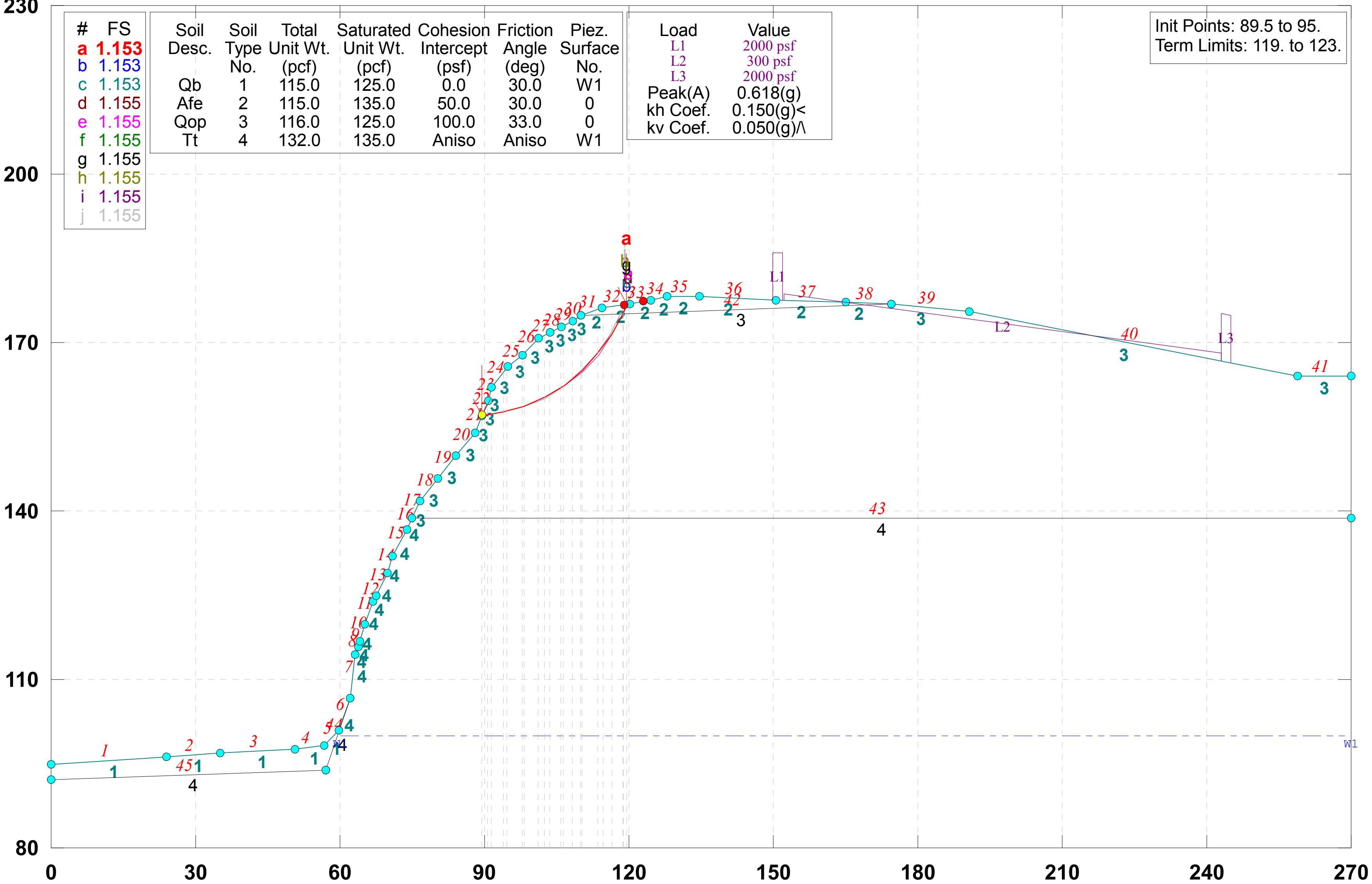
```
   10        121.233      173.821
   11        123.060      177.408
Circle Center At X =    86.178 ; Y =   194.194 ; and Radius =    40.546
***  FOS =     1.501   Theta (ki=1.0) =    28.67  ***
                    Lambda =   0.547
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         88.000      153.689
    2         92.480      154.108
    3         96.884      155.036
    4        101.153      156.459
    5        105.231      158.361
    6        109.066      160.716
    7        112.607      163.492
    8        115.808      166.655
    9        118.628      170.162
   10        121.028      173.968
   11        122.643      177.325
Circle Center At X =    86.581 ; Y =   193.036 ; and Radius =    39.372
***  FOS =     1.502   Theta (ki=1.0) =    28.72  ***
                    Lambda =   0.548
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         88.000      153.689
    2         92.480      154.108
    3         96.884      155.036
    4        101.153      156.459
    5        105.231      158.361
    6        109.066      160.716
    7        112.607      163.492
    8        115.808      166.655
    9        118.628      170.162
   10        121.028      173.968
   11        122.643      177.325
Circle Center At X =    86.581 ; Y =   193.036 ; and Radius =    39.372
***  FOS =     1.502   Theta (ki=1.0) =    28.72  ***
                    Lambda =   0.548
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         88.000      153.689
    2         92.480      154.108
    3         96.884      155.036
    4        101.153      156.459
    5        105.231      158.361
    6        109.066      160.716
    7        112.607      163.492
    8        115.808      166.655
    9        118.628      170.162
   10        121.028      173.968
   11        122.643      177.325
Circle Center At X =    86.581 ; Y =   193.036 ; and Radius =    39.372
***  FOS =     1.502   Theta (ki=1.0) =    28.72  ***
                    Lambda =   0.548
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         88.000      153.689
    2         92.478      154.132
    3         96.881      155.063
    4        101.155      156.471
    5        105.249      158.338
    6        109.114      160.643
    7        112.703      163.357
    8        115.973      166.449
    9        118.885      169.880
   10        121.404      173.609
   11        123.441      177.484
```

```
Circle Center At X =    86.209 ; Y =   194.660 ; and Radius =    41.010
***  FOS =     1.508   Theta (ki=1.0) =    28.55  ***
                    Lambda =   0.544
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         88.000      153.689
    2         92.478      154.132
    3         96.881      155.063
    4        101.155      156.471
    5        105.249      158.338
    6        109.114      160.643
    7        112.703      163.357
    8        115.973      166.449
    9        118.885      169.880
   10        121.404      173.609
   11        123.441      177.484
Circle Center At X =    86.209 ; Y =   194.660 ; and Radius =    41.010
***  FOS =     1.508   Theta (ki=1.0) =    28.55  ***
                    Lambda =   0.544
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         88.000      153.689
    2         92.480      154.116
    3         96.884      155.038
    4        101.159      156.444
    5        105.251      158.316
    6        109.109      160.632
    7        112.686      163.363
    8        115.938      166.474
    9        118.823      169.927
   10        121.307      173.679
   11        123.233      177.442
Circle Center At X =    86.414 ; Y =   194.074 ; and Radius =    40.416
***  FOS =     1.509   Theta (ki=1.0) =    28.56  ***
                    Lambda =   0.544
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         88.000      153.689
    2         92.480      154.116
    3         96.884      155.038
    4        101.159      156.444
    5        105.251      158.316
    6        109.109      160.632
    7        112.686      163.363
    8        115.938      166.474
    9        118.823      169.927
   10        121.307      173.679
   11        123.233      177.442
Circle Center At X =    86.414 ; Y =   194.074 ; and Radius =    40.416
***  FOS =     1.509   Theta (ki=1.0) =    28.56  ***
                    Lambda =   0.544
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         88.000      153.689
    2         92.480      154.116
    3         96.884      155.038
    4        101.159      156.444
    5        105.251      158.316
    6        109.109      160.632
    7        112.686      163.363
    8        115.938      166.474
    9        118.823      169.927
   10        121.307      173.679
   11        123.233      177.442
Circle Center At X =    86.414 ; Y =   194.074 ; and Radius =    40.416
***  FOS =     1.509   Theta (ki=1.0) =    28.56  ***
```

```
                    Lambda =   0.544
**** END OF GSTABL7 OUTPUT ****
```

7557 NEWMAN, 216 NEPTUNE X-X' SEISMIC

x:\shared\word perfect data\carlsbad\7500\7557 newman, 216 neptune\a4 slope stability\x-x' qop failure bishop seismic.pl2 Run By: GeoSoils, Inc. 4/14/2020 01:29PM



#	FS
a	**1.153**
b	1.153
c	1.153
d	1.155
e	1.155
f	1.155
g	1.155
h	1.155
i	1.155
j	1.155

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Qb	1	115.0	125.0	0.0	30.0	W1
Afe	2	115.0	135.0	50.0	30.0	0
Qop	3	116.0	125.0	100.0	33.0	0
Tt	4	132.0	135.0	Aniso	Aniso	W1

Load	Value
L1	2000 psf
L2	300 psf
L3	2000 psf
Peak(A)	0.618(g)
kh Coef.	0.150(g)<
kv Coef.	0.050(g)/\

Init Points: 89.5 to 95.
Term Limits: 119. to 123.

GSTABL7 v.2 FSmin=1.153
Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

W.O. 7557-A-SC
PLATE E-8

```
                             ***  GSTABL7  ***
                 ** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **
           ** Original Version 1.0, January 1996; Current Ver. 2.005.3, Feb. 2013 **
                   (All Rights Reserved-Unauthorized Use Prohibited)
     *********************************************************************************
                         SLOPE STABILITY ANALYSIS SYSTEM
            Modified Bishop, Simplified Janbu, or GLE Method of Slices.
            (Includes Spencer & Morgenstern-Price Type Analysis)
            Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
            Nonlinear Undrained Shear Strength, Curved Phi Envelope,
            Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
            Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
     *********************************************************************************
     Analysis Run Date:        4/14/2020
     Time of Run:              01:29PM
     Run By:                   GeoSoils, Inc.
     Input Data Filename:      X:\shared\Word Perfect Data\CARLSBAD\7500\7557 Newman, 216 N
 eptune\A4 Slope Stability\x-x' qop failure  bishop seismic.in
     Output Filename:          X:\shared\Word Perfect Data\CARLSBAD\7500\7557 Newman, 216 N
 eptune\A4 Slope Stability\x-x' qop failure  bishop seismic.OUT
     Unit System:              English
     Plotted Output Filename:  X:\shared\Word Perfect Data\CARLSBAD\7500\7557 Newman, 216 N
 eptune\A4 Slope Stability\x-x' qop failure  bishop seismic.PLT
     PROBLEM DESCRIPTION:  7557 NEWMAN, 216 NEPTUNE
                           X-X' SEISMIC
     BOUNDARY COORDINATES
        41 Top   Boundaries
        45 Total Boundaries
     Boundary     X-Left     Y-Left    X-Right    Y-Right    Soil Type
        No.        (ft)       (ft)       (ft)       (ft)     Below Bnd
        1          0.00      95.00      24.00      96.20        1
        2         24.00      96.20      35.15      96.90        1
        3         35.15      96.90      50.62      97.68        1
        4         50.62      97.68      56.60      98.16        1
        5         56.60      98.16      59.61     100.80        1
        6         59.61     100.80      62.10     106.50        4
        7         62.10     106.50      63.00     114.36        4
        8         63.00     114.36      63.80     115.80        4
        9         63.80     115.80      64.00     116.85        4
       10         64.00     116.85      65.10     119.80        4
       11         65.10     119.80      66.80     123.80        4
       12         66.80     123.80      67.50     124.76        4
       13         67.50     124.76      69.80     128.80        4
       14         69.80     128.80      71.00     131.93        4
       15         71.00     131.93      74.00     136.80        4
       16         74.00     136.80      74.90     138.83        4
       17         74.90     138.83      76.75     141.90        3
       18         76.75     141.90      80.49     145.76        3
       19         80.49     145.76      84.19     149.85        3
       20         84.19     149.85      88.08     153.77        3
       21         88.08     153.77      89.37     156.81        3
       22         89.37     156.81      90.67     159.81        3
       23         90.67     159.81      91.50     161.88        3
       24         91.50     161.88      94.72     165.77        3
       25         94.72     165.77      97.93     167.78        3
       26         97.93     167.78     101.30     170.64        3
       27        101.30     170.64     103.62     171.67        3
       28        103.62     171.67     105.93     172.75        3
       29        105.93     172.75     108.30     173.72        3
       30        108.30     173.72     109.95     174.84        3
       31        109.95     174.84     114.58     176.22        2
       32        114.58     176.22     120.10     176.82        2
       33        120.10     176.82     124.53     177.70        2
       34        124.53     177.70     127.85     178.12        2
       35        127.85     178.12     134.71     178.08        2
       36        134.71     178.08     150.49     177.53        2
       37        150.49     177.53     164.88     177.14        2
       38        164.88     177.14     174.53     176.85        2
       39        174.53     176.85     190.80     175.66        3
       40        190.80     175.66     259.00     164.00        3
```

W.O. 7557-A-SC
PLATE E-9

```
    41        259.00     164.00     270.00     164.00        3
    42        109.95     174.84     174.53     176.85        3
    43         74.90     138.83     270.00     138.83        4
    44         57.00      94.00      62.10     106.50        4
    45          0.00      92.00      57.00      94.00        4
 User Specified Y-Origin =       80.00(ft)
 Default X-Plus Value = 0.00(ft)
 Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  4 Type(s) of Soil
 Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
 Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
  No.  (pcf)    (pcf)     (psf)     (deg)    Param.   (psf)     No.
   1   115.0    125.0       0.0     30.0    0.00       0.0      1
   2   115.0    135.0      50.0     30.0    0.00       0.0      0
   3   116.0    125.0     100.0     33.0    0.00       0.0      0
   4   132.0    135.0    1000.0     37.0    0.00       0.0      1
ANISOTROPIC STRENGTH PARAMETERS
     1 soil type(s)
 Soil Type  4 Is Anisotropic
 Number Of Direction Ranges Specified =  4
 Direction    Counterclockwise     Cohesion     Friction
   Range       Direction Limit    Intercept       Angle
    No.            (deg)            (psf)         (deg)
     1             -90.0           1000.00         37.00
     2              -5.0           1000.00         37.00
     3               5.0            900.00         37.00
     4              90.0           1000.00         37.00
 ANISOTROPIC SOIL NOTES:
    (1) An input value of 0.01 for C and/or Phi will cause Aniso
        C and/or Phi to be ignored in that range.
    (2) An input value of 0.02 for Phi will set both Phi and
        C equal to zero, with no water weight in the tension crack.
    (3) An input value of 0.03 for Phi will set both Phi and
        C equal to zero, with water weight in the tension crack.
 1 PIEZOMETRIC SURFACE(S) SPECIFIED
 Unit Weight of Water =  62.40 (pcf)
 Piezometric Surface No.  1 Specified by  2 Coordinate Points
 Pore Pressure Inclination Factor =  0.50
   Point      X-Water     Y-Water
    No.         (ft)        (ft)
     1         60.00      100.00
     2        270.00      100.00
BOUNDARY LOAD(S)
     3 Load(s) Specified
 Load        X-Left      X-Right     Intensity     Deflection
  No.         (ft)         (ft)        (psf)          (deg)
   1        150.00       152.00       2000.0          0.0
   2        152.10       242.90        300.0          0.0
   3        243.00       245.00       2000.0          0.0
 NOTE - Intensity Is Specified As A Uniformly Distributed
        Force Acting On A Horizontally Projected Surface.
 Specified Peak Ground Acceleration Coefficient (A) =   0.618(g)
 Specified Horizontal Earthquake Coefficient (kh) =   0.150(g)
 Specified Vertical Earthquake Coefficient (kv) =   0.050(g)
 Specified Seismic Pore-Pressure Factor =   0.000
 A Critical Failure Surface Searching Method, Using A Random
 Technique For Generating Circular Surfaces, Has Been Specified.
 4999 Trial Surfaces Have Been Generated.
 4999 Surface(s) Initiate(s) From Each Of     1 Points Equally Spaced
 Along The Ground Surface Between  X =  89.50(ft)
                                 and  X =  95.00(ft)
 Each Surface Terminates Between   X = 119.00(ft)
                                and   X = 123.00(ft)
 Unless Further Limitations Were Imposed, The Minimum Elevation
 At Which A Surface Extends Is  Y =      0.00(ft)
  4.50(ft) Line Segments Define Each Trial Failure Surface.
 Restrictions Have Been Imposed Upon The Angle Of Initiation.
 The Angle Has Been Restricted Between The Angles Of -10.0
 And   6.0 deg.
```

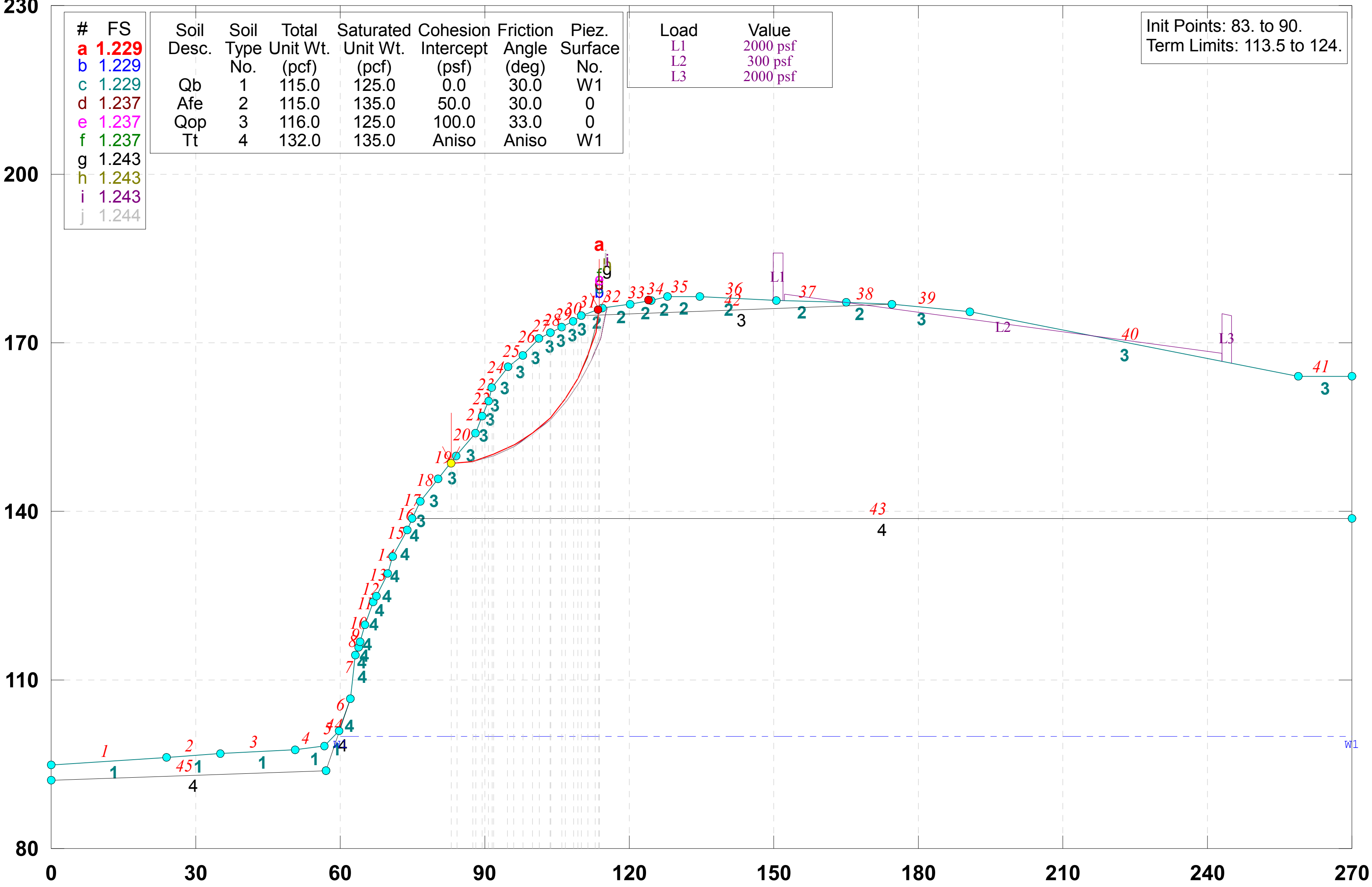
```
   Following Are Displayed The Ten Most Critical Of The Trial
         Failure Surfaces Evaluated. They Are
         Ordered - Most Critical First.
         * * Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2) * *
         Selected ki function = Bi-linear
         Selected Lambda Coefficient =  1.00
         Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
         (if applicable) have been applied to the slice base(s)
         on which they intersect.
         Specified Tension Crack Water Force Factor =   0.000
         Total Number of Trial Surfaces Attempted =  4999
         Number of Trial Surfaces With Valid FS = 4999
         Statistical Data On All Valid FS Values:
            FS Max =   1.409   FS Min =   1.153   FS Ave =   1.315
            Standard Deviation =    0.072   Coefficient of Variation =    5.44 %
                    ((Modified Bishop FS for Critical Surface =  1.205))
         Failure Surface Specified By 10 Coordinate Points
           Point      X-Surf      Y-Surf
            No.        (ft)        (ft)
             1         89.500      157.110
             2         93.977      157.569
             3         98.357      158.600
             4        102.568      160.186
             5        106.540      162.301
             6        110.208      164.909
             7        113.509      167.967
             8        116.390      171.424
             9        118.802      175.223
            10        119.518      176.757
         Circle Center At X =    88.251 ; Y =   191.774 ; and Radius =    34.686
         ***  FOS =     1.153   Theta (ki=1.0) =    27.07  ***
                              Lambda =   0.511
              Individual data on the    20  slices
                       Water  Water     Tie     Tie     Earthquake
                       Force  Force    Force   Force       Force   Surcharge
Slice  Width   Weight   Top    Bot     Norm     Tan     Hor     Ver    Load
 No.    (ft)    (lbs)  (lbs)  (lbs)    (lbs)   (lbs)   (lbs)   (lbs)   (lbs)
  1      1.2     175.1    0.0     0.0       0.      0.    26.3     8.8      0.0
  2      0.8     344.0    0.0     0.0       0.      0.    51.6    17.2      0.0
  3      2.5    1704.6    0.0     0.0       0.      0.   255.7    85.2      0.0
  4      0.7     661.0    0.0     0.0       0.      0.    99.1    33.0      0.0
  5      3.2    3222.0    0.0     0.0       0.      0.   483.3   161.1      0.0
  6      0.4     465.9    0.0     0.0       0.      0.    69.9    23.3      0.0
  7      2.9    3494.9    0.0     0.0       0.      0.   524.2   174.7      0.0
  8      1.3    1614.1    0.0     0.0       0.      0.   242.1    80.7      0.0
  9      1.1    1338.7    0.0     0.0       0.      0.   200.8    66.9      0.0
 10      2.3    2907.0    0.0     0.0       0.      0.   436.1   145.4      0.0
 11      0.6     760.0    0.0     0.0       0.      0.   114.0    38.0      0.0
 12      1.8    2129.7    0.0     0.0       0.      0.   319.5   106.5      0.0
 13      1.7    1941.0    0.0     0.0       0.      0.   291.1    97.0      0.0
 14      0.3     300.6    0.0     0.0       0.      0.    45.1    15.0      0.0
 15      3.3    3434.0    0.0     0.0       0.      0.   515.1   171.7      0.0
 16      1.1     924.5    0.0     0.0       0.      0.   138.7    46.2      0.0
 17      1.8    1253.2    0.0     0.0       0.      0.   188.0    62.7      0.0
 18      2.3     886.7    0.0     0.0       0.      0.   133.0    44.3      0.0
 19      0.1      12.0    0.0     0.0       0.      0.     1.8     0.6      0.0
 20      0.7      60.0    0.0     0.0       0.      0.     9.0     3.0      0.0
         Failure Surface Specified By 10 Coordinate Points
           Point      X-Surf      Y-Surf
            No.        (ft)        (ft)
             1         89.500      157.110
             2         93.977      157.569
             3         98.357      158.600
             4        102.568      160.186
             5        106.540      162.301
             6        110.208      164.909
             7        113.509      167.967
             8        116.390      171.424
             9        118.802      175.223
            10        119.518      176.757
```

```
Circle Center At X =    88.251 ; Y =   191.774 ; and Radius =    34.686
***  FOS =     1.153   Theta (ki=1.0) =    27.07  ***
                    Lambda =   0.511
Failure Surface Specified By 10 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         89.500      157.110
    2         93.977      157.569
    3         98.357      158.600
    4        102.568      160.186
    5        106.540      162.301
    6        110.208      164.909
    7        113.509      167.967
    8        116.390      171.424
    9        118.802      175.223
   10        119.518      176.757
Circle Center At X =    88.251 ; Y =   191.774 ; and Radius =    34.686
***  FOS =     1.153   Theta (ki=1.0) =    27.07  ***
                    Lambda =   0.511
Failure Surface Specified By 10 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         89.500      157.110
    2         93.976      157.577
    3         98.356      158.606
    4        102.572      160.180
    5        106.555      162.275
    6        110.241      164.856
    7        113.572      167.882
    8        116.494      171.304
    9        118.959      175.068
   10        119.796      176.787
Circle Center At X =    88.127 ; Y =   192.386 ; and Radius =    35.303
***  FOS =     1.155   Theta (ki=1.0) =    26.99  ***
                    Lambda =   0.509
Failure Surface Specified By 10 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         89.500      157.110
    2         93.976      157.577
    3         98.356      158.606
    4        102.572      160.180
    5        106.555      162.275
    6        110.241      164.856
    7        113.572      167.882
    8        116.494      171.304
    9        118.959      175.068
   10        119.796      176.787
Circle Center At X =    88.127 ; Y =   192.386 ; and Radius =    35.303
***  FOS =     1.155   Theta (ki=1.0) =    26.99  ***
                    Lambda =   0.509
Failure Surface Specified By 10 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         89.500      157.110
    2         93.978      157.553
    3         98.361      158.575
    4        102.573      160.159
    5        106.543      162.277
    6        110.203      164.895
    7        113.492      167.966
    8        116.353      171.439
    9        118.738      175.256
   10        119.417      176.746
Circle Center At X =    88.418 ; Y =   191.367 ; and Radius =    34.274
***  FOS =     1.155   Theta (ki=1.0) =    27.07  ***
                    Lambda =   0.511
Failure Surface Specified By 10 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
```

```
    1         89.500      157.110
    2         93.978      157.553
    3         98.361      158.575
    4        102.573      160.159
    5        106.543      162.277
    6        110.203      164.895
    7        113.492      167.966
    8        116.353      171.439
    9        118.738      175.256
   10        119.417      176.746
Circle Center At X =    88.418 ; Y =   191.367 ; and Radius =    34.274
***  FOS =     1.155   Theta (ki=1.0) =    27.07  ***
                   Lambda =   0.511
Failure Surface Specified By 10 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         89.500      157.110
    2         93.980      157.529
    3         98.364      158.545
    4        102.572      160.140
    5        106.528      162.285
    6        110.160      164.941
    7        113.404      168.061
    8        116.199      171.587
    9        118.496      175.456
   10        119.025      176.703
Circle Center At X =    88.693 ; Y =   190.369 ; and Radius =    33.269
***  FOS =     1.155   Theta (ki=1.0) =    27.17  ***
                   Lambda =   0.513
Failure Surface Specified By 10 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         89.500      157.110
    2         93.980      157.529
    3         98.364      158.545
    4        102.572      160.140
    5        106.528      162.285
    6        110.160      164.941
    7        113.404      168.061
    8        116.199      171.587
    9        118.496      175.456
   10        119.025      176.703
Circle Center At X =    88.693 ; Y =   190.369 ; and Radius =    33.269
***  FOS =     1.155   Theta (ki=1.0) =    27.17  ***
                   Lambda =   0.513
Failure Surface Specified By 10 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         89.500      157.110
    2         93.980      157.529
    3         98.364      158.545
    4        102.572      160.140
    5        106.528      162.285
    6        110.160      164.941
    7        113.404      168.061
    8        116.199      171.587
    9        118.496      175.456
   10        119.025      176.703
Circle Center At X =    88.693 ; Y =   190.369 ; and Radius =    33.269
***  FOS =     1.155   Theta (ki=1.0) =    27.17  ***
                   Lambda =   0.513
          **** END OF GSTABL7 OUTPUT ****
```

7557 NEWMAN, 216 NEPTUNE X-X' Qop Failure Static

x:\shared\word perfect data\carlsbad\7500\7557 newman, 216 neptune\a4 slope stability\x-x' qop failure bishop spencer-1.2.pl2 Run By: GeoSoils, Inc. 4/7/2020 11:57AM



#	FS
a	1.229
b	1.229
c	1.229
d	1.237
e	1.237
f	1.237
g	1.243
h	1.243
i	1.243
j	1.244

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
Qb	1	115.0	125.0	0.0	30.0	W1
Afe	2	115.0	135.0	50.0	30.0	0
Qop	3	116.0	125.0	100.0	33.0	0
Tt	4	132.0	135.0	Aniso	Aniso	W1

Load	Value
L1	2000 psf
L2	300 psf
L3	2000 psf

Init Points: 83. to 90.
Term Limits: 113.5 to 124.

GSTABL7 v.2 FSmin=1.229
Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2)

W.O. 7557-A-SC
PLATE E-14

```
                              ***  GSTABL7  ***
                 ** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **
          ** Original Version 1.0, January 1996; Current Ver. 2.005.3, Feb. 2013 **
                   (All Rights Reserved-Unauthorized Use Prohibited)
    *********************************************************************************
                        SLOPE STABILITY ANALYSIS SYSTEM
           Modified Bishop, Simplified Janbu, or GLE Method of Slices.
           (Includes Spencer & Morgenstern-Price Type Analysis)
           Including Pier/Pile, Reinforcement, Soil Nail, Tieback,
           Nonlinear Undrained Shear Strength, Curved Phi Envelope,
           Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water
           Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.
    *********************************************************************************
    Analysis Run Date:        4/7/2020
    Time of Run:              11:57AM
    Run By:                   GeoSoils, Inc.
    Input Data Filename:      X:\shared\Word Perfect Data\CARLSBAD\7500\7557 Newman, 216 N
eptune\A4 Slope Stability\x-x' qop failure  bishop spencer-1.2.in
    Output Filename:          X:\shared\Word Perfect Data\CARLSBAD\7500\7557 Newman, 216 N
eptune\A4 Slope Stability\x-x' qop failure  bishop spencer-1.2.OUT
    Unit System:              English
    Plotted Output Filename:  X:\shared\Word Perfect Data\CARLSBAD\7500\7557 Newman, 216 N
eptune\A4 Slope Stability\x-x' qop failure  bishop spencer-1.2.PLT
    PROBLEM DESCRIPTION:  7557 NEWMAN, 216 NEPTUNE
                        X-X' Qop Failure Static
    BOUNDARY COORDINATES
       41 Top   Boundaries
       45 Total Boundaries
    Boundary     X-Left     Y-Left    X-Right    Y-Right    Soil Type
       No.        (ft)       (ft)       (ft)       (ft)     Below Bnd
        1          0.00      95.00      24.00      96.20        1
        2         24.00      96.20      35.15      96.90        1
        3         35.15      96.90      50.62      97.68        1
        4         50.62      97.68      56.60      98.16        1
        5         56.60      98.16      59.61     100.80        1
        6         59.61     100.80      62.10     106.50        4
        7         62.10     106.50      63.00     114.36        4
        8         63.00     114.36      63.80     115.80        4
        9         63.80     115.80      64.00     116.85        4
       10         64.00     116.85      65.10     119.80        4
       11         65.10     119.80      66.80     123.80        4
       12         66.80     123.80      67.50     124.76        4
       13         67.50     124.76      69.80     128.80        4
       14         69.80     128.80      71.00     131.93        4
       15         71.00     131.93      74.00     136.80        4
       16         74.00     136.80      74.90     138.83        4
       17         74.90     138.83      76.75     141.90        3
       18         76.75     141.90      80.49     145.76        3
       19         80.49     145.76      84.19     149.85        3
       20         84.19     149.85      88.08     153.77        3
       21         88.08     153.77      89.37     156.81        3
       22         89.37     156.81      90.67     159.81        3
       23         90.67     159.81      91.50     161.88        3
       24         91.50     161.88      94.72     165.77        3
       25         94.72     165.77      97.93     167.78        3
       26         97.93     167.78     101.30     170.64        3
       27        101.30     170.64     103.62     171.67        3
       28        103.62     171.67     105.93     172.75        3
       29        105.93     172.75     108.30     173.72        3
       30        108.30     173.72     109.95     174.84        3
       31        109.95     174.84     114.58     176.22        2
       32        114.58     176.22     120.10     176.82        2
       33        120.10     176.82     124.53     177.70        2
       34        124.53     177.70     127.85     178.12        2
       35        127.85     178.12     134.71     178.08        2
       36        134.71     178.08     150.49     177.53        2
       37        150.49     177.53     164.88     177.14        2
       38        164.88     177.14     174.53     176.85        2
       39        174.53     176.85     190.80     175.66        3
       40        190.80     175.66     259.00     164.00        3
```

```
    41        259.00     164.00     270.00     164.00        3
    42        109.95     174.84     174.53     176.85        3
    43         74.90     138.83     270.00     138.83        4
    44         57.00      94.00      62.10     106.50        4
    45          0.00      92.00      57.00      94.00        4
 User Specified Y-Origin =       80.00(ft)
 Default X-Plus Value = 0.00(ft)
 Default Y-Plus Value = 0.00(ft)
ISOTROPIC SOIL PARAMETERS
  4 Type(s) of Soil
 Soil  Total  Saturated  Cohesion Friction   Pore   Pressure   Piez.
 Type Unit Wt. Unit Wt. Intercept   Angle  Pressure Constant Surface
  No.  (pcf)    (pcf)     (psf)     (deg)    Param.   (psf)     No.
   1   115.0    125.0       0.0     30.0    0.00       0.0      1
   2   115.0    135.0      50.0     30.0    0.00       0.0      0
   3   116.0    125.0     100.0     33.0    0.00       0.0      0
   4   132.0    135.0    1000.0     37.0    0.00       0.0      1
ANISOTROPIC STRENGTH PARAMETERS
     1 soil type(s)
 Soil Type  4 Is Anisotropic
 Number Of Direction Ranges Specified =  4
 Direction    Counterclockwise     Cohesion     Friction
   Range       Direction Limit    Intercept       Angle
    No.            (deg)            (psf)         (deg)
     1             -90.0           1000.00         37.00
     2              -5.0           1000.00         37.00
     3               5.0            900.00         37.00
     4              90.0           1000.00         37.00
 ANISOTROPIC SOIL NOTES:
    (1) An input value of 0.01 for C and/or Phi will cause Aniso
        C and/or Phi to be ignored in that range.
    (2) An input value of 0.02 for Phi will set both Phi and
        C equal to zero, with no water weight in the tension crack.
    (3) An input value of 0.03 for Phi will set both Phi and
        C equal to zero, with water weight in the tension crack.
 1 PIEZOMETRIC SURFACE(S) SPECIFIED
 Unit Weight of Water =  62.40 (pcf)
 Piezometric Surface No.  1 Specified by  2 Coordinate Points
 Pore Pressure Inclination Factor =  0.50
   Point      X-Water     Y-Water
    No.         (ft)        (ft)
     1         60.00      100.00
     2        270.00      100.00
BOUNDARY LOAD(S)
     3 Load(s) Specified
 Load        X-Left      X-Right     Intensity     Deflection
  No.         (ft)         (ft)        (psf)          (deg)
   1        150.00       152.00       2000.0          0.0
   2        152.10       242.90        300.0          0.0
   3        243.00       245.00       2000.0          0.0
 NOTE - Intensity Is Specified As A Uniformly Distributed
        Force Acting On A Horizontally Projected Surface.
 A Critical Failure Surface Searching Method, Using A Random
 Technique For Generating Circular Surfaces, Has Been Specified.
 4999 Trial Surfaces Have Been Generated.
 4999 Surface(s) Initiate(s) From Each Of     1 Points Equally Spaced
 Along The Ground Surface Between  X =  83.00(ft)
                             and  X =  90.00(ft)
 Each Surface Terminates Between   X = 113.50(ft)
                            and   X = 124.00(ft)
 Unless Further Limitations Were Imposed, The Minimum Elevation
 At Which A Surface Extends Is  Y =      0.00(ft)
  4.50(ft) Line Segments Define Each Trial Failure Surface.
 Restrictions Have Been Imposed Upon The Angle Of Initiation.
 The Angle Has Been Restricted Between The Angles Of -10.0
 And   6.0 deg.
 Following Are Displayed The Ten Most Critical Of The Trial
       Failure Surfaces Evaluated. They Are
       Ordered - Most Critical First.
       * * Safety Factors Are Calculated By GLE (Spencer`s) Method (0-2) * *
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          Selected ki function = Bi-linear
          Selected Lambda Coefficient =  1.00
          Forces from Reinforcement, Piers/Piles, Soil Nails, and Applied Forces
          (if applicable) have been applied to the slice base(s)
          on which they intersect.
          Specified Tension Crack Water Force Factor =   0.000
          Total Number of Trial Surfaces Attempted =  4999
          Number of Trial Surfaces With Valid FS = 4999
          Statistical Data On All Valid FS Values:
             FS Max =   1.671   FS Min =   1.229   FS Ave =   1.515
             Standard Deviation =    0.115   Coefficient of Variation =    7.60 %
                     ((Modified Bishop FS for Critical Surface =  1.236))
          Failure Surface Specified By 11 Coordinate Points
            Point      X-Surf      Y-Surf
             No.        (ft)        (ft)
              1         83.000      148.535
              2         87.476      148.994
              3         91.845      150.074
              4         96.020      151.753
              5         99.920      153.997
              6        103.469      156.764
              7        106.598      159.999
              8        109.244      163.638
              9        111.358      167.611
             10        112.896      171.840
             11        113.773      175.980
          Circle Center At X =    81.958 ; Y =   180.699 ; and Radius =    32.182
          ***  FOS =     1.229   Theta (ki=1.0) =    33.67  ***
                              Lambda =   0.666
               Individual data on the    23  slices
                         Water  Water     Tie     Tie     Earthquake
                         Force  Force    Force   Force       Force   Surcharge
 Slice  Width   Weight    Top    Bot     Norm     Tan     Hor     Ver    Load
  No.    (ft)    (lbs)   (lbs)  (lbs)    (lbs)   (lbs)   (lbs)   (lbs)   (lbs)
   1      1.2      82.4     0.0     0.0       0.      0.     0.0     0.0      0.0
   2      3.3    1021.8     0.0     0.0       0.      0.     0.0     0.0      0.0
   3      0.6     307.8     0.0     0.0       0.      0.     0.0     0.0      0.0
   4      1.3     895.9     0.0     0.0       0.      0.     0.0     0.0      0.0
   5      1.3    1310.0     0.0     0.0       0.      0.     0.0     0.0      0.0
   6      0.8    1055.1     0.0     0.0       0.      0.     0.0     0.0      0.0
   7      0.3     482.5     0.0     0.0       0.      0.     0.0     0.0      0.0
   8      2.9    4462.7     0.0     0.0       0.      0.     0.0     0.0      0.0
   9      1.3    2214.9     0.0     0.0       0.      0.     0.0     0.0      0.0
  10      1.9    3296.4     0.0     0.0       0.      0.     0.0     0.0      0.0
  11      2.0    3509.5     0.0     0.0       0.      0.     0.0     0.0      0.0
  12      1.4    2483.6     0.0     0.0       0.      0.     0.0     0.0      0.0
  13      2.2    3825.9     0.0     0.0       0.      0.     0.0     0.0      0.0
  14      0.2     258.4     0.0     0.0       0.      0.     0.0     0.0      0.0
  15      2.3    3777.2     0.0     0.0       0.      0.     0.0     0.0      0.0
  16      0.7    1025.1     0.0     0.0       0.      0.     0.0     0.0      0.0
  17      1.7    2409.4     0.0     0.0       0.      0.     0.0     0.0      0.0
  18      0.9    1210.8     0.0     0.0       0.      0.     0.0     0.0      0.0
  19      0.7     842.9     0.0     0.0       0.      0.     0.0     0.0      0.0
  20      1.4    1430.3     0.0     0.0       0.      0.     0.0     0.0      0.0
  21      1.5    1027.4     0.0     0.0       0.      0.     0.0     0.0      0.0
  22      0.7     184.6     0.0     0.0       0.      0.     0.0     0.0      0.0
  23      0.2      12.1     0.0     0.0       0.      0.     0.0     0.0      0.0
          Failure Surface Specified By 11 Coordinate Points
            Point      X-Surf      Y-Surf
             No.        (ft)        (ft)
              1         83.000      148.535
              2         87.476      148.994
              3         91.845      150.074
              4         96.020      151.753
              5         99.920      153.997
              6        103.469      156.764
              7        106.598      159.999
              8        109.244      163.638
              9        111.358      167.611
             10        112.896      171.840
```

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   11        113.773      175.980
Circle Center At X =    81.958 ; Y =   180.699 ; and Radius =    32.182
***  FOS =     1.229   Theta (ki=1.0) =    33.67  ***
                    Lambda =   0.666
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         83.000      148.535
    2         87.476      148.994
    3         91.845      150.074
    4         96.020      151.753
    5         99.920      153.997
    6        103.469      156.764
    7        106.598      159.999
    8        109.244      163.638
    9        111.358      167.611
   10        112.896      171.840
   11        113.773      175.980
Circle Center At X =    81.958 ; Y =   180.699 ; and Radius =    32.182
***  FOS =     1.229   Theta (ki=1.0) =    33.67  ***
                    Lambda =   0.666
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         83.000      148.535
    2         87.480      148.953
    3         91.857      150.002
    4         96.041      151.658
    5         99.948      153.890
    6        103.501      156.651
    7        106.628      159.888
    8        109.266      163.534
    9        111.361      167.516
   10        112.873      171.755
   11        113.729      175.966
Circle Center At X =    82.293 ; Y =   180.270 ; and Radius =    31.743
***  FOS =     1.237   Theta (ki=1.0) =    33.53  ***
                    Lambda =   0.663
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         83.000      148.535
    2         87.480      148.953
    3         91.857      150.002
    4         96.041      151.658
    5         99.948      153.890
    6        103.501      156.651
    7        106.628      159.888
    8        109.266      163.534
    9        111.361      167.516
   10        112.873      171.755
   11        113.729      175.966
Circle Center At X =    82.293 ; Y =   180.270 ; and Radius =    31.743
***  FOS =     1.237   Theta (ki=1.0) =    33.53  ***
                    Lambda =   0.663
Failure Surface Specified By 11 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         83.000      148.535
    2         87.480      148.953
    3         91.857      150.002
    4         96.041      151.658
    5         99.948      153.890
    6        103.501      156.651
    7        106.628      159.888
    8        109.266      163.534
    9        111.361      167.516
   10        112.873      171.755
   11        113.729      175.966
Circle Center At X =    82.293 ; Y =   180.270 ; and Radius =    31.743
```

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***  FOS =     1.237   Theta (ki=1.0) =    33.53  ***
                    Lambda =   0.663
Failure Surface Specified By 12 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         83.000      148.535
    2         87.477      148.985
    3         91.857      150.019
    4         96.063      151.619
    5        100.023      153.756
    6        103.669      156.395
    7        106.937      159.488
    8        109.771      162.984
    9        112.122      166.820
   10        113.950      170.933
   11        115.222      175.249
   12        115.388      176.308
Circle Center At X =    81.862 ; Y =   182.633 ; and Radius =    34.118
***  FOS =     1.243   Theta (ki=1.0) =    33.44  ***
                    Lambda =   0.660
Failure Surface Specified By 12 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         83.000      148.535
    2         87.477      148.985
    3         91.857      150.019
    4         96.063      151.619
    5        100.023      153.756
    6        103.669      156.395
    7        106.937      159.488
    8        109.771      162.984
    9        112.122      166.820
   10        113.950      170.933
   11        115.222      175.249
   12        115.388      176.308
Circle Center At X =    81.862 ; Y =   182.633 ; and Radius =    34.118
***  FOS =     1.243   Theta (ki=1.0) =    33.44  ***
                    Lambda =   0.660
Failure Surface Specified By 12 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         83.000      148.535
    2         87.477      148.985
    3         91.857      150.019
    4         96.063      151.619
    5        100.023      153.756
    6        103.669      156.395
    7        106.937      159.488
    8        109.771      162.984
    9        112.122      166.820
   10        113.950      170.933
   11        115.222      175.249
   12        115.388      176.308
Circle Center At X =    81.862 ; Y =   182.633 ; and Radius =    34.118
***  FOS =     1.243   Theta (ki=1.0) =    33.44  ***
                    Lambda =   0.660
Failure Surface Specified By 12 Coordinate Points
  Point      X-Surf      Y-Surf
   No.        (ft)        (ft)
    1         83.000      148.535
    2         87.479      148.969
    3         91.860      149.996
    4         96.066      151.596
    5        100.021      153.742
    6        103.656      156.396
    7        106.905      159.509
    8        109.710      163.028
    9        112.023      166.888
   10        113.801      171.022
   11        115.014      175.355
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   12        115.143      176.281
Circle Center At X =    82.030 ; Y =   182.150 ; and Radius =    33.629
***  FOS =     1.244   Theta (ki=1.0) =    33.43  ***
                    Lambda =   0.660
          **** END OF GSTABL7 OUTPUT ****
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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

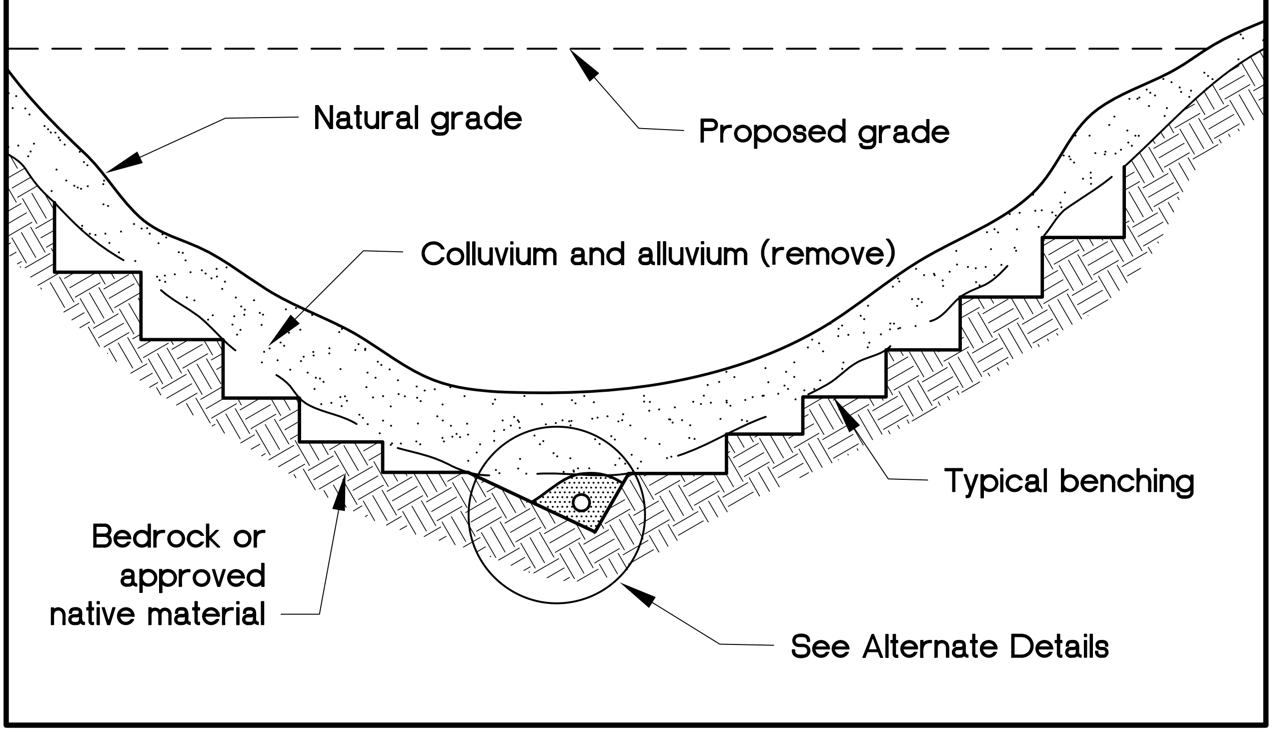
It is the contractor's responsibility to provide safe access into trenches where compaction testing is needed. Our personnel are directed not to enter any excavation or vertical cut which: 1) is 5 feet or deeper unless shored or laid back; 2) displays any evidence of instability, has any loose rock or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

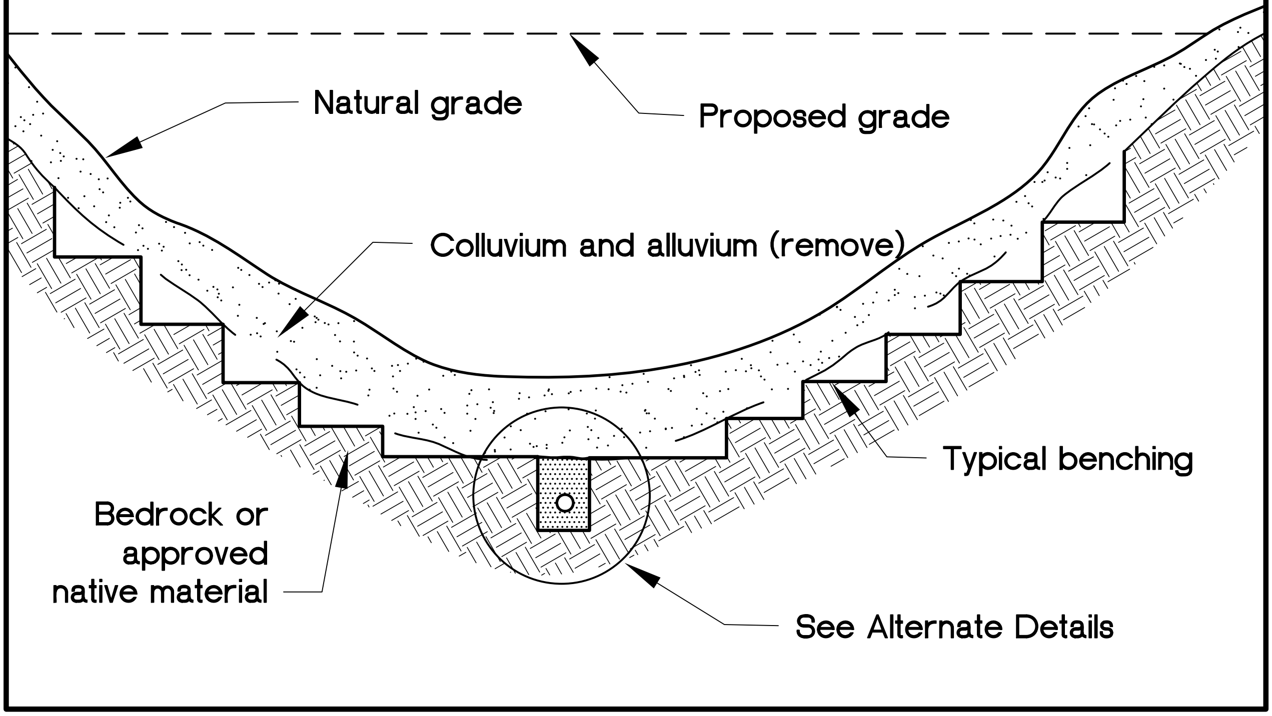
If the contractor fails to provide safe access to trenches for compaction testing, our company policy requires that the soil technician withdraw and notify his/her supervisor. The contractor’s representative will be contacted in an effort to affect a solution. All backfill not tested due to safety concerns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

TYPE A

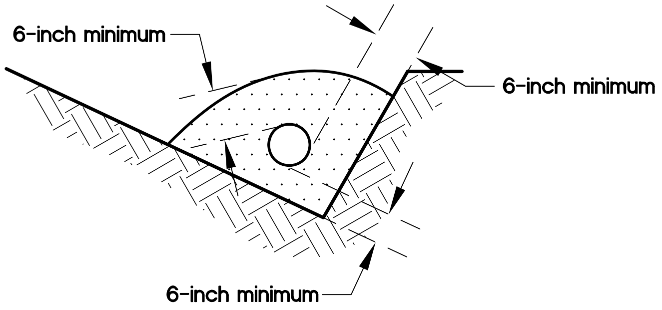


TYPE B

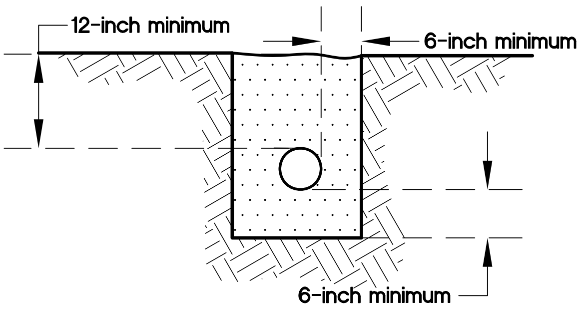


Selection of alternate subdrain details, location, and extent of subdrains should be evaluated by the geotechnical consultant during grading.

GeoSoils, Inc.	CANYON SUBDRAIN DETAIL	Plate F-1



A-1



B-1

Filter material: Minimum volume of 9 cubic feet per lineal foot of pipe.

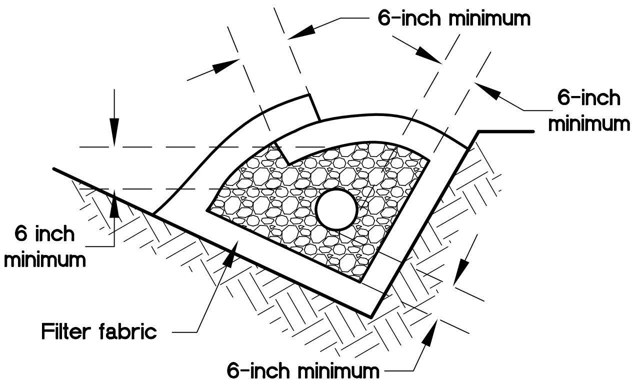
Perforated pipe: 6-inch-diameter ABS or PVC pipe or approved substitute with minimum 8 perforations (¼-inch diameter) per lineal foot in bottom half of pipe (ASTM D-2751, SDR-35, or ASTM D-1527, Schd. 40).

For continuous run in excess of 500 feet, use 8-inch-diameter pipe (ASTM D-3034, SDR-35, or ASTM D-1785, Schd. 40).

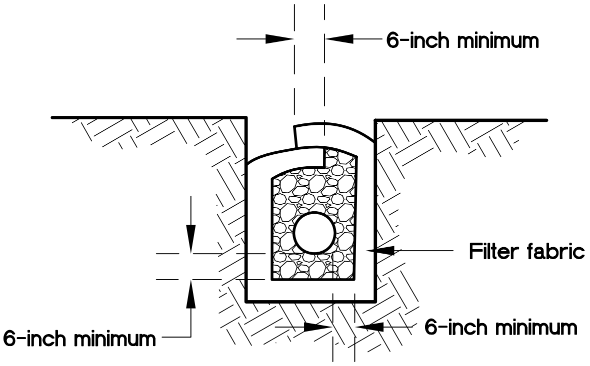
FILTER MATERIAL

Sieve Size	Percent Passing
1 inch	100
¾ inch	90-100
⅜ inch	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

ALTERNATE 1: PERFORATED PIPE AND FILTER MATERIAL



A-2

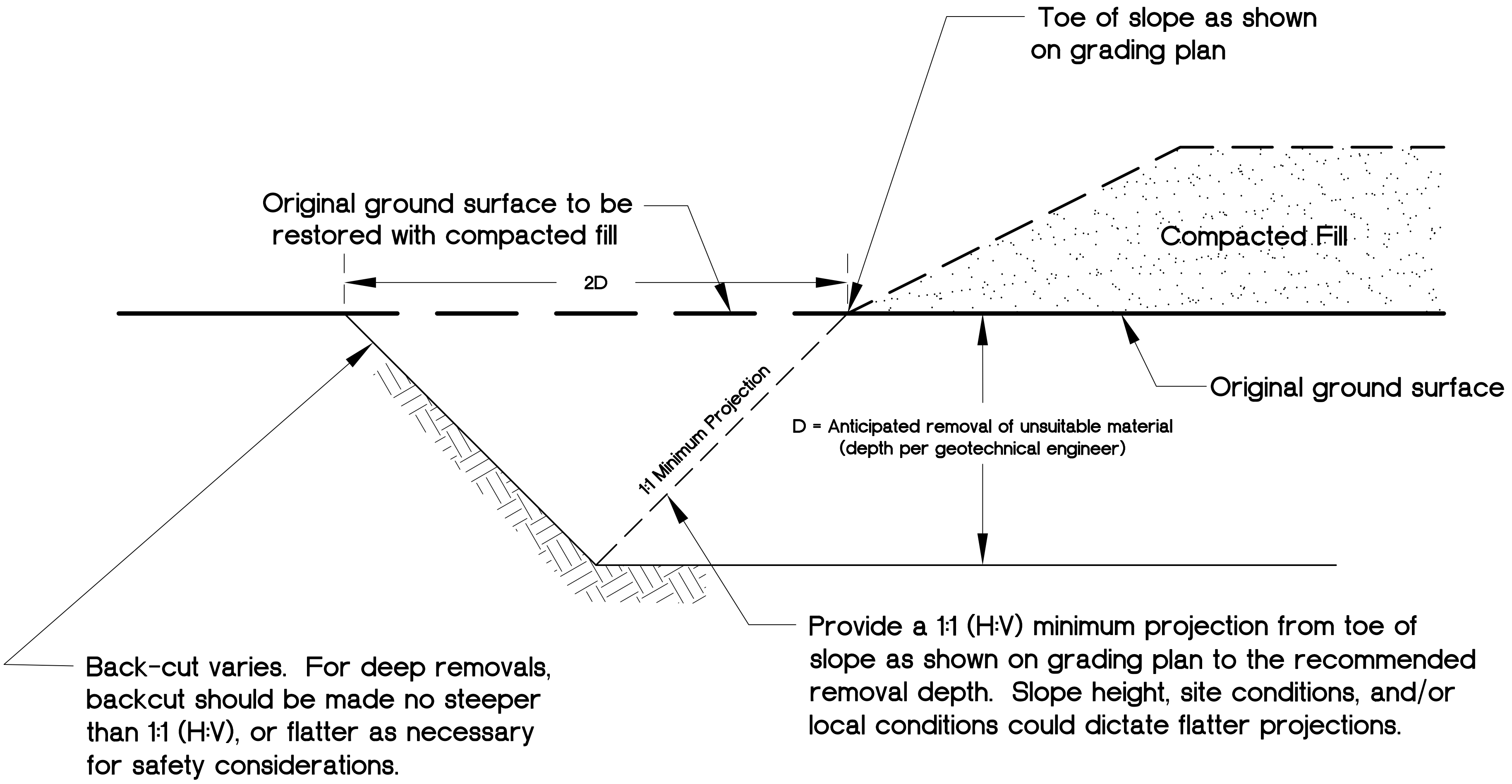


B-2

Gravel Material: 9 cubic feet per lineal foot.
Perforated Pipe: See Alternate 1
Gravel: Clean ¾-inch rock or approved substitute.
Filter Fabric: Mirafi 140 or approved substitute.

ALTERNATE 2: PERFORATED PIPE, GRAVEL, AND FILTER FABRIC

GeoSoils, Inc.	CANYON SUBDRAIN ALTERNATE DETAILS	Plate F-2



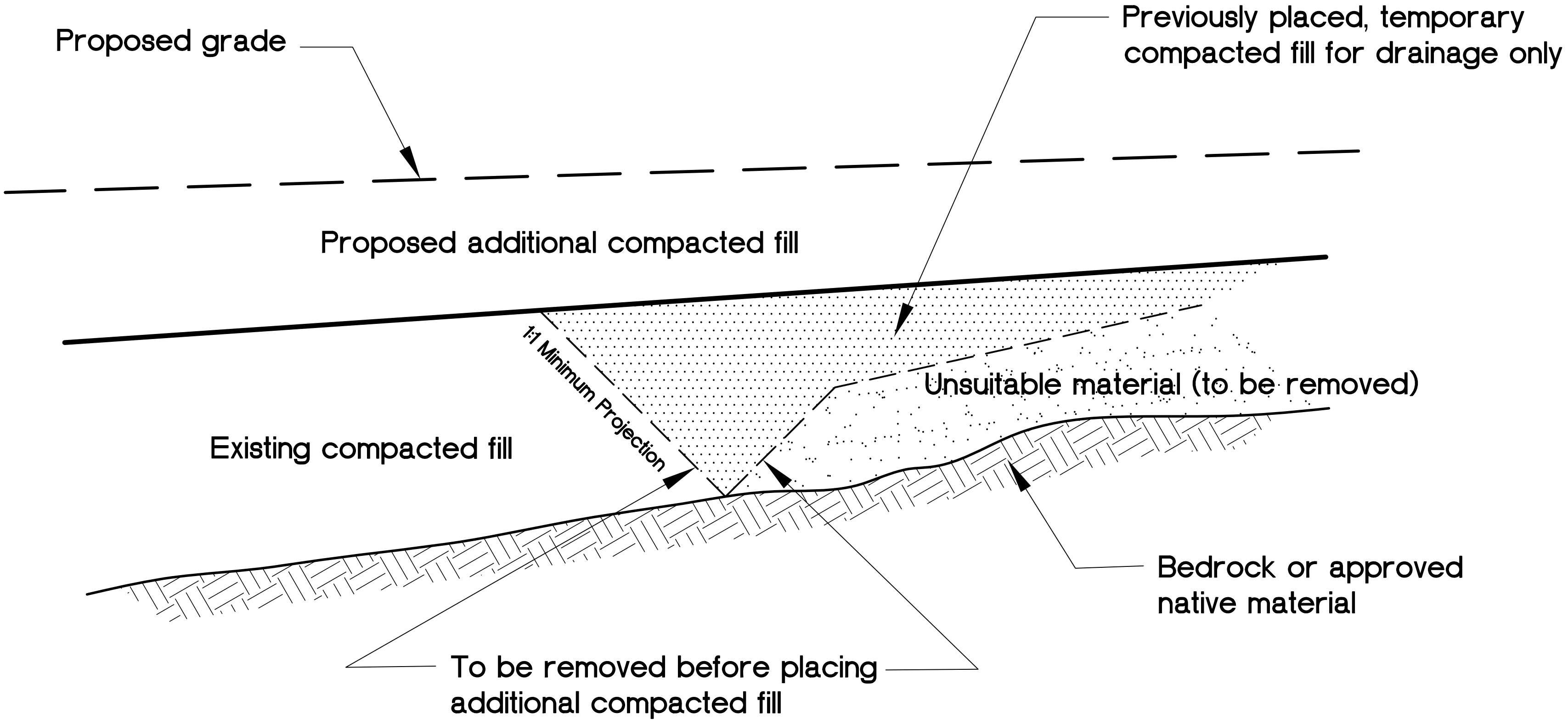
Back-cut varies. For deep removals, backcut should be made no steeper than 1:1 (H:V), or flatter as necessary for safety considerations.

Provide a 1:1 (H:V) minimum projection from toe of slope as shown on grading plan to the recommended removal depth. Slope height, site conditions, and/or local conditions could dictate flatter projections.

GeoSoils, Inc.

FILL SLOPE TOEING OUT ON FLAT ALLUVIATED CANYON DETAIL

Plate F-3



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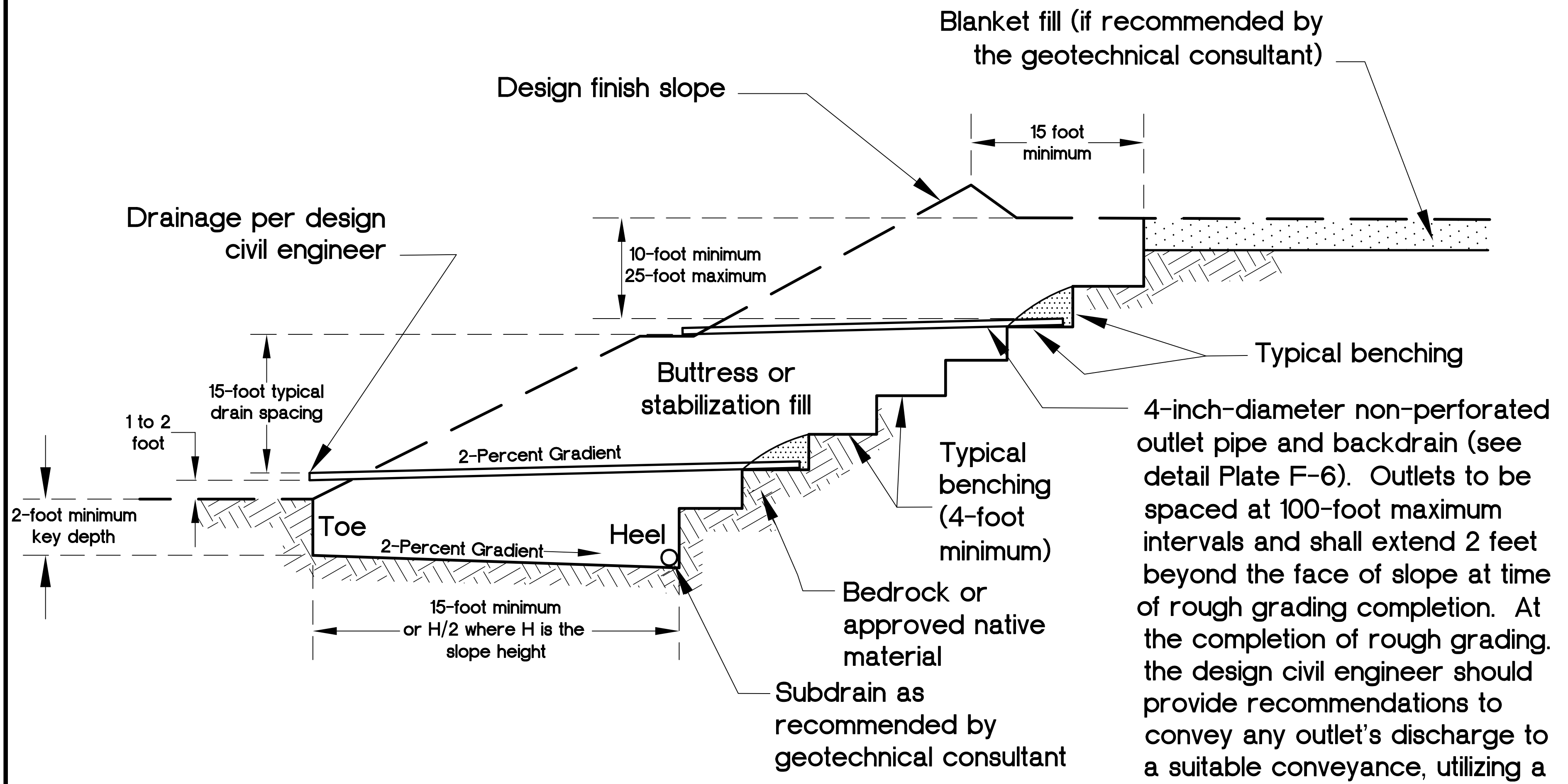
REMOVAL ADJACENT TO EXISTING FILL ADJOINING CANYON FILL DETAIL

Plate F-4

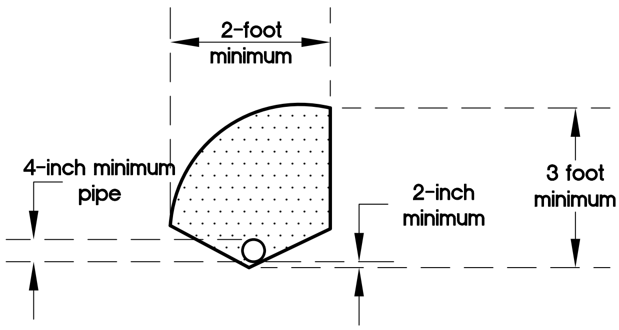


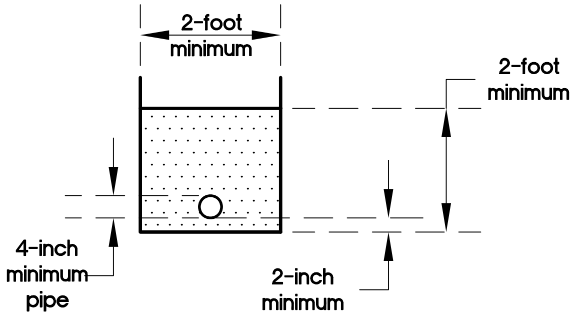
4-inch-diameter non-perforated outlet pipe and backdrain (see detail Plate F-6). Outlets to be spaced at 100-foot maximum intervals and shall extend 2 feet beyond the face of slope at time of rough grading completion. At the completion of rough grading. the design civil engineer should provide recommendations to convey any outlet's discharge to a suitable conveyance, utilizing a non-erosive device.

GeoSoils, Inc.

TYPICAL STABILIZATION / BUTTRESS FILL DETAIL

Plate F-5





Filter Material: Minimum of 5 cubic feet per lineal foot of pipe or 4 cubic feet per lineal feet of pipe when placed in square cut trench.

Alternative in Lieu of Filter Material: Gravel may be encased in approved filter fabric. Filter fabric shall be Mirafi 140 or equivalent. Filter fabric shall be lapped a minimum of 12 inches in all joints.

Minimum 4-Inch-Diameter Pipe: ABS-ASTM D-2751, SDR 35; or ASTM D-1527 Schedule 40, PVC-ASTM D-3034, SDR 35; or ASTM D-1785 Schedule 40 with a crushing strength of 1,000 pounds minimum, and a minimum of 8 uniformly-spaced perforations per foot of pipe. Must be installed with perforations down at bottom of pipe. Provide cap at upstream end of pipe. Slope at 2 percent to outlet pipe. Outlet pipe to be connected to subdrain pipe with tee or elbow.

Notes: 1. Trench for outlet pipes to be backfilled and compacted with onsite soil.

2. Backdrains and lateral drains shall be located at elevation of every bench drain. First drain located at elevation just above lower lot grade. Additional drains may be required at the discretion of the geotechnical consultant.

Filter Material shall be of the following specification or an approved equivalent.

Sieve Size	Percent Passing
1 inch	100
¾ inch	90-100
⅜ inch	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

Gravel shall be of the following specification or an approved equivalent.

Sieve Size	Percent Passing
1½ inch	100
No. 4	50
No. 200	8

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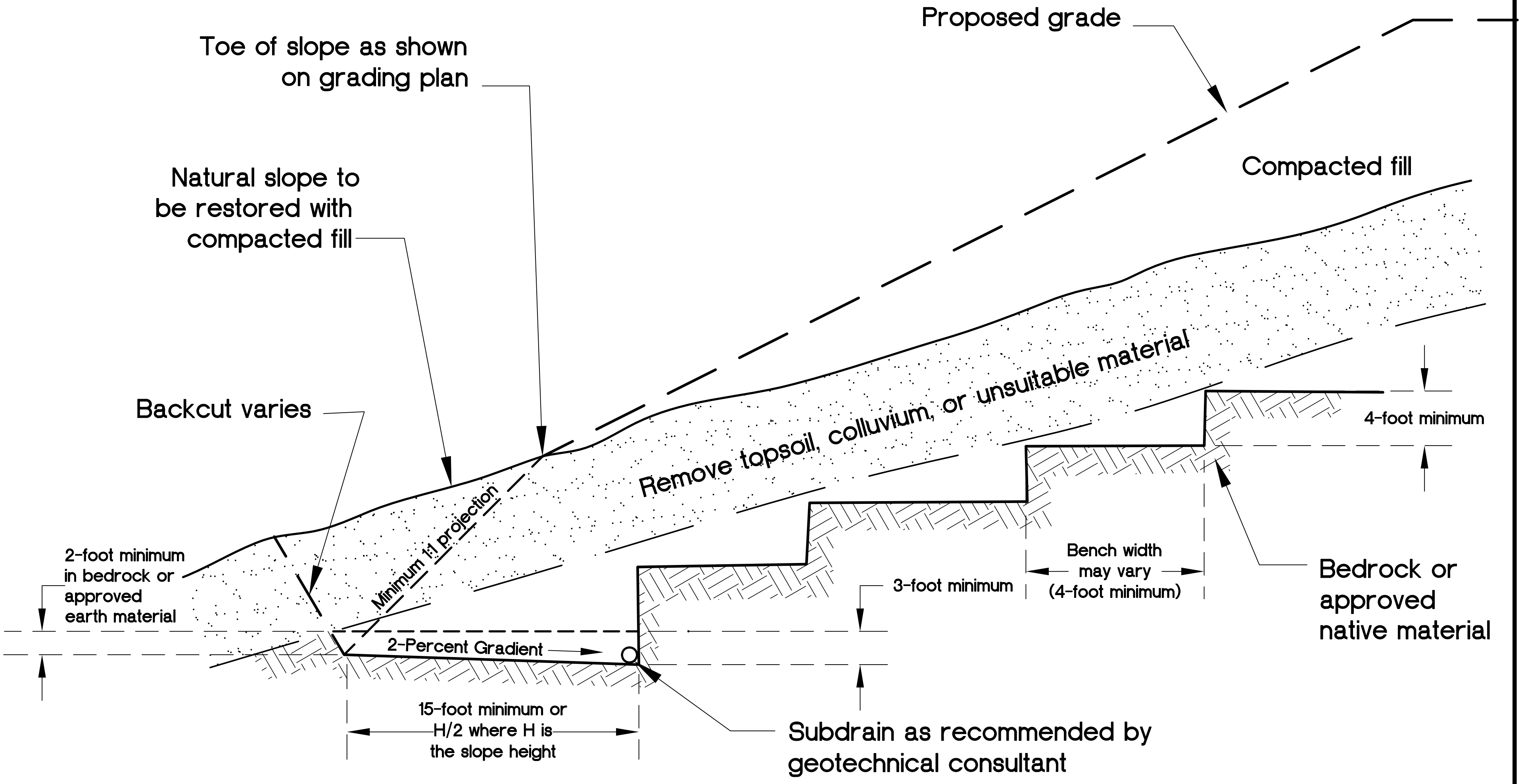
TYPICAL BUTTRESS SUBDRAIN DETAIL

Plate F-6



NOTES:

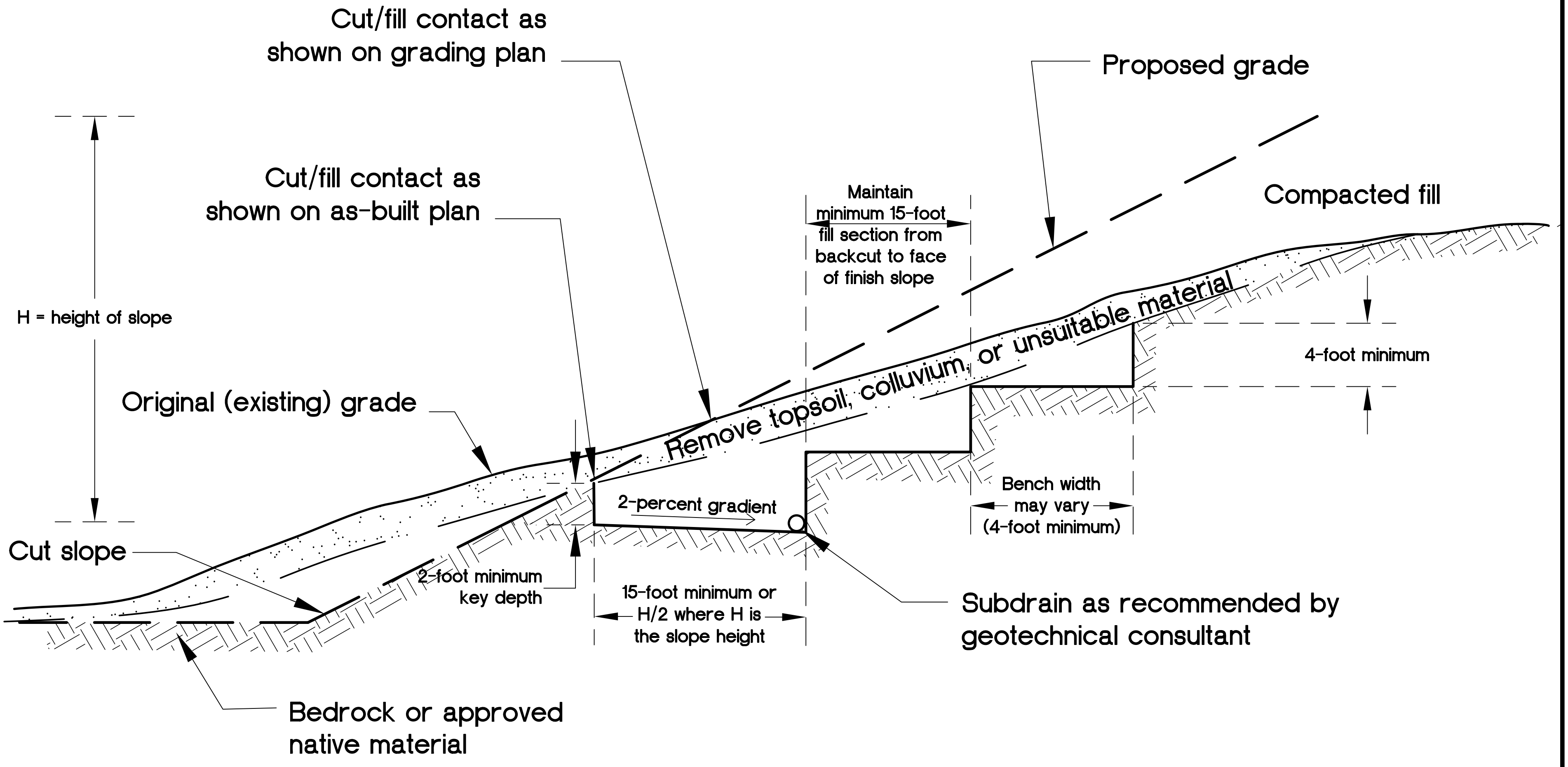
1. Where the natural slope approaches or exceeds the design slope ratio, special recommendations would be provided by the geotechnical consultant.

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

GeoSoils, Inc.

FILL OVER NATURAL (SIDEHILL FILL) DETAIL

Plate F-7

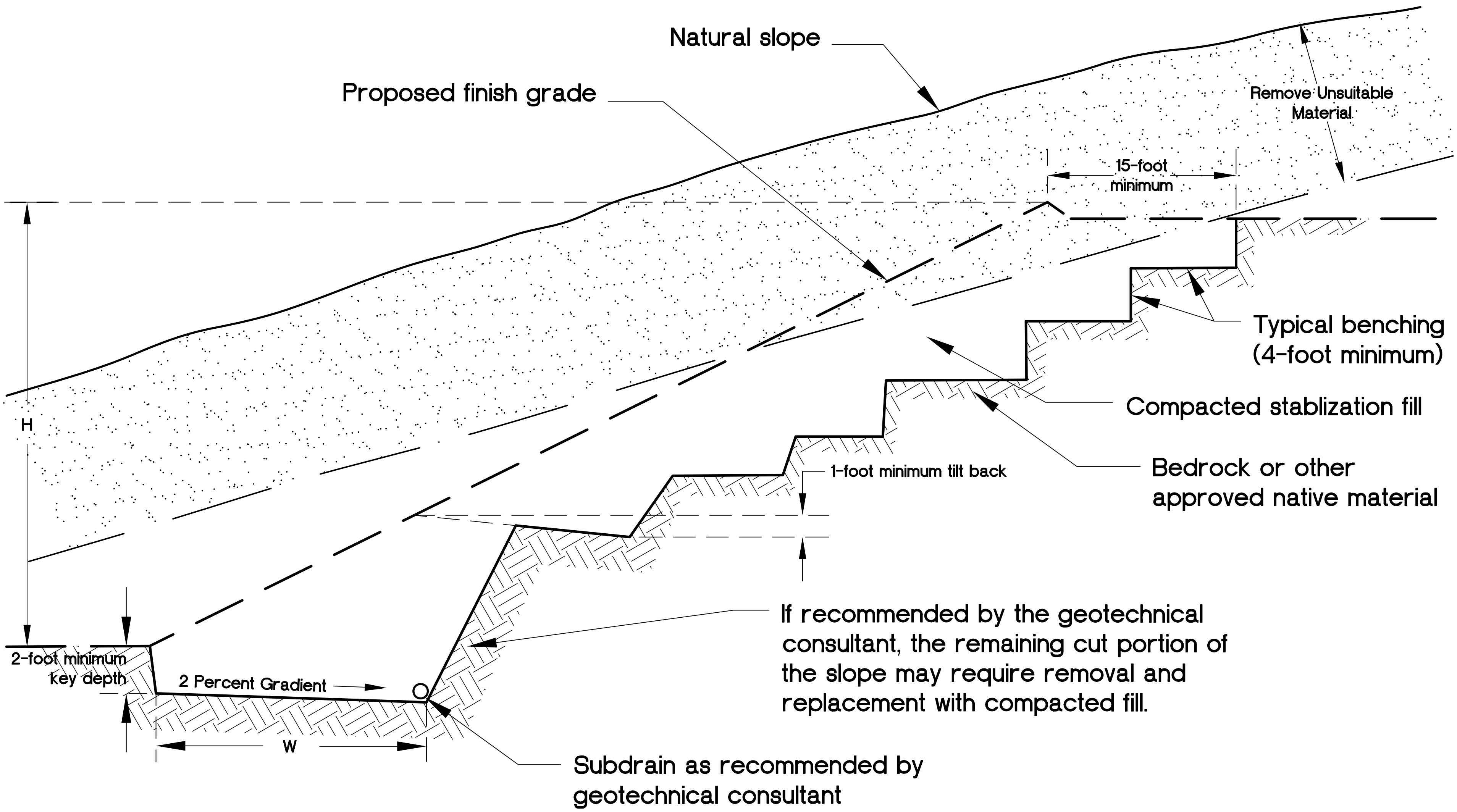


NOTE: The cut portion of the slope should be excavated and evaluated by the geotechnical consultant prior to construction of the fill portion.

GeoSoils, Inc.

FILL OVER CUT DETAIL

Plate F-8



NOTES:

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

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

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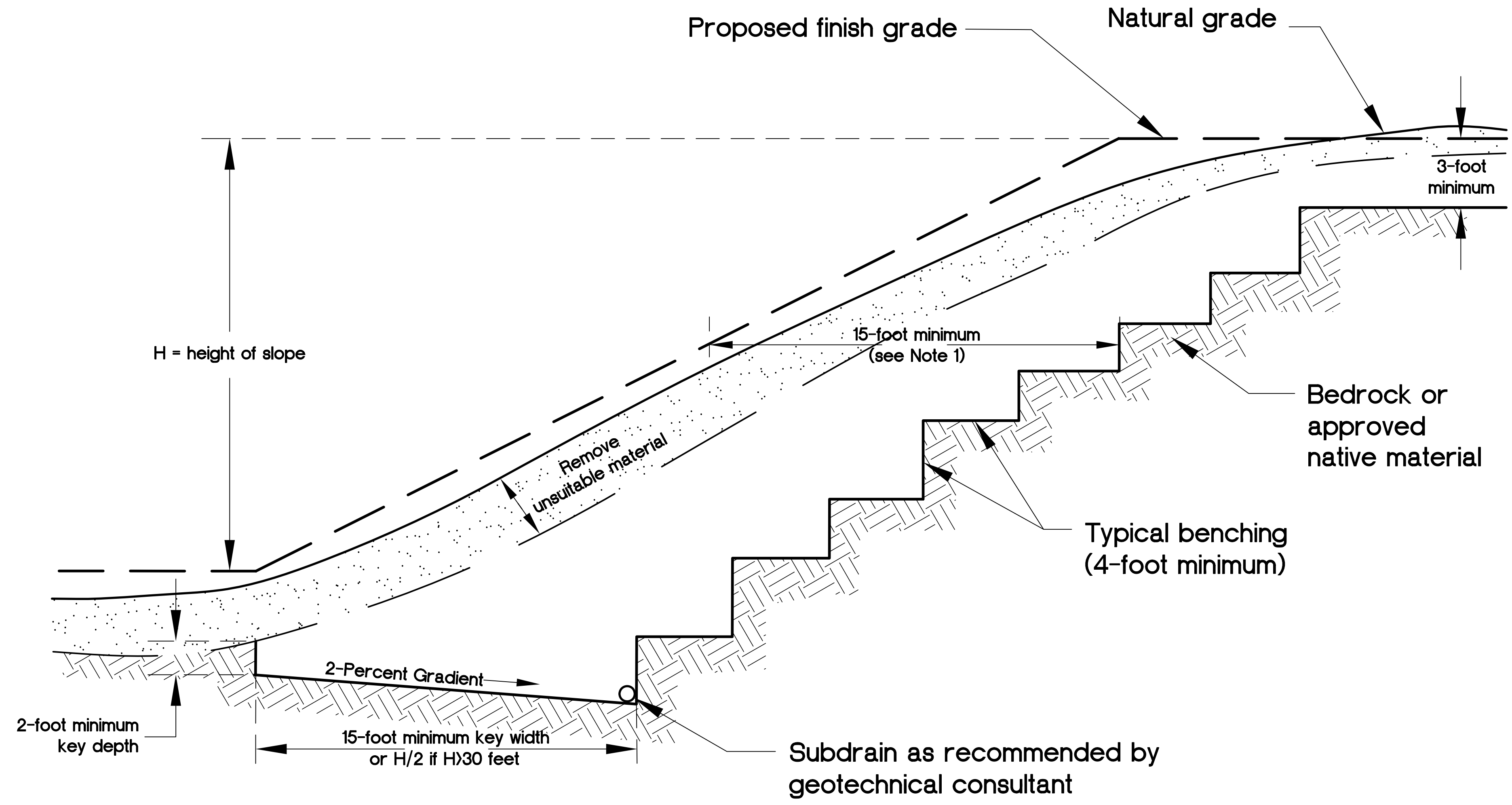
STABLIZATION FILL FOR UNSTABLE MATERIAL EXPOSED IN CUT SLOPE DETAIL

Plate F-9



NOTES:
1. 15-foot minimum to be maintained from proposed finish slope face to backcut.
2. The need and disposition of drains will be evaluated by the geotechnical consultant based on field conditions.
3. Pad overexcavation and recompaction should be performed if evaluated to be necessary by the geotechnical consultant.

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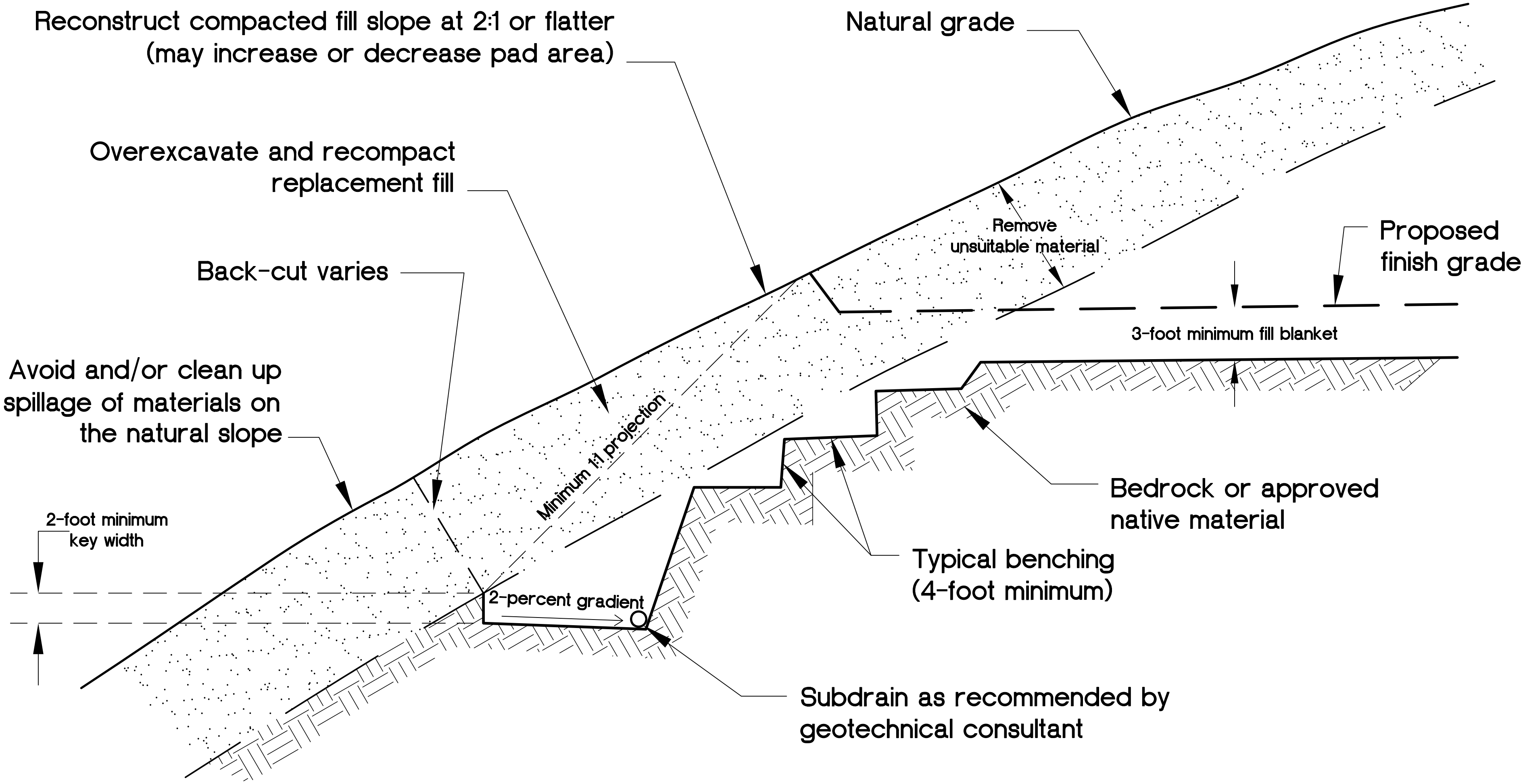
SKIN FILL OF NATURAL GROUND DETAIL

Plate F-10



NOTES:

1. Subdrain and key width requirements will be evaluated based on exposed subsurface conditions and thickness of overburden.
2. Pad overexcavation and recompaction should be performed if evaluated necessary by the geotechnical consultant.

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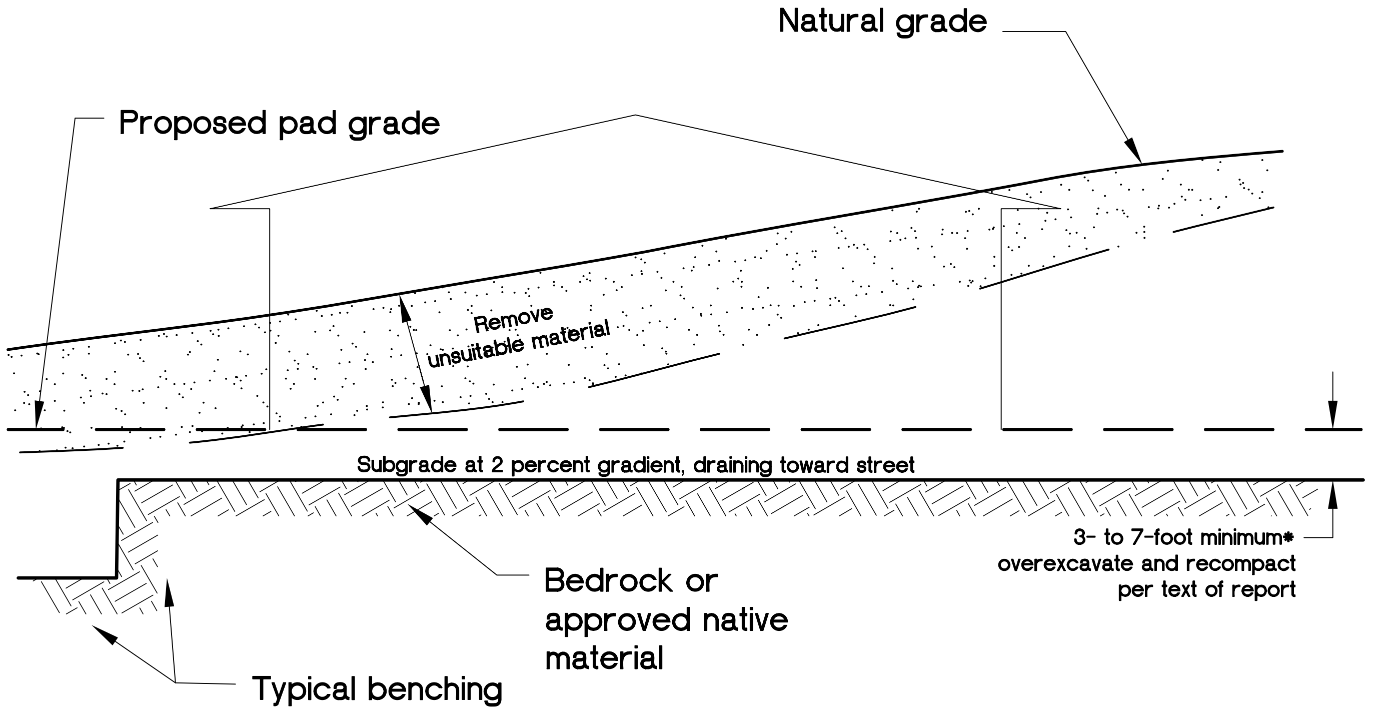
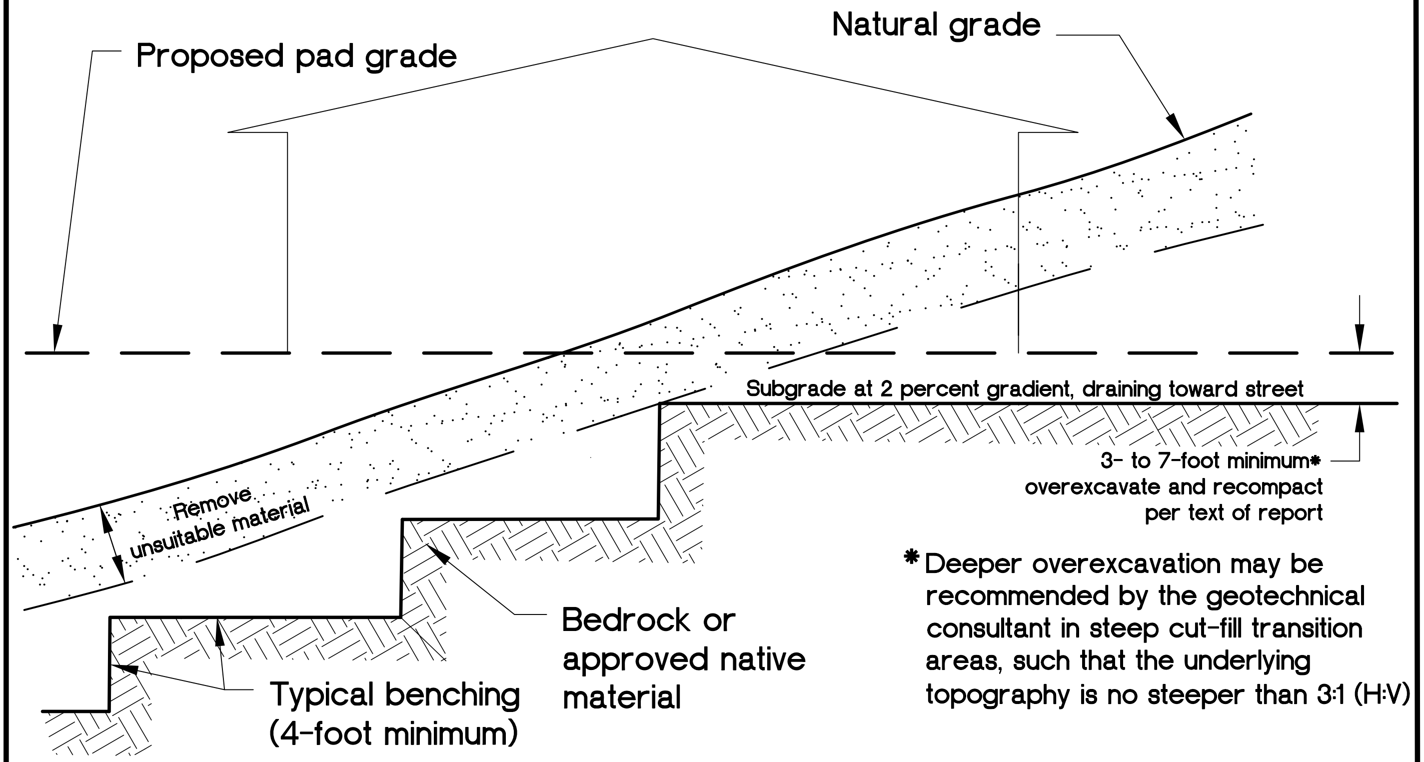
DAYLIGHT CUT LOT DETAIL



CUT LOT OR MATERIAL-TYPE TRANSITION



* Deeper overexcavation may be recommended by the geotechnical consultant in steep cut-fill transition areas, such that the underlying topography is no steeper than 3:1 (H:V)

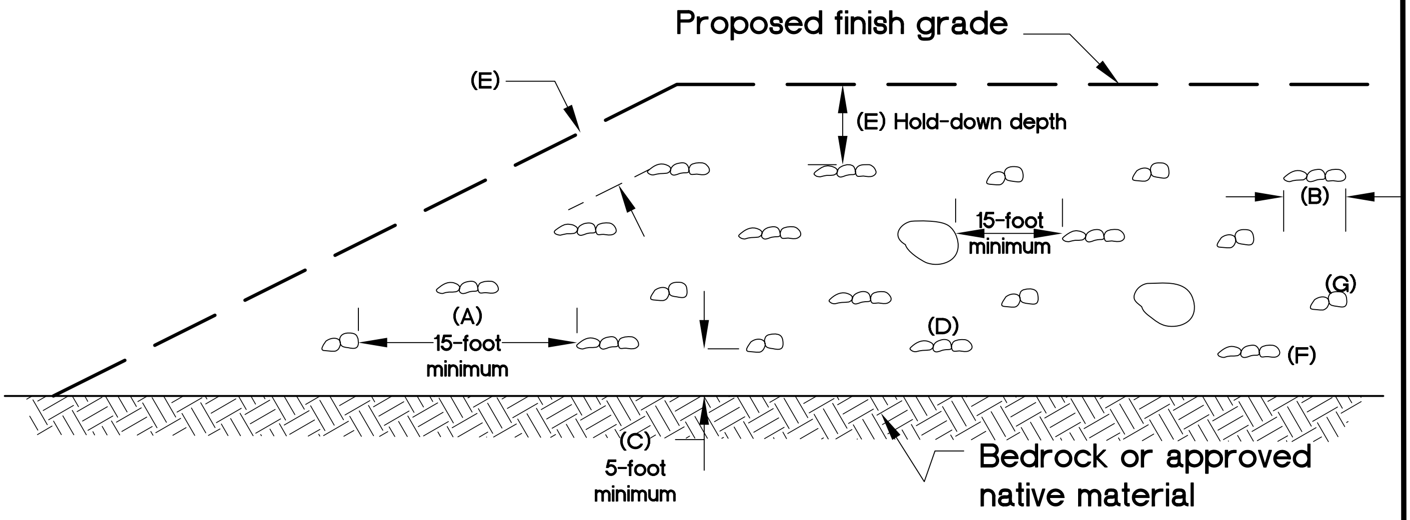
CUT-FILL LOT (DAYLIGHT TRANSITION)

GeoSoils, Inc.

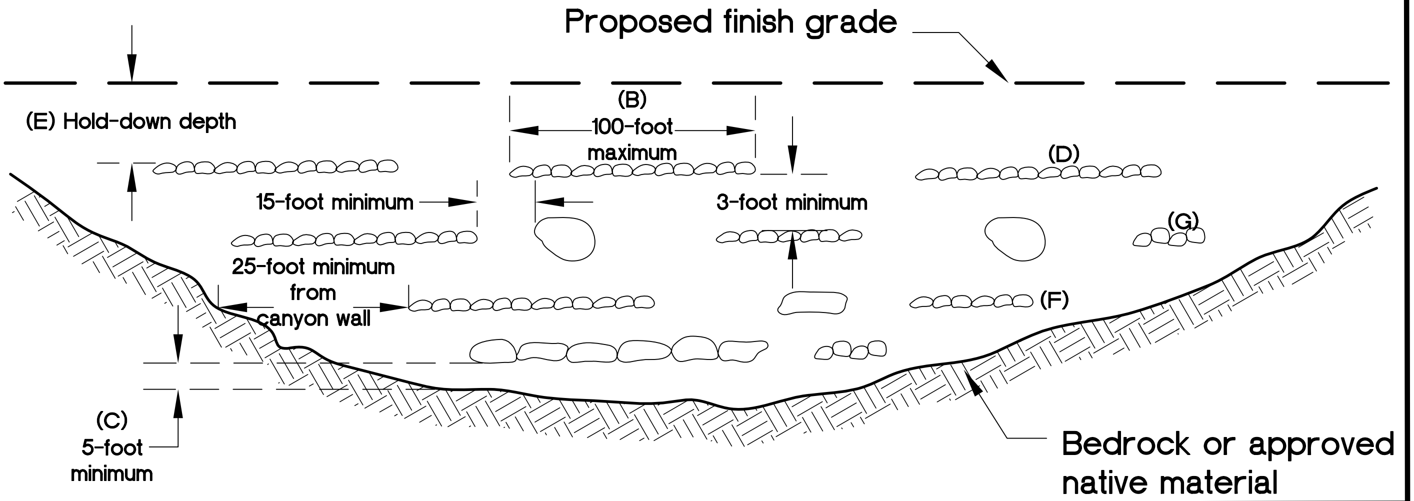
TRANSITION LOT DETAILS

Plate F-12

VIEW NORMAL TO SLOPE FACE



VIEW PARALLEL TO SLOPE FACE



NOTES:

- A. One equipment width or a minimum of 15 feet between rows (or windrows).
- B. Height and width may vary depending on rock slze and type of equipment. Length of windrow shall be no greater than 100 feet.
- C. If approved by the geotechnical consultant, windrows may be placed direclty on competent material or bedrock, provided adequate space is available for compaction.
- D. Orientation of windrows may vary but should be as recommended by the geotechnical engineer and/or engineering geologist. Staggering of windrows is not necessary unless recommended.
- E. Clear area for utility trenches, foundations, and swimming pools; Hold-down depth as specified in text of report, subject to governing agency approval.
- F. All fill over and around rock windrow shall be compacted to at least 90 percent relative compaction or as recommended.
- G. After fill between windrows is placed and compacted, with the lift of fill covering windrow, windrow should be proof rolled with a D-9 dozer or equivalent.

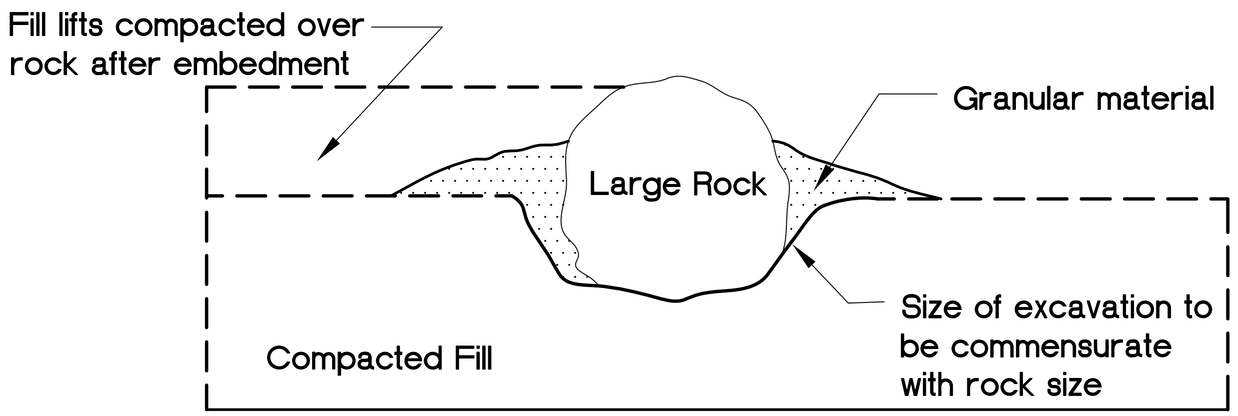
VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED

GeoSoils, Inc.

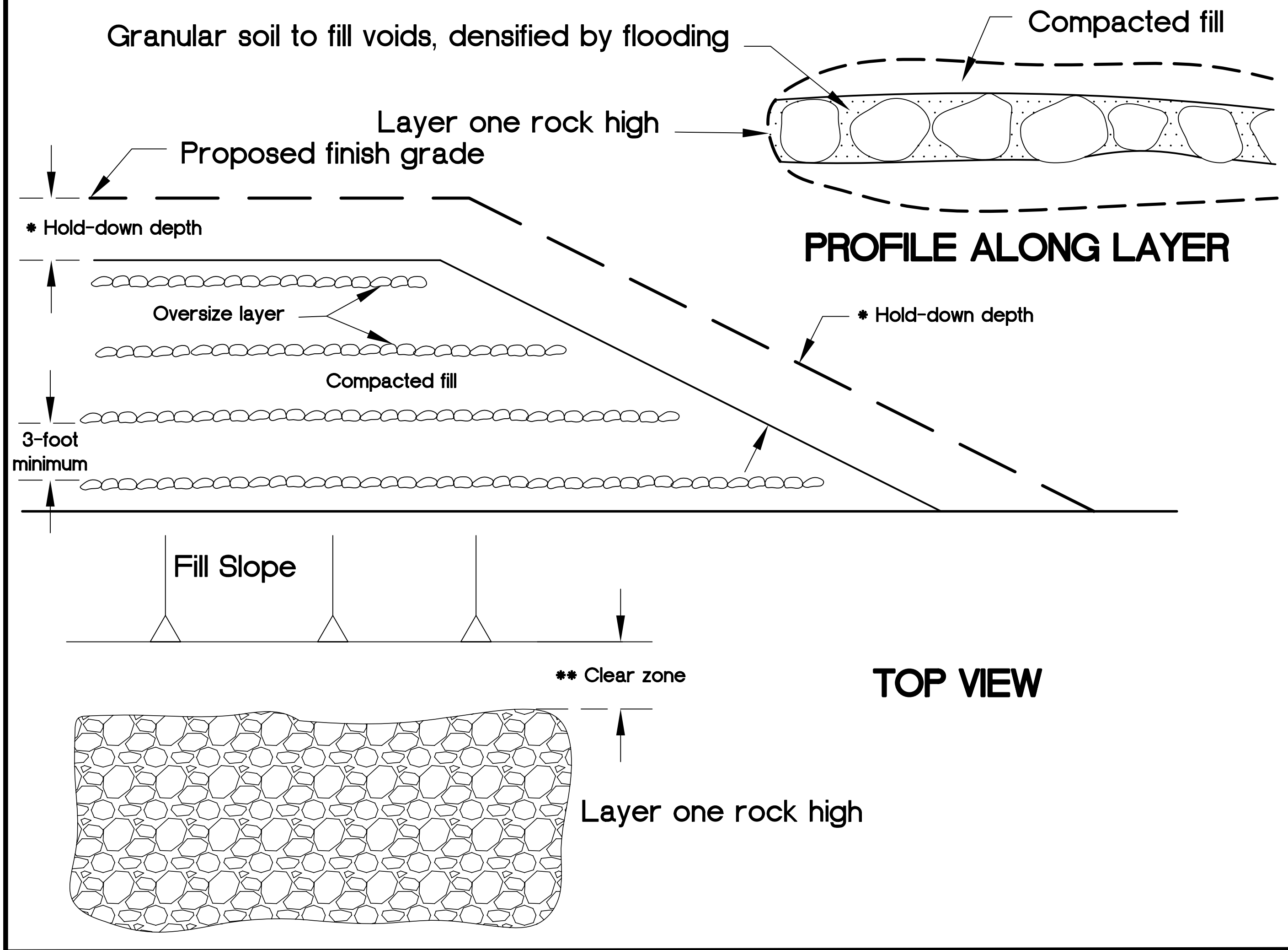
OVERSIZE ROCK DISPOSAL DETAIL

Plate F-13

ROCK DISPOSAL PITS



ROCK DISPOSAL LAYERS

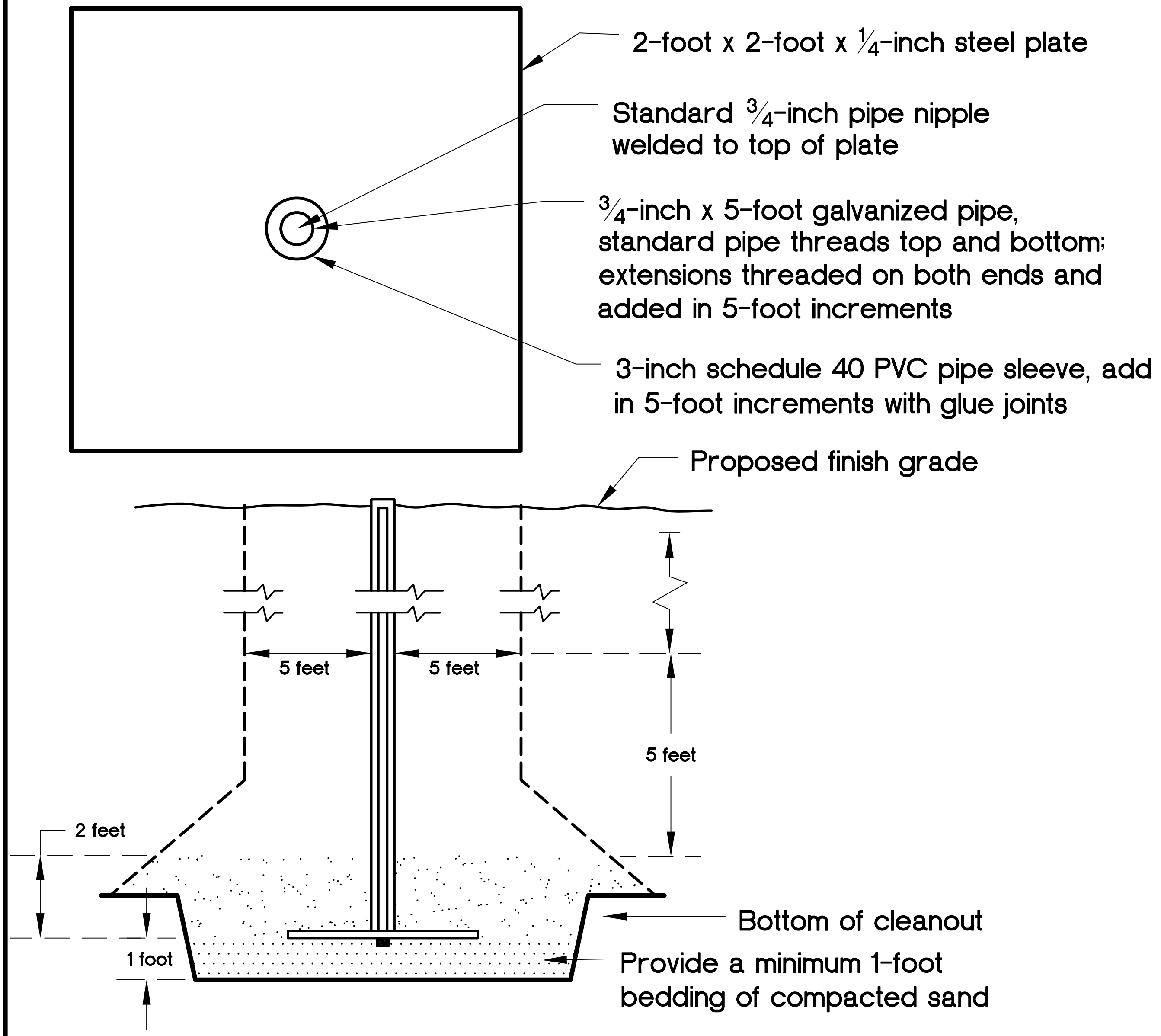


* Hold-down depth or below lowest utility as specified in text of report, subject to governing agency approval.

** Clear zone for utility trenches, foundations, and swimming pools, as specified in text of report.

VIEWS ARE DIAGRAMMATIC ONLY AND MAY BE SUPERSEDED BY REPORT RECOMMENDATIONS OR CODE
ROCK SHOULD NOT TOUCH AND VOIDS SHOULD BE COMPLETELY FILLED IN

GeoSoils, Inc.	ROCK DISPOSAL DETAIL	Plate F-14



2-foot x 2-foot x $\frac{1}{4}$-inch steel plate

Standard $\frac{3}{4}$-inch pipe nipple welded to top of plate

$\frac{3}{4}$-inch x 5-foot galvanized pipe, standard pipe threads top and bottom; extensions threaded on both ends and added in 5-foot increments

3-inch schedule 40 PVC pipe sleeve, add in 5-foot increments with glue joints

Proposed finish grade

Bottom of cleanout

Provide a minimum 1-foot bedding of compacted sand

NOTES:

1. Locations of settlement plates should be clearly marked and readily visible (red flagged) to equipment operators.
2. Contractor should maintain clearance of a 5-foot radius of plate base and withiin 5 feet (vertical) for heavy equipment. Fill within clearance area should be hand compacted to project specifications or compacted by alternative approved method by the geotechnical consultant (in writing, prior to construction).
3. After 5 feet (vertical) of fill is in place, contractor should maintain a 5-foot radius equipment clearance from riser.
4. Place and mechanically hand compact initial 2 feet of fill prior to establishing the initial reading.
5. In the event of damage to the settlement plate or extension resulting from equipment operating within the specified clearance area, contractor should immediately notify the geotechnical consultant and should be responsible for restoring the settlement plates to working order.
6. An alternate design and method of installation may be provided at the discretion of the geotechnical consultant.

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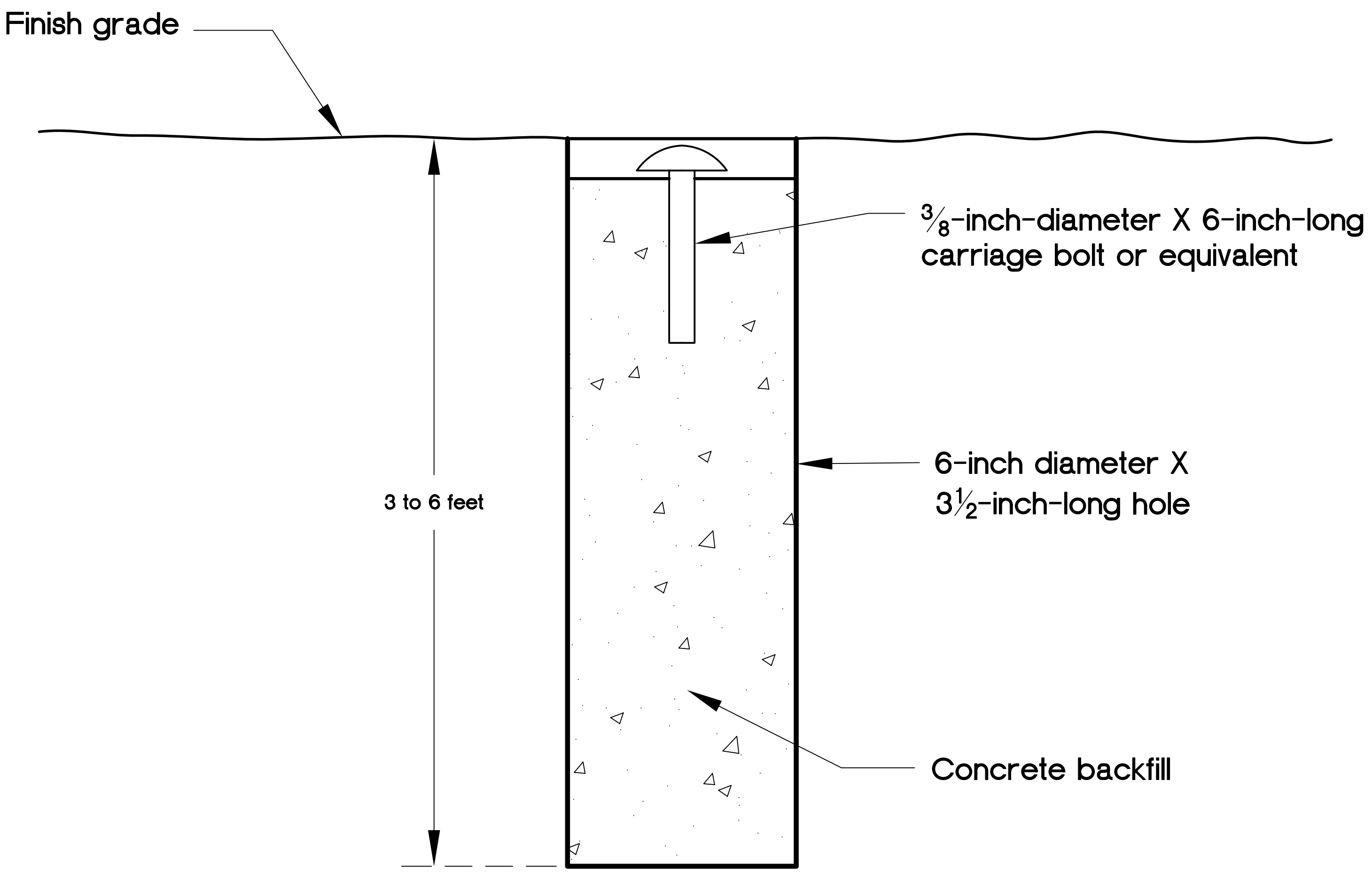
SETTLEMENT PLATE AND RISER DETAIL

Plate F-18



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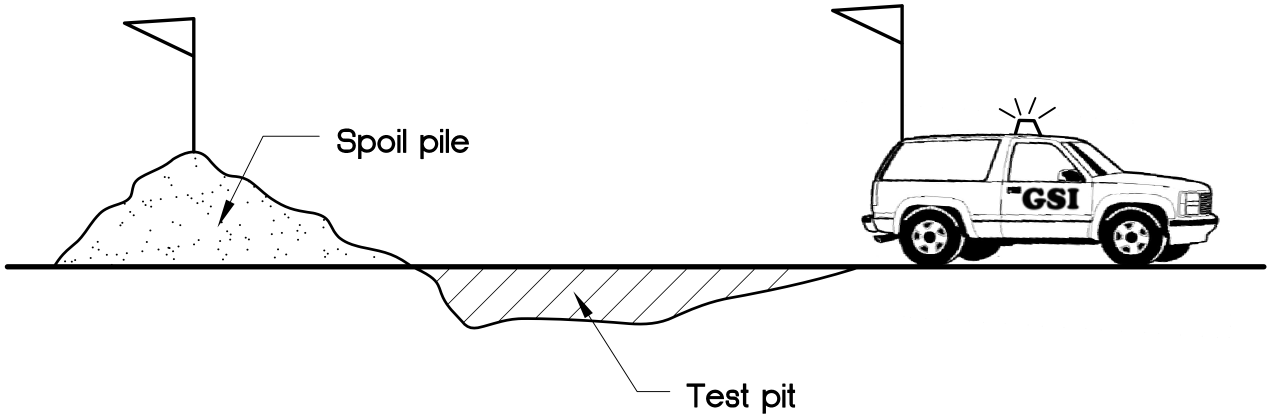
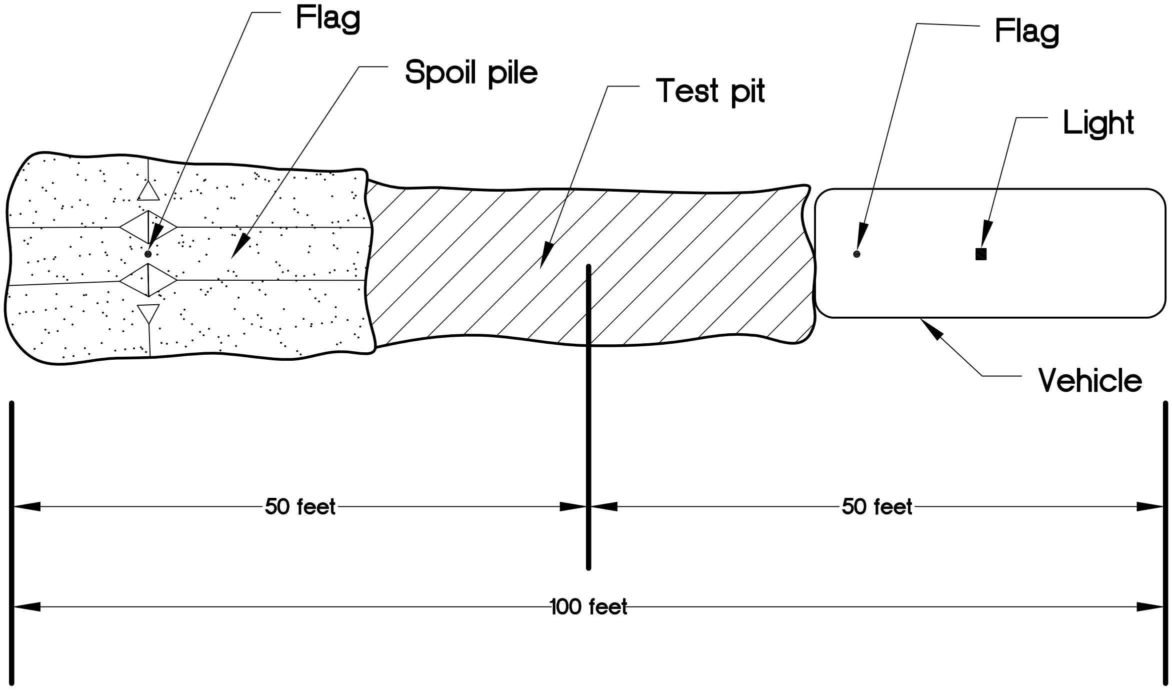
TYPICAL SURFACE SETTLEMENT MONUMENT

Plate F-19

SIDE VIEW



TOP VIEW



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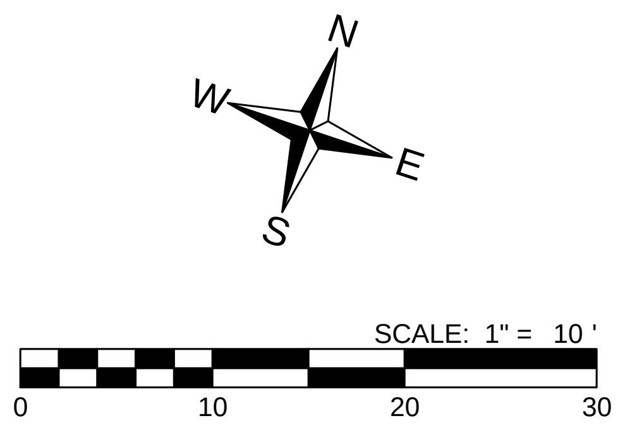
TEST PIT SAFETY DIAGRAM

Plate F-20

GSI LEGEND

Qb	QUATERNARY BEACH DEPOSITS
Qcol	QUATERNARY COLLUVIUM
Qop	QUATERNARY OLD PARALIC DEPOSITS, CIRCLED WHERE BURIED
Tt	TERTIARY TORREY SANDSTONE, CIRCLED WHERE BURIED
⁓?⁓	APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN
B-4 ◓ TD=46'	APPROXIMATE LOCATION OF EXPLORATORY BORING WITH TOTAL DEPTH IN FEET
B-1 ⊕ TD=47 ½'	APPROXIMATE LOCATION OF EXPLORATORY BORING PROJECTED FROM 210 NEPTUNE (GSI, 2000) WITH TOTAL DEPTH IN FEET
N.A.P.	NOT A PART OF THIS STUDY
X—X'	LOCATION OF GEOLOGIC CROSS SECTION



BASE MAP PROVIDED BY:

COHN + ASSOCIATES

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.

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GEOTECHNICAL MAP

Plate 1

W.O. 7557-A-SC | DATE: 04/20 | SCALE: 1" = 10'

GSI LEGEND

Qb — QUATERNARY BEACH DEPOSITS

Qcol — QUATERNARY COLLUVIUM

Qop — QUATERNARY OLD PARALIC DEPOSITS

Tt — TERTIARY TORREY SANDSTONE

—?— — APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN

▽ — APPROXIMATE GROUNDWATER ELEVATION

—·—·2° — APPARENT DIP OF BEDDING

🟩 — CURRENT TIDAL RANGE 2019

🟥 — TIDAL RANGE 2094 W/ 0.5% PROBILITY OF EXCEEDANCE W/ 6.3 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. 7557-A-SC
DATE: 04/20
SCALE: 1"=10'

GEOTECHNICAL RESPONSE TO “REVIEW OF ‘PRELIMINARY GEOTECHNICAL INVESTIGATION, 216 NEPTUNE AVENUE, ENCINITAS, CALIFORNIA,’” DATED FEBRUARY 6, 2021, BY GEOPACIFICA

FOR

MR. WESLEY NEWMAN
2033 SAN ELIJO AVENUE, #131
CARDIFF, CALIFORNIA 92007

W.O. 7557-A5-SC   APRIL 13, 2021

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

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

April 13, 2021

W.O. 7557-A5-SC

Mr. Wesley Newman
2033 San Elijo Avenue, #131
Cardiff, California 92007

Subject: Geotechnical Response to "Review of 'Preliminary Geotechnical Investigation, 216 Neptune Avenue, Encinitas, California,'" dated February 6, 2021, by GeoPacifica

Dear Mr. Newman:

In accordance with the request of your Architect, Mr. Gary Cohn, and as required by the City of Encinitas, GeoSoils, Inc. (GSI) is responding herein to the review comments by the City's technical reviewer, Mr. James Knowlton of GeoPacifica, regarding GSI's April 14, 2020 geotechnical investigation report for the site. The scope of our services has included a review of the selected references in the Appendix, geologic and engineering analyses, and preparation of this review response. Unless specifically superceded herein, the conclusions and recommendations contained in the subject GSI report remain pertinent and applicable, and should be appropriately implemented during project planning, design, and construction.

REVIEW RESPONSE

For ease of review, the reviewers comments are repeated below in *italic*, followed by GSI's response.

Review Comment No. 1

Although the consultant and civil engineer for the project has done an elaborate photogrammetric analysis over a long history of the site and has addressed sea level rise the analysis does not take into account the physical properties of the soil material at the site and does not account for the major cause of rapid sea bluff erosion along the coast, major tides and a major storm occurring at the same time. These events can cause multiple feet of erosion at the toe of the bluff at one time. It is not predictable and can and will happen. Although the bluff retreat rate is the opinion of the geotechnical consultant and the civil engineer, it is not a rate that has been accepted or approved by the California Coastal Commission staff and will probably result in an appeal of the project. Erosion rates of between .33-.50 feet per year are a more acceptable erosion rate based upon current reports and documentation utilized by the City of Encinitas and Coastal Commission staff. Just for an erosion rate for the next 75 years the rate should be from 25 feet to approximately 40 feet for an erosion rate setback.

Response No. 1

We used reasonable soil strength parameters in our slope stability analysis. In fact, a special effort was made to be as accurate as economically possible when modeling site soil physical properties in our slope stability analysis. To that end, samples were retrieved from a supplemental boring (for this purpose), every 2½ feet in the old paralic deposits, until very dense bedrock (Torrey Sandstone), was encountered.

As far as using soils strength to determine a historic erosion rate, the actual rate is a direct measurement of the bluff retreat over a long time period. The determined rate reflects the strengths of the bluff material specifically at this site. The future retreat rate, which is based upon the rate of sea level rise (SLR) and the historic retreat rate, does account for the site specific soil strength parameters utilized.

We agree that times of high tides and high waves can result in direct marine erosion at the base of the bluff. However, the actual erosion event, the bluff failure, does not necessarily occur during high tide and high waves (for instance, the unfortunate recent Leucadia Seabluffe failure, which occurred on a warm and sunny day, at low tide). We agree that, for the most part, bluff erosion is episodic and occurs a few feet per event. However, by using a longer time period for the historic photograph analysis (88 years), the rate is a site specific, documented representative average erosion rate, that includes past periods of high tides and high waves and rainstorms, all occurring simultaneously.

The historic rate provided by GSI was determined using the CCC guidelines for evaluating bluff erosion rates, and used the best available science. The greater erosion rates in the area mentioned by the reviewer are not site specific, not as analytical as the photogrammetry analysis used by GSI, and certainly not as precise. Furthermore, the US Army Corps of Engineers ([USACE] 2015) even document that some bluffs in Encinitas have pedogenic (soil forming) cementation (iron oxide), have been stable dating back to the 1890s (no subaerial [bluff top] erosion). This suggests no substantive marine erosion. In fact, the bluffs at the subject site exhibit this resistant pedogenic development, as shown in the photograph on the following page.

Review Comment No. 2

Although the geotechnical report did a slope stability analysis and determined a factor of safety setback and depicted it on a geologic cross-section, the numerical amount of setback for the 1.5 factor of safety was never given in the report. Please provide the amount of setback.

Response No. 2

The numerical amount of the setback (width) for the 1.5 factor of safety (FOS) is 11.15 feet, or about 11 feet to the top of bluff. As indicated in GSI (2020), the accelerated future erosion rate owing to SLR, indicates a range of bluff erosion (over 75 years) of 1.94 (low) to 9.31 (high) feet. When added to the 11 feet FOS line, the additive FOS + 75-year erosion setback line falls well within the prescriptive 40-foot top of bluff setback, using the best available science.

View to the east-southeast showing the vacant lot at 216 Neptune Ave., and the resistant iron oxide pedogenic cementation of the upper bluffs in this area.

Review Comment No. 3

Although the City of Encinitas Local Coastal Plan indicates the determination of bluff setback should be 40 feet if the erosion rate or factor of safety is less than 40 feet. However, to comply with coastal commission requirements the consultant should combine the factor of safety setback and erosion rate setback to determine the geologic setback line for the property (Reference #2 ["Establishing Development Setbacks from Coastal Bluffs, by Mark Johnsson for CCC, 2003]*). It appears that the erosion rate setback (not the one presented by the geotechnical consultant) and the factor of safety setback may exceed the 40 foot setback proposed and approved by the geotechnical consultant. A discussion should be held with the applicant and the City of Encinitas.*

Response No. 3

A "Zoom" meeting was held with the City of Encinitas, the applicant's representative, Mr. Gary Cohn, and GSI on April 6, 2021, and GSI was directed to prepare this response. As indicated above, using the best available science, and methods prescribed by the Coastal Commission to determine erosion rates, the geologic setback line falls well within the prescriptive 40-foot top of bluff setback.

However, in the spirit of compromise, rather than using the site specific rates derived for this site, GSI will utilize the lowest erosion rate for the Encinitas reach (Benumoff and Griggs, 1999), representing the lower bound for the site, and the mean erosion rate for the Encinitas reach (Benumoff and Griggs, 1999), representing the upper bound for the site. These values are 4 cm/yr and 7.7 cm/yr, respectively. This corresponds to 0.13 ft/yr and 0.25 ft/yr, respectively, which are an order of magnitude larger than our site specific determinations.

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 study (PLSA, 2020) and Benumoff and Griggs (1999), is between 0.13 and 0.25 ft/yr = R_1.
2. Avg SLR rate over 87 years (1932 to 2019), based on NOAA (Gloss Station Handbook Scripps Pier, La Jolla) is 2.148 mm/yr = 0.085 inch/yr x 1 ft/12 in = 0.007083 ft/yr = S1
3. Future SLR rate (2095), under *medium-high risk aversion scenario* = 6.3 ft/75 yrs = 0.084 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 GSI [2020], 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 2056). We have utilized the endpoints of the range of 0.13 ft/yr and 0.25 ft/yr for this time interval. The erosion rate should marginally increase for the following 25 years (2057-2081), and we have reasonably added ⅓ of the change in erosion rate in 2095, to the initial erosion rate. During the more asymptotic SLR end of the 75-year design life (2082-2095), 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 bluff erosion rates discussed above are indicated in the calculations below:

Low Site Rate

At year 2095, under *medium-high risk aversion scenario (0.5% Probability)*,

$$R_2 = R_1\ (S_2/S_1)^m$$
$$R_2 = (0.13\text{ ft/yr})\ (0.084\text{ ft/yr/[}0.007083\text{ ft/yr]})^{⅓}$$
$$R_2 = (0.13)\ (11.86)^{⅓}$$
$$R_2 = (0.13)(2.28)\ = 0.296\text{ ft/yr in the year 2095.}$$

Based on the above, the retreat rate will change from 0.13 to 0.296 ft/yr, and the difference between the 75-year commencement and end of the design life, Δ= 0.166 ft/yr, from 2020 to 2095.

FUTURE BLUFF RETREAT BASED ON SLR CURVE INCREMENTS			
APPLICABLE DATES	BLUFF RETREAT RATE (FT/YR)	DURATION (YEARS)	BLUFF RETREAT (FEET)
2020-2056 (0.13) SLR rate	0.13	37	4.81
2057-2081 (0.13 + ⅓[0.166]= 0.186) increase in SLR rate	0.186	25	4.65
2082-2095 (Calculated SLR rate in 2095)	0.296	13	3.85
Totals		**75**	**13.31**

As shown above, the onsite coastal bluff may experience approximately 13.5 feet of retreat over the 75-year design life of the proposed residential structure. 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 2095, based on the ±13.5 feet of bluff retreat, with an assumed SLR of 6.3 feet over that interval.

High Site Rate

At year 2095, assuming the high site specific rate applied under *medium-high risk aversion scenario (0.5% Probability)*,

$$R_2 = R_1\ (S_2/S_1)^m$$
$$R_2 = (0.25\text{ ft/yr})\ (0.084\text{ ft/yr/[}0.007083\text{ ft/yr]})^{⅓}$$
$$R_2 = (0.25)\ (11.86)^{⅓}$$
$$R_2 = (0.25)(2.28)\ = 0.57\text{ ft/yr in the year 2095.}$$

Based on the above, the retreat rate will change from 0.25 to 0.57 ft/yr, and the difference between the 75-year commencement and end of the design life, Δ= 0.32 ft/yr, from 2020 to 2095.

FUTURE BLUFF RETREAT BASED ON SLR CURVE INCREMENTS			
APPLICABLE DATES	BLUFF RETREAT RATE (FT/YR)	DURATION (YEARS)	BLUFF RETREAT (FEET)
2020-2056 (0.25) SLR rate	0.25	37	9.25
2057-2081 (0.25 + ⅓[0.32]= 0.357) increase in SLR rate	0.357	25	8.93
2082-2095 (Calculated SLR rate in 2095)	0.57	13	7.41
Totals		75	25.59

As shown above, using the high rate, the onsite coastal bluff may experience approximately 25.5 feet of retreat over the 75-year design life of the proposed residential structure. 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 2095, based on the ±25½ feet of bluff retreat, with an assumed SLR of 6.3 feet over that interval. We note that regardless of the lower or upper bound Benumoff and Griggs (1999) erosion rate utilized, when added to the static FOS line from the bluff edge, these rates plot inside of the City of Encinitas prescriptive 40-foot bluff-edge setback zone.

As indicated above,the distance from the bluff edge to the setback is shown graphically on Plates 1 and 2, and is numerically summarized in the table below:

CASE	EROSION DISTANCE FROM BLUFF EDGE TO 75 YEAR RATE (FEET)	DISTANCE FROM EDGE OF BLUFF TO FOS = 1.5 (FEET)	DISTANCE FROM EDGE OF BLUFF TO GEOLOGIC SETBACK LINE (FEET)
Lower Limit over 75 years	13.31	11.15	24.46
Upper limit over 75 years	25.59	11.15	37.09

Accordingly, Encinitas prescriptive 40-foot bluff-edge setback governs.

GSI reiterates:

- The proposed project 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).
- 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 residential structure 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.

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
No. 1340

DAVID W. SKELLY
No. RCE 47857
Exp. 12/31/21

John P. Franklin
Engineering Geologist, CEG 1340

David W. Skelly
Civil Engineer, RCE 47857

JPF/DWS/mn

Attachments: Appendix - Selected References
Plate 1a - Geotechnical Map
Plate 2a - Geologic Cross Section X-X’

Distribution: (3) Addressee (3 wet signed)

APPENDIX

SELECTED REFERENCES

APPENDIX

SELECTED REFERENCES

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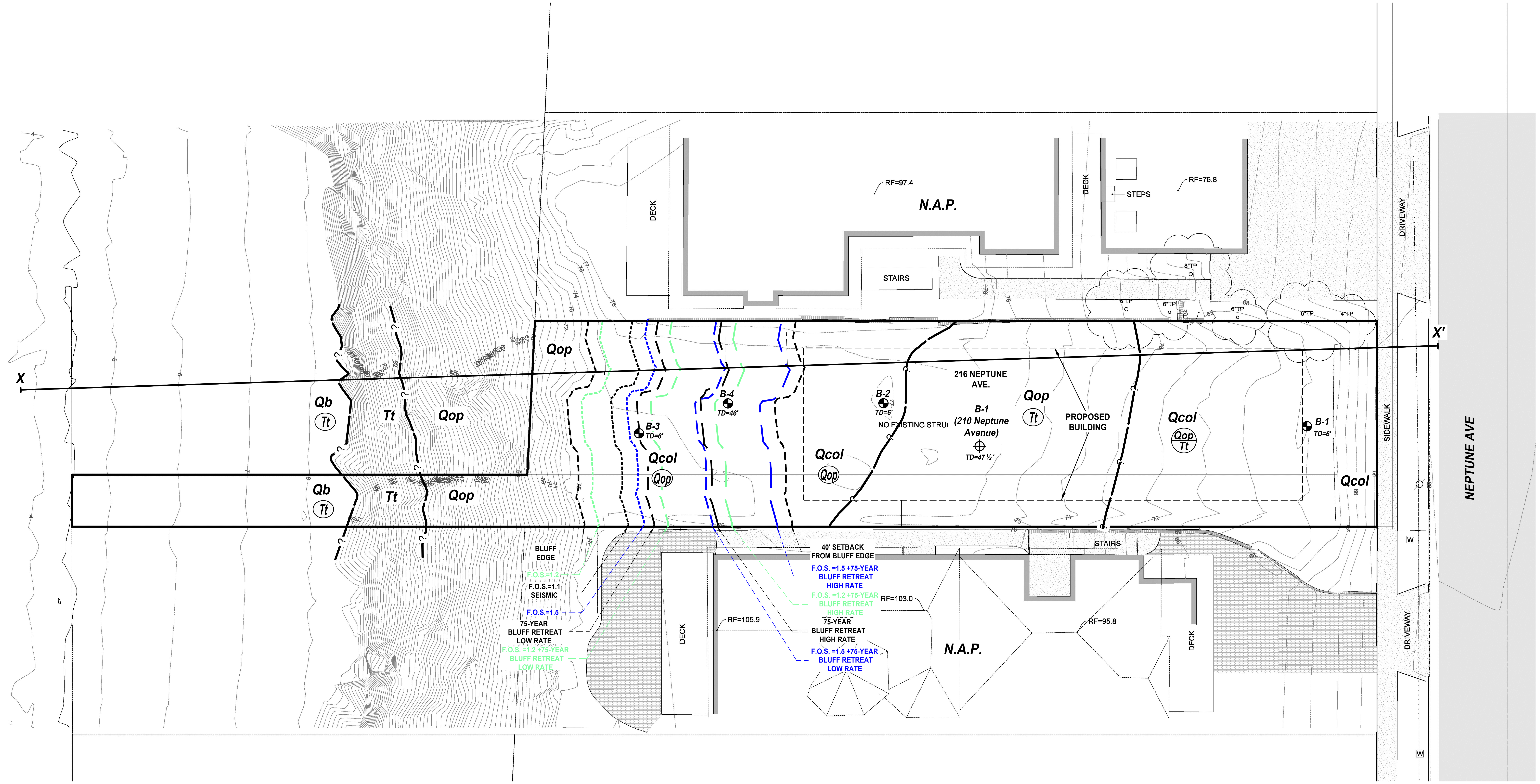


LEGEND

FOUND MONUMENT

AS INDICATED

PROPERTY LINE

RIGHT-OF-WAY LINE

CENTER LINE

ADJOINING PROPERTY LINE

TIE LINE / REFERENCE LINE

EASEMENT LINE

OE — POWER - OVERHEAD

x — x — FENCE

WALL - RETAINING

BUILDING OUTLINE

BUILDING OVERHANG

INDEX CONTOUR LINE

INTERMEDIATE CONTOUR LINE

×60.4 SPOT ELEVATION

CONCRETE

BRICK

ASPHALT

W METER - WATER

UTILITY POLE

CLEANOUT

S MH - SANITARY

8" TD TREE - DECIDUOUS

8" TP TREE - PALM

8" TC TREE - CONIFEROUS

RF ROOF

GSI LEGEND

Qb — QUATERNARY BEACH DEPOSITS

Qcol — QUATERNARY COLLUVIUM

Qop — QUATERNARY OLD PARALIC DEPOSITS, CIRCLED WHERE BURIED

Tt — TERTIARY TORREY SANDSTONE, CIRCLED WHERE BURIED

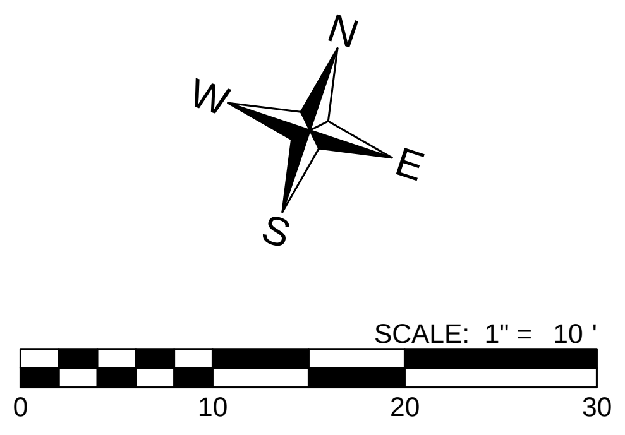
—?— — APPROXIMATE LOCATION OF GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN

B-4 TD=46' — APPROXIMATE LOCATION OF EXPLORATORY BORING WITH TOTAL DEPTH IN FEET

B-1 TD=47½' — APPROXIMATE LOCATION OF EXPLORATORY BORING PROJECTED FROM 210 NEPTUNE (GSI, 2000) WITH TOTAL DEPTH IN FEET

N.A.P. — NOT A PART OF THIS STUDY

X X' — LOCATION OF GEOLOGIC CROSS SECTION



BASE MAP PROVIDED BY:

COHN + ASSOCIATES

ALL LOCATIONS ARE APPROXIMATE

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GeoSoils, Inc.

GEOTECHNICAL MAP

Plate 1a

W.O. 7557-A4-SC | DATE: 04/21 | SCALE: 1" = 10'

ALL LOCATIONS ARE APPROXIMATE

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GeoSoils, Inc.

GEOLOGIC
CROSS SECTION X-X'

Plate 2a

W.O. 7557-A-SC	DATE: 04/21	SCALE: 1"=10'

GEOTECHNICAL RESPONSE TO "PROPOSED PROJECT AT 216 NEPTUNE AVE/NEWMAN (CITY CASE NO. CDP-003343-2019)," DATED FEBRUARY 3, 2022, BY CALIFORNIA COASTAL COMMISSION

FOR

MR. WESLEY NEWMAN
2258 HIGH TRAIL
VISTA, CALIFORNIA 92084

W.O. 7557-A6-SC REVISED MARCH 22, 2022

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Revised
March 22, 2022

W.O. 7557-A6-SC

Mr. Wesley Newman
2258 High Trail
Vista, California 92084

Subject: Geotechnical Response to "Proposed Project at 216 Neptune Ave/Newman (City Case No. CDP-003343-2019)," dated February 3, 2022, by California Coastal Commission

Dear Mr. Newman:

In accordance with the request of your Architect, Mr. Gary Cohn, GeoSoils, Inc. (GSI) is responding herein to the review comments by the California Coastal Commission (CCC), regarding GSI's geotechnical investigation report for the site. The scope of our services has included a review of the selected references in the Appendix, geologic and engineering analyses, and preparation of this review response. Unless specifically superceded herein, the conclusions and recommendations contained in the referenced GSI reports remain pertinent and applicable, and should be appropriately implemented during project planning, design, and construction.

REVIEW RESPONSE

For ease of review, the CCC's comments are repeated below in *italic*, followed by GSI's response.

Review Comment No. 1.1

The applicant's projections of future bluff retreat, based on historical retreat rates estimates of 0.02 – 0.09 ft/yr and use of the simplified SCAPE equation (Ashton et al., 2011) to project bluff retreat rates with future sea level rise (SLR), is not sufficiently precautionary for several reasons: (1)The estimated historical bluff retreat rates for the site (0.02 – 0.09 ft/yr, based on analysis of aerial photos spanning 1932-2018) are notably lower than retreat rates observed elsewhere along the Encinitas bluffs, and are lower than previous estimates for the immediate project vicinity in previous studies.

Even assuming the applicant's study characterizes the historic bluff retreat rate more accurately than previous studies, there is no evidence to suggest that the bluff at the project site is intrinsically more resistant to erosion in comparison to the bluffs in the surrounding area, or that the lower rates observed at the site are not simply due to the sequencing of bluff erosion events.

As such, the higher bluff retreat rates observed by Benumoff and Griggs (1999) provide a more precautionary baseline for projecting future bluff retreat at the site and should be considered in evaluating the bluff top setback that will minimize hazards and assure the stability of the proposed development over a 75 year period.

Response No. 1.1

GSI's data is based on physical science. The historical bluff retreat rates provided were site-specific, and were based on a photogrammetric analysis by California-licensed civil engineers, Pasco Laret Suiter & Associates (PLSA). They were able to conclusively demonstrate that over the period 1932 to 2018 (86 years), the site specific historical erosion rate of the edge of bluff ranged between a low of 1.6 feet/86 years to a high of 7.8 feet/86 years. This is equivalent to a site specific historic rate of 0.0186 (low) to 0.0907 (high) feet/year, and corresponds to a buff retreat rate ranging from about 1.4 to 7 feet over 75 years. This data was corroborated by a review of aerial photographs, from as far back as 1932. This site specific data is considered the best available science in this regard. The CCC typically requires the best available science to be utilized.

It is well known geologically, that the well-cemented nature of the Torrey sandstone is resistant to erosion, more so than the Del Mar and Friars Formations, which are often associated with slope instability. This is the reason for its signature profile "Type C(c)," per Emery and Kuhn (1982). In fact, the study cited by the reviewer (Benumoff and Griggs [1999]) shows that the Torrey bedrock's resistance to erosion, is only exceeded by the more resistant and older Cretaceous age bedrock in La Jolla, along this reach of the California coastline. Furthermore, the US Army Corps of Engineers ([USACE] 2015) even document that some upper bluffs in Encinitas have pedogenic (soil forming) cementation (iron oxide), have been stable dating back to the 1890s (no subaerial [bluff top] erosion). This suggests no substantive marine erosion. GSI notes that the bluffs at the subject site exhibit this resistant pedogenic development, as shown in the photograph below.

Finally, the historic rate provided by GSI was determined using the CCC guidelines for evaluating bluff erosion rates, and used the best available science, including photogrammetry. The greater erosion rates in the area mentioned by the reviewer are not site specific, not as analytical as the photogrammetry analysis used by GSI, and certainly not as precise. We point out that the coauthor, Dr. Benjamin Benumof, of "Benumoff and Griggs (1999)," cited by the reviewer, has previously noticed the CCC he does not agree with their application/interpretation of his paper to their estimates of coastal erosion, specifically for this particular reach of the coastline.

Review Comment No. 1.2

The value of m =0.33 ("site specific response factor" used in the simplified SCAPE equation) as selected by the applicant has no site-specific basis; rather, it is a judgement based on the observation that the simplified SCAPE equation produced higher bluff retreat projections than other models for the bluffs at San Onofre State Beach in another study (Young et al. 2014). It remains uncertain how well the simplified SCAPE equation predicts bluff retreat at the project site, and whether there is good reason to reduce the m parameter from the default of m = 0.5 recommended in the Ashton et al. study. At the very least, bluff retreat projections using m = 0.5 should also be considered in the setback analysis.

Response No. 1.2

The reviewer has ignored the site specific reasons provided in GSI (2020) for justification of the value used. These reasons are enumerated starting on page 18 of GSI (2020). Briefly they are summarized as follows:

- The simplified numerical model is limited to evaluating shoreline erosion along rocky coasts. The site is not characterized as a rocky coast.

- The authors of the equation 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. The subject site falls in the latter category.

- The simplified numerical model does not consider longshore sediment transport, which can either build or decay protective beaches. Longshore sediment transport occurs at the subject site.

- 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. Thus, the use of m=1 is not justified.

  A negative feedback or nonlinear system (0<m<1) include 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 sediment volume beaches. The site is not dominated by wave-driven erosion and has a high sediment volume beach. Thus, the use of m=0.5 is not justified.

  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. While the lower bluff is comprised of hard rock, the use of m=0 is not justified, owing to the other criteria.

  Regarding GSIs (2020) Figure 6 (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 design 6.3 feet of SLR is postulate to occur for design purposes [GSI, 2020]). Given the closeness to the BB:0 line, m must be less than 0.5, and closer to m=0. Therefore, m= 0.333 (or ⅓) is justified for this site.

- Ashton, et al. (2011), does not recommend m = 0.5. To be sure, m=0.5 was considered for this site, but it is not appropriate.

Review Comment No.1.3

Based on this preliminary analysis, use of the other, more conservative historical retreat rates discussed above, along with both the simplified SCAPE equation and the USGS

CoSMoS Bluff Retreat tool, yields significantly larger estimates of future bluff retreat under high sea level rise conditions (i.e., 6.3 – 7.1 feet in 2100). When combined with the approximately 11 foot bluff edge setback necessary to achieve a 1.5 (static) factor of safety (FOS) under present-day conditions, as provided in the applicant's analysis, the 75-year bluff retreat values from our analysis suggest total geologic setbacks in excess of 40 feet would be necessary to assure stability over the full project life. (See Table 1 below)....

Response No. 1.3

The limitations of the SCAPE equation, and misapplication of the Benumof and Griggs (1999) paper by the CCC are discussed above. The reviewer again ignored the discussion in GSI (2020) regarding COSMOS, starting on page 14 of that document. This is summarized below:

- The CoSMoS website contains a disclaimer stating that, **"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."** This is exactly what the CCC is doing, despite that admonishment.

- Mr. Patrick Limber of the USGS has stated, **"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.

- The 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. Neither the Hapke and Reid (2007) nor the Hapke, et al. (2006) reports are intended to override comprehensive, detailed site-specific analysis of cliff retreat and annualized retreat rate. In addition, the Hapke and Reid (2007) study explicitly reports a retreat rate uncertainty of 0.2 m/yr (0.656 ft/yr), which is an order of magnitude greater than the bluff retreat rate we have calculated for the Newman property. The use of COSMOS is not justified at the subject site.

Review Comment No.1.4

Based on this preliminary analysis, staff recommends a total setback from the bluff edge of at least 50 feet, along with a special condition requiring the removal or relocation of any portion of the proposed development that becomes threatened by erosion or instability in the future (See additional comments on special conditions below).

Response No. 1.4

The principles discussed herein support the proposed 40-foot setback. However, we understand that in the interest of saving time and money, the property owner will agree to a 51-foot setback from the bluff edge.

Review Comment No. 2

***Basement**: Section 30.34.020(B)(1)(a) of the City's certified IP states, in part "[...] Any new construction shall be specifically designed and constructed such that it could be removed in the event of endangerment [...]." The City's certified LUP Public Safety Element Policy 1.6(f) also contains similar language to require that any new construction be designed and constructed such that it could be removed in the event of endangerment. The Commission has previously prohibited basements on blufftop lots, finding that a basement in such a location is inconsistent with these LCP policies for removal, and others, because 1) it is unlikely that a basement could be specifically designed and constructed such that it could be removed in case of endangerment, 2) once exposed, a basement would essentially serve the same purpose as a shoreline protective device in the same manner that caissons and deepened foundations do, and 3) constructing a basement in a potentially geologically unstable environment such as within a coastal bluff may create adverse impacts on the integrity of the bluff itself if the basement structure were ever required to be removed (See A-6-ENC-16-0067/Meardon, A-6-ENC-16-0060/Martin, A-6-ENC-16-0068/Hurst). Therefore, the city must require that the project be redesigned without the basement in order to find it consistent with the City's LCP.*

Response No. 2

1) In the unlikely event that erosion occurs to the extent that the primary structure is in danger, removing the basement would not significantly decrease slope instability or accelerate the bluff erosion rate. There is no geotechnical reason why the basement could not be removed, provided it is not exposed in the bluff face.

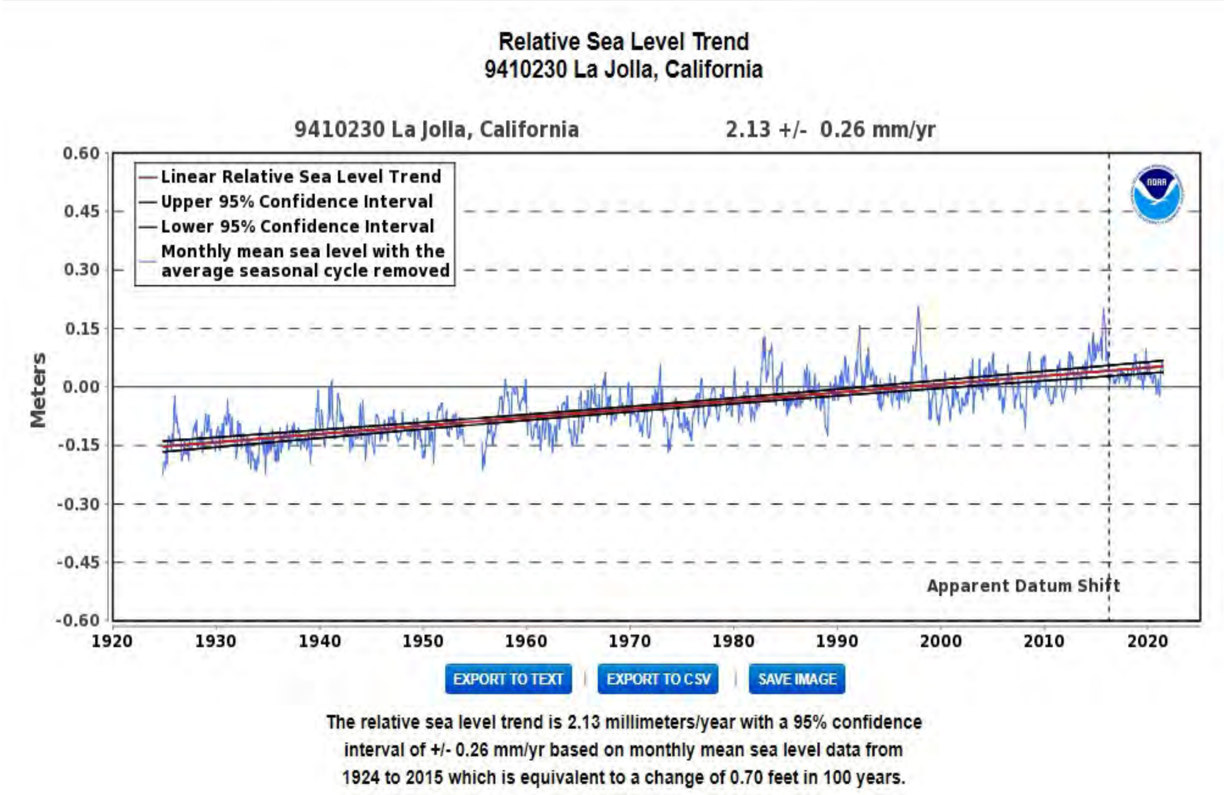
2) The purpose of caissons and deepened foundations is for lateral stability, not shoreline protection.

3) The CCC has approved basements on Neptune Avenue previously.

ACTUAL SEA LEVEL RISE DATA CHECK

The CCC SLR Guidance (CCCSLRG) requires the use of the " best available SLR science." The 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 about 10 years old. The SLR models used as the basis for the COPC and CCCSLRG have been in place for about 2 decades. The accuracy of any model can be determined by comparing the measured SLR (real time data) to the model predicted SLR (model prediction). If the model does not predict, with any accuracy, what has happened in the approximate last two decades, 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.

The National Oceanic and Atmospheric Administration (NOAA) has been measuring SLR globally, and specifically in La Jolla, which is the closest NOAA station to Encinitas. The NOAA La Jolla SLR rate is 2.13 mm/yr as shown in Figure 1.

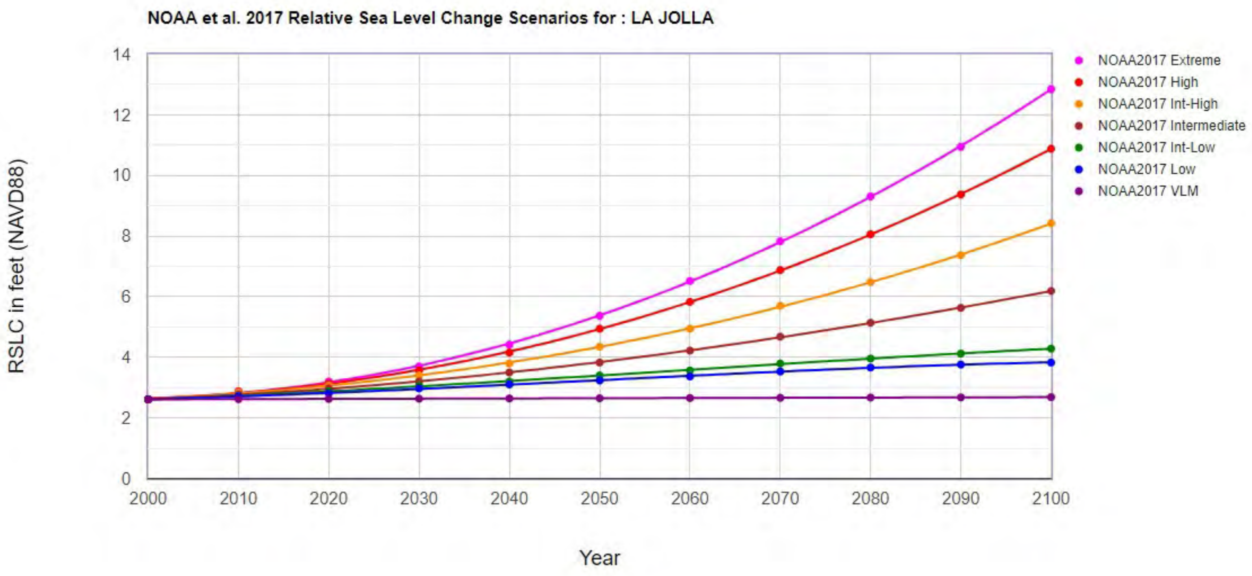

Relative Sea Level Trend
9410230 La Jolla, California

The relative sea level trend is 2.13 millimeters/year with a 95% confidence interval of +/- 0.26 mm/yr based on monthly mean sea level data from 1924 to 2015 which is equivalent to a change of 0.70 feet in 100 years.

Figure 1. Latest measure SLR at La Jolla from NOAA.

The rate can be used to calculate a sea level rise of 46.7 mm (0.154 ft) over the last 22 years (2000 through December 2021). If we assume that the La Jolla rates do not change significantly in the next 8 years (which is likely) the amount of La Jolla SLR to the year 2030 will be about 0.21 feet.

NOAA also provides plots of the most current SLR model projections (best available science) over time starting in the year 2000. Figure 2, is the model projections taken from NOAA, which is more current SLR science than the COPC. To see which model is more accurately predicting SLR, the data for La Jolla can be either plotted onto the curves or estimated from the table below the curves.



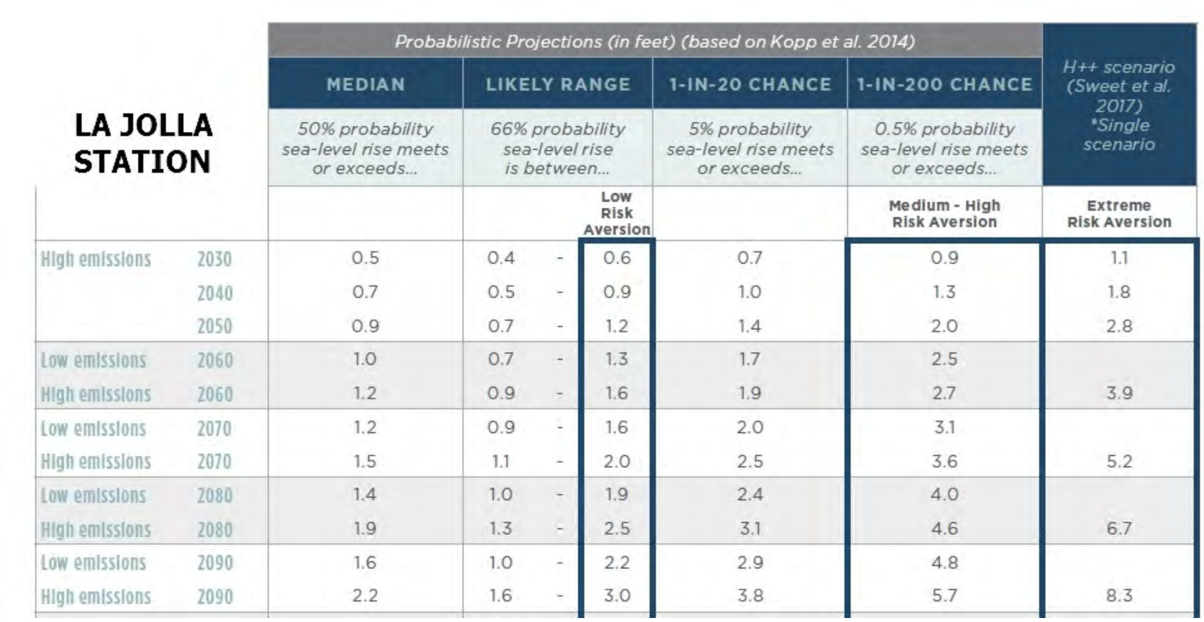
La Jolla SLR
Scenarios for LA JOLLA
NOAA2017 VLM: 0.00072 feet/yr
All values are expressed in feet

Year	NOAA2017 VLM	NOAA2017 Low	NOAA2017 Int-Low	NOAA2017 Intermediate	NOAA2017 Int-High	NOAA2017 High	NOAA2017 Extreme
2000	2.61	2.61	2.61	2.61	2.61	2.61	2.61
2010	2.62	2.71	2.71	2.78	2.84	2.87	2.87
2020	2.63	2.81	2.87	2.97	3.07	3.14	3.20
2030	2.63	2.97	3.07	3.20	3.40	3.56	3.70
2040	2.64	3.10	3.20	3.50	3.79	4.15	4.42
2050	2.65	3.24	3.40	3.83	4.35	4.94	5.37
2060	2.66	3.37	3.56	4.22	4.94	5.83	6.52
2070	2.66	3.53	3.79	4.68	5.70	6.88	7.83
2080	2.67	3.66	3.96	5.14	6.48	8.06	9.31
2090	2.68	3.76	4.12	5.63	7.37	9.37	10.95
2100	2.68	3.83	4.29	6.19	8.42	10.88	12.85

Figure 2. NOAA 2021 SLR projections for La Jolla.

Recognizing that in the year 2000 the SLR zero line is 2.61 feet, and using the current La Jolla SLR data (trends), La Jolla SLR should be (2.61 + 0.21 feet) 2.82 feet in the year 2030. Looking at the table in Figure 2 for the year 2030 (8 years from now) reveals that La Jolla SLR is tracking below the NOAA 2017 "Low" SLR model curve. The NOAA "Low" model predicts a SLR rise total in the year 2100 of about 1.22 feet.

The CCCSLRG document requires that a project designer determine the range of SLR using the "best available science." The California Ocean Protection Council (COPC) update included SLR estimates and probabilities for La Jolla, the closest SLR estimates to Encinitas. Figure 3 provides the March 2018 COPC data (from the Kopp, et al., 2014 report) with the SLR adopted estimates (in feet), and the probabilities of those estimate to meet or exceed the 1991-2009 mean, based upon the best available science. The NOAA SLR information provided above is more current than the CCCSLRG. The checking of the models is the "best available science" for SLR prediction and is required by the CCC to be used.



LA JOLLA STATION		Probabilistic Projections (in feet) (based on Kopp et al. 2014)					H++ scenario (Sweet et al. 2017) *Single scenario
		MEDIAN	LIKELY RANGE		1-IN-20 CHANCE	1-IN-200 CHANCE	
		50% probability sea-level rise meets or exceeds...	66% probability sea-level rise is between...		5% probability sea-level rise meets or exceeds...	0.5% probability sea-level rise meets or exceeds...	
				Low Risk Aversion		Medium - High Risk Aversion	Extreme Risk Aversion
High emissions	2030	0.5	0.4 -	0.6	0.7	0.9	1.1
	2040	0.7	0.5 -	0.9	1.0	1.3	1.8
	2050	0.9	0.7 -	1.2	1.4	2.0	2.8
Low emissions	2060	1.0	0.7 -	1.3	1.7	2.5	
High emissions	2060	1.2	0.9 -	1.6	1.9	2.7	3.9
Low emissions	2070	1.2	0.9 -	1.6	2.0	3.1	
High emissions	2070	1.5	1.1 -	2.0	2.5	3.6	5.2
Low emissions	2080	1.4	1.0 -	1.9	2.4	4.0	
High emissions	2080	1.9	1.3 -	2.5	3.1	4.6	6.7
Low emissions	2090	1.6	1.0 -	2.2	2.9	4.8	
High emissions	2090	2.2	1.6 -	3.0	3.8	5.7	8.3

Figure 3. SLR estimates from the State of California, 2018.

In contrast to the measured SLR at La Jolla, the model the CCC is requiring to be analyzed is the "high" emissions scenario and the 0.5% probability shown in Figure 3. For the year 2030 the CCC required SLR is 0.9 feet, which is over 4 times greater than the 0.209 feet that is being measured. Over the 75- to 100-year life of the development this results in very significant difference between what the SLR the CCC requires and what is currently occurring. The best available science using current SLR data shows that the La Jolla SLR trend is tracking more closely to the "likely range." There is no current SLR science that supports the CCCSLRG estimates. There are current measurements that support the use of a much lower SLR estimate over the 75 year life of the development.

CLOSURE

In summary, the GSI report used the best available science, and was signed and stamped by a California-licensed civil engineer and geologist.

GSI reiterates:

- The proposed project 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).
- 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 residential structure 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.

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.

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JPF/DWS/sh

Attachment: Appendix - Selected References

Distribution: (3) Addressee (3 wet signed)

APPENDIX

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JUNE 9, 2022

SUBJECT: Review of Preliminary Geotechnical Investigation , 216 Neptune Avenue, Encinitas, California

References:

1. Preliminary Geotechnical Investigation, Proposed Residential Development, 216 Neptune Avenue, Encinitas, San Diego County, California, by GeoSoils, Inc., dated April 14, 2020
2. Establishing Development Setbacks from Coastal Bluffs, by Mark Johnsson, Staff Geologist for the California Coastal Commission, dated January 16, 2003.
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4. Proposed Project at 216 Neptune Avenue/Newman, City Case No. CDP-003343-2019, by California Coastal Commission, dated February 3, 2022
5. Geotechnical Response to “Proposed Project at 216 Neptune Avenue/Newman(City Case No. CDP-003343-2019), dated February 2,2022, by California Coastal Commission, by GeoSoils, Inc., dated March 22, 2022

6. Response to Coastal Commission Review of 216 Neptune Coastal Development Permit(City of Encinitas) by Cohn + Associates Architecture Planning, dated May 23, 2022

In response to your request I have reviewed the referenced reports and plans for conformance to the requirements of the City of Encinitas Municipal Code, approved LCP and current California Coastal Commission requirements. I have also visited the subject site.

Based upon my review and site visit the following are my comments and recommendations on the conformance of the geotechnical reports to the requirements of the City of Encinitas Municipal Code and it local coastal plan and comments on the response by Coastal Commission Staff on erosion rates and basement construction along the Encinitas shoreline.

- The Coastal Commission Staff presented a letter report on the proposed project at 216 Neptune Avenue(February 6, 2022) and made several comments regarding site stability and geologic setback. The staff quoted and referenced a report by Benumoff/Griggs(1999) giving erosion rates for Encinitas bluffs. However, based upon statements of the principal writer for the report, Mr. Ben Benumoff, the study has been misused and its data taken out of context. The erosion rate quoted in the study was intended for site specific locations along the Encinitas coastline. It has been made very clear by Mr. Benumoff that erosion rates for coastal bluffs should be site specific and can vary considerably, based upon the soil/bedrock that is present at the bluff.
- The Coastal Commission Staff also referenced the use of the Scape equation(Ashton, et al. 2011) and that the geotechnical consultant, Geosoils, Inc., may have used a lower site specific factor than appropriate and gave examples of differing rates.
- Coastal Staff also referenced USGS CoSMoS bluff retreat tool. This tool has not been adopted by the City of Encinitas and is considered extremely liberal in their estimate of bluff retreat and not reflective of actual bluff retreat.
- Geosoils,Inc. responded to the comments by the Coastal Commission Staff in a report dated March 22, 2022. They presented rebuttals to the

use of Benumoff/Griggs erosion rates, the use of the SCAPE equation of m= 0.5 and the referenced USGS CoSMoS bluff retreat rates. The conclusion reached by Geosoils,Inc. was that the use of the recommended 40 foot setback was valid, based upon actual physical science and the data used by the consultant. Based upon an addition evaluation of the SCAPE equation and comments by the Coastal Commission the geotechnical consultant, GeoSoils, Inc., stated that, although they felt that their evaluation of a setback of 40-feet was still technically valid, a setback of 51-feet should be used as a conservative value utilizing their evaluation of values presented by Coastal Staff.

- Coastal Staff also had an issue with the construction of a basement for this project. For almost 30 years the coastal commission has approved numerous blufftop projects in the City of Encinitas and has allowed the construction of basements. There is no credible evidence or data to justify the argument that coastal staff presents for denying the construction of a basement on a coastal bluff.

Recommendations

Based upon my review of the referenced documents this reviewer finds the recommended setback of 51-feet is in accordance with the requirements of the City of Encinitas Municipal Code and LCP. I approve this recommended setback. This is supported by an approval of an approved setback of 52 feet for a proposed structure in the 100 block of 5th Street, just south of the proposed project. The factor of safety setback and erosion were almost identical to the values used for this project.

The construction of a basement is both technically possible and does not violate any requirements of the City of Encinitas. The City of Encinitas has a long history with the California Coastal Commission of processing and approving coastal bluff property construction with proposed basements. It is technically and physically possible to remove basements if the blufftop structure is endangered. The proposed development is approved, from a geological and geotechnical standpoint and meets the requirements of the City of Encinitas.

James F Knowlton

RCE 55754 CEG 1045

SURFRIDER
FOUNDATION
SAN DIEGO COUNTY

December 9, 2022

Delivered via email

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

Re: W17b - Appeal No. A-6-ENC-22-0059 (Newman, 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. We agree with the grounds for appeal brought forth by Chair Brownsey and Vice Chair Hart, and also support the Staff Report's determination that Substantial Issue should be found.

We reviewed the CDP, and concurred with the Staff Report's findings that the bluff setback rate was not qualified using the additive approach required in Section 30.34.020 (D) of the City of Encinitas' approved LCP. Therefore, this development has NOT been determined to have a factor of safety of 1.5 over a 75 year lifespan as required in Encinitas' LCP.

In addition to the other discrepancies pointed out in the Staff Report, it's worth noting that GeoSoils, Inc. used a flawed bluff retreat rate in their report and failed to account for sea level rise. They calculated an extremely low-end retreat rate of only 1.4 to 7 feet over 75 years, which is notably lower than both actual observed, and scientifically estimated, retreat rates along the Encinitas bluffs. For example, a 2015 Army Corp of Engineers study[1], conducted under national scientific standards for a 50-year Encinitas-Solana Beach Coastal Storm Damage Reduction Project, estimated an average retreat rate of 1 ft/year for the *exact same stretch of bluffs in Encinitas.*

We also concur that the CIty of Encinitas failed to memorialize conditions that the

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[1] https://www.spl.usace.army.mil/Missions/Civil-Works/Projects-Studies/Solana-Encinitas-Shoreline-Study/

proposed structure would not be entitled to future shoreline or bluff protective devices; additionally, removal/relocation of any portion of the proposed structure that becomes threatened by future erosion/bluff instability should have been memorialized as well. As stated in the Staff Report, the City's LCP Section 30.34.020(d) prohibits new development from requiring any future shoreline protection.

Lastly, the project was approved with a basement in contradiction to Section 30.34.020(B)(1) of the City's IP, which requires that new construction is designed to be removable in the event of endangerment. A basement built deep into an eroding coastal bluff is not realistically removable and would create permanent, adverse impacts to the bluff itself. Also, the basement would serve as a de facto seawall if exposed over time due to erosion. Commission Staff is showing consistency on this point, as evidenced by their recommendation to prohibit a similar basement in Encinitas as a special condition for Item W18a, application No. A-6-ENC-20-0022 (Hanlon, Encinitas).

Thank you for the opportunity to provide comments on this project. Surfrider recommends that the Coastal Commission find Substantial Issue with this project.

Sincerely,

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

Mitch Silverstein
Policy Coordinator
San Diego County Chapter, Surfrider Foundation