January 8, 2021 File No. 33348-08

The Cove at El Niguel
c/o Laguna Niguel Properties, Inc.
27422 Portola Parkway, Suite 300
Foothill Ranch, CA 92610

Attention: Ms. Deborah Hon
Mr. Brian Diaz / Recupero and Associates, Inc.

Subject: **GEOTECHNICAL REVIEW OF TENTATIVE TRACT MAP**
Tentative Tract No. 17721
The Cove at El Niguel
30667 Crown Valley Parkway
Laguna Niguel, California

References: See Appendix A

Dear Ms. Hon:

American Geotechnical, Inc. (AG) is pleased to present this Geotechnical Review of Tentative Tract Map report addressing the proposed redevelopment of The Cove at El Niguel (formerly known as Crowne Cove) residential properties located at 30667 Crown Valley Parkway in the City of Laguna Niguel, California (the Site). The purpose of our work was to review the proposed site development plan for Tentative Tract No. 17721 prepared by Hunsaker & Associates deated December 9, 2020, review previously approved preliminary geotechnical investigation report and response report pertaining to the previous site development plans, and various geotechnical reports for the adjacent properties prepared by both our office and other consultants, revisit the site and perform inclinometers and piezometer readings to evaluate the current site conditions and groundwater conditions beneath the site, evaluate the relationship of geotechnical conditions to the proposed redevelopment, document our findings and conclusions, and formulate recommendations for grading and construction.

The latest Tentative Tract Map (TTM) for Tentative Tract No. 17721 has been prepared by the project civil engineer, Hunsaker & Associates, dated December 9, 2020. The project consists of development of six (6) triplex and two (2) duplex buildings for a total of 22 units on Lot 1 of the Site. Buildings 1, 2, 4, 5, 6, and 8 are triplex buildings and Buildings 3 and 7 are duplex buildings. The TTM site development plan has been reviewed and evaluated by our office. Geotechnical opinions and recommendations are formulated as set forth in this report. The TTM site development plan was used for the base of our updated Geotechnical Map (**Plate 1**), upon which the findings of our field investigation are presented.

It is important to point out that American Geotechnical (AG) was the Engineer of Record during completion of the late 1998 - 2000 mass grading operations and structural restraint system installation conducted at the Site and to the west, associated with the stabilization/repair of the March 19, 1998 Via Estoril Landslide (also known as the Niguel Summit Landslide). The landslide catastrophically destroyed part of a former 41-unit condominium development on the site as well as nine (9) single-family tract homes in the project above. That entire 41-unit project was demolished/sacrificed in its entirety in order to perform the grading needed to stabilize the former landslide. Our prior involvement included geotechnical and structural evaluations and design work along with geotechnical and structural observations and testing through completion of the landslide stabilization. As such, we feel our professionals are in the best position to evaluate the proposed project and provide recommendations for proceeding with the new project.

Based on our review of the background documents with regard to the geologic and geotechnical site conditions, our subsurface investigations, laboratory testing, geologic and engineering analyses, as well as our slope stability analyses, it is our opinion that the proposed The Cove at El Niguel redevelopment project is feasible from a geotechnical standpoint. As is commonly the case, the site does possess some adverse geotechnical/geologic conditions that will require special attention before, during, and after construction. Of primary concern is that the proposed redevelopment be constructed along the lower east side of the Site (Lot 1) beyond the limits of the past landslide. The landslide was remediated between 1998 and 2000. The geotechnical remediation was designed and overseen by professionals from our office. The currently proposed construction includes relatively minor grading and construction to accommodate the proposed plan. Structures are proposed along the lower east side of the site beyond the limits of the past landslide grading. No development is proposed in the higher elevation area, hillside portion of the property (Lot “A”). Our analyses indicate that the landslide area was stabilized by past grading activities and is suitable to accommodate the plan as currently proposed. This conclusion is contingent upon adopting the recommendations in this report and incorporating these recommendations into construction.

We appreciate the opportunity to be of service. Should you have any questions regarding the information provided herein, please do not hesitate to contact this office. When additional plans become available, they should be forwarded to this office for review and comment.

Respectfully Submitted,

AMERICAN GEOTECHNICAL, INC.

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Enclosures:
Figure 1 – Site Location Map
Figure 2 – Mat Foundation Detail

Plate 1 – Preliminary Geologic Map
Plate 2 – Geotechnical Cross Sections DR-DR', E-E', F-F', G-G', H-H' & I-I'
Plate 3 – Geotechnical Cross Section J-J'
Plate 4 – Retaining Wall Design Criteria (TTM Section B-B')
Plate 5 – Retaining Wall Design Criteria (TTM Section I-I')
Plate 6 – Retaining Wall Design Criteria (TTM Section K-K')
Plate 7 – MSE Wall Design Criteria (TTM Section H-H')
Plate 8 – MSE Wall Design Criteria (TTM Section J-J')
Plate 9 – MSE Wall Design Criteria (TTM Section L-L')

Appendix A – References
Appendix B – City of Laguna Niguel Geotechnical Review Sheet and Notice of Incompleteness
Appendix C – Slope Inclinometer Plots
Appendix D – Piezometer Readings
Appendix E – Existing Storm Drain Inspection Report
Appendix F – Probabilistic Seismic Hazard Analysis
Appendix G – Results of Slope Stability Analyses
Appendix H – Stability Analysis of 15.5-Foot High Mechanically Stabilized Earth (MSE) Wall
Appendix I – Standard Guidelines for Grading Projects

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wpdata/OC/33348-08 – Geotechnical Review of Site Development Plan – JH GWA DS 1-8-2021 DL

1.0 INTRODUCTION

1.1 PURPOSE

This report presents the results of our geotechnical review of Tentative Tract Map (TTM), Tentative Tract No. 17721 for the proposed The Cove at El Niguel (formerly known as Crowne Cove) redevelopment project located at 30667 Crown Valley Parkway in the City of Laguna Niguel, California (see **Figure 1**, Site Location Map). The purpose of this work is to review and evaluate the current site development plan with respect to previous geologic and geotechnical work completed on site, and formulate recommendations for grading and construction.

1.2 SCOPE OF SERVICES

The scope of work performed for this investigation included the following:

- Review previous subsurface investigation reports for the Site and adjacent areas prepared by AG and other consulting firms. A list of report references is presented in **Appendix A**;
- Review the City of Laguna Niguel Geotechnical Review Sheet dated November 6, 2013 and Notice of Incompleteness for Tentative Map TT 17721, Site Development Permit SP 16-04 (Crowne Cove) dated October 26, 2016 and comply with review comments. The November 6, 2013 Geotechnical Review Sheet and the October 26, 2016 Notice of Incompleteness are attached in **Appendix B** for easy reference;
- Review and analyze Tentative Tract Map (TTM), Tentative Tract 17721, dated December 9, 2020, prepared by Hunsaker & Associates, which proposes to develop 8 buildings (6 triplex and 2 duplex buildings) for a total of 22 units of condominimum on Lot 1 at the Site;
- Conduct inclinometer readings in three (3) existing inclinometer casings (AGI-26, AGI-27, and AGI-31) (**Appendix C**) and measure groundwater levels in existing seven (7) piezometers (AGP-9, AGP-10, AGPZ-1 to AGPZ-5) within the project boundary (**Appendix D**);
- Review the existing storm drain video scoping and report by others (**Appendix E**);
- Perform probabilistic seismic hazard analysis using U.S. Geological Survey (USGS) Unified Hazrd Tool to estimate probabilistic seismic hazard conditions at the site (**Appendix F**)
- Prepare an updated Geotechnical/Geologic Map utilizing data gathered for previous reports and the December 9, 2020 TTM by Hunsaker & Associates as a base map;

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TITLE: SITE LOCATION MAP THE COVE AT EL NIGUEL		
SCALE: N.T.S	DATE: JAN 2021	FILE NO.: 33348-08

FIGURE 1

- Update seven (7) geologic cross-sections (DR-DR’, E-E’, F-F’, G-G’, H-H’, I-I’, and J-J’) completed for previous reports to illustrate the proposed grading and redevelopment conditions from the December 9, 2020 TTM by Hunsaker and Associates;
- Conduct updated slope stability analyses utilizing Cross-Sections DR-DR’ to evaluate the stability of the Site for the proposed redevelopment (**Appendix G**);
- Perform stability analyses of a 15.5-foot high Mechanically Stabilized Earth (MSE) wall at the west perimeter of the redevelopment area. (**Appendix H**);
- Analyze all resultant engineering and geologic data to develop grading, foundation, retaining walls, MSE walls, and other soil-related design parameters for the proposed redevelopment;
- Prepare this report summarizing the findings and conclusions of our investigation, and present site-specific geotechnical/geologic recommendations for consideration during grading and construction.

2.0 HISTORICAL DEVELOPMENT SUMMARY

The Cove at El Niguel (formerly known as Crowne Cove) site has a somewhat complex geologic/ geotechnical and development history that was described in detail in the previous AG reports (see **Appendix A**). A brief summary of the historical development follows.

The site was originally developed in circa 1979 as a 10-building townhome project that included 41 individual condominiums. Distress to the development appears to have started around 1986. Initial distress was not recognized as landslide related. In 1998 the Via Estoril Landslide (also known as Niguel Summit Landslide) occurred. The slide involved the upslope area of the original development that encroached on the original development, and destroyed or damaged multiple buildings. The landslide was repaired between 1998 and 2000 and involved installation of a caisson wall with tieback anchors, removal of Crowne Cove condominium buildings and associated structures, partial removal of the landslide mass, installation of subdrains, and construction of a compacted fill buttress.

Since the Via Estoril Landslide repair, AG has prepared two Geotechnical Feasibility Study reports, dated February 16, 2012 and April 18, 2012 for the proposed Crowne Cove Trumark 41-Unit Redevelopment Plan and Crowne Cove 40-unit Redevelopment Plan, respectively. The City of Laguna Niguel's geotechnical reviewer, GMU Geotechnical (GMU) issued a Geotechnical Review Sheet dated August 28, 2012 providing comments on planning feasibility-related issues and concerns. Later in 2013, AG had completed geotechnical investigations for two different redevelopment plans proposing a 38-Unit Condominimum Complex and Optional 16-Unit Single Family Residence Development, respectively, and prepared a Preliminary Geotechnical Investigation report dated July 17, 2013 in accordance with the comments on August 28, 2012 Geotechnical Review Sheet. The City's geotechnical reviewer, GMU, issued another Geotechnical Review Sheet dated August 23, 2013 providing a few additional comments in addition to the previous August 28, 2012 Geotechnical Review Sheet. AG subsequently prepared a response report dated September 27, 2013 to address the August 23, 2013 Geotechnical Review Sheet. In addition, per GMU's request, we prepared a letter commenting on the inclinometer AGI-31 dated October 23, 2013. After issuing our October 23, 2013 letter, a Geotechnical Review Sheet dated November 6, 2013 was issued by the City of Laguna Niguel with Conditional Approval of Document Submitted. There were 7 numbered conditions of approval in the November 6, 2013 Geotechnical Review Sheet, which are repeated below in *Italic* font:

1. *A grading plan review report will be required.*

2. *The report should contain all necessary recommendations for proposed grading, wall and foundation systems as well as all necessary technical calculations and justification.*

3. *All available slope inclinometers will need to be re-read and evaluated. The evaluation shall be contained in the grading plan review report. Inclinometer AGI-31 should be specifically addressed.*

4. *The existing fill at the back of the development should be evaluated relative to the proposed grading and improvements.*

5. *The report should contain specific recommendations for pre-construction surveys and monitoring during construction.*

6. *Maintenance recommendations (i.e., for surface drainage, subsurface drains, streets, utilities, etc.) should be provided within the evaluation report and under separate cover for later distribution to homeowners and/or HOA.*

7. *Full time observation and testing will be required during rough grading operations.*

Since then, the proposed site development plans have been revised/updated by the concerned parties in accordance with the City's planning comments. In 2016, the City of Laguna Niguel Community Development Department issued a Notice of Incompleteness for Tentative Tract 17721, Site Development Permit SP 16-04 (Crowne Cove), dated October 26, 2016. Page 2 of the October 26, 2016 Notice of Incompleteness includes a list of incomplete items. The first comment on the list includes geotechnical issues and is repeated below in *Italic* font:

1. *The Geotechnical Consultant should review the current plans and provide an update letter. The letter should include, at a minimum:*

   a. *Discussion of current site conditions;*

   b. *Current monitoring, including slope inclinometers, piezometers, subdrain video discussion, and results of recent visual observation of surface drainage facilities (i.e., terrace and down drains). These results should include offsite monitoring devices/drains that may impact the proposed development;*

   c. *Detailed discussion of slope stability, including the existing gravity buttress. Both temporary and gross stability should be discussed, and calculations provided as necessary;*

   d. *Discussion of the proposed development's geotechnical impacts on the site and adjacent properties.*

In the meantime, AG reviewed a Precise Site Development Plan prepared by Michael Baker International of Irvine and prepared a geotechnical review of the Precise Site Development Plan report dated December

9, 2016. All of the items listed in the City's October 26, 2016 Notice of Incompleteness have been addressed in detail and presented in our December 9, 2016 report.

The planning process and site redevelopment design continued by the concerned parties in 2017 and 2018. The City of Laguna Niguel issued a subsequent Notice of Incompleteness for Tentative Map TT 17721, Site Development Permit SP 16-04 (Crowne Cove), dated September 29, 2017 (3rd Notice) and September 6, 2018 (4th Notice), respectively. No geotechnical-related comments or incomplete items are listed in these two Notices of Incompleteness. A copy of the City of Laguna Niguel Geotechnical Review Sheet dated August 23, 2013 and Notice of Incompleteness dated October 26, 2016 are included in **Appendix B** for easy reference.

Because the basic geologic and geotechnical conditions relating to the Site have been established and conditionally approved, we do not intend to repeat those conditions here in this report, except as they directly relate to the new proposed redevelopment plan. Boring logs and other data are included in our previous reports as listed in **Appendix A**.

2.1 DESCRIPTION OF EXISTING SITE CONDITIONS

The subject site lies directly west of Crown Valley Parkway in the City of Laguna Niguel, California (see **Figure 1**, Site Location Map), approximately 40 vertical feet above the distal Arroyo Salado Valley (i.e., Salt Creek). From a regional standpoint, the parcel lies within the southwestern portion of the San Juan Capistrano 7.5-Minute Topographic Quadrangle Map (USGS, 1968, photo revised 1981). Elevations along Crown Valley Parkway are on the order of 360 feet AMSL. A 2:1 (horizontal:vertical) fill slope on the order of 10 feet in height occurs along the east property boundary above Crown Valley Parkway. This slope ascends to a relatively flat-lying, nearly rectangular-shaped pad area comprising the east area of the property (Lot 1). This pad area has elevations of around 370 feet AMSL on the east and 380 feet on the west. An east-facing fill slope of varying steepness extends on the order of around 160 feet in height. It ascends from the western edge of the lower pad to another gently sloping surface. This upper surface extends between elevation 440 on the east to the western property boundary at around 450 feet in elevation AMSL.

2.2 PROPOSED REDEVELOPMENT

The following discussion is based on the 30-scale, Tentative Tract Map, entitled "*The Cove at El Niguel, 30667 Crown Valley Parkway, Laguna Niguel, Tentative Tract No. 17721*," prepared by Hunsaker &

Associates, dated December 9, 2020. The TTM includes five sheets, Sheets TTM-1, SP-1, SP-2, PG-1, and PU-1. Sheet TTM-1 is a Tentative Tract No. 17721, Sheet SP-1 is a Site Plan, Sheet SP-2 is a Site Plan Site Sections, Sheet PG-1 is a Preliminary Grading Plan, and Sheet PU-1 is a Preliminary Utility Plan.

The property at the site has Assessor Parcel Number (APN) 656-231-02 and Tentative Tract No. 17721 is for condominium purposes. There are two lots within the subject property, Lot 1 and Lot “A” (open space). Lot 1 includes 2.0 acres of residential area and Lot “A” includes 2.2 acres of open space. The gross area of the property is 4.2 acres (Lot 1 and Lot “A”). Existing land use at the site is a vacant sloped property. The site is bounded on the south by multi-family residential properties, on the west by open space/single-family residential properties, on the north by single-family residential properties, and on the east by Crown Valley Parkway.

It is proposed to redevelop Lot 1 of Tentative Tract No. 17721. Lot 1 includes 2.0 acres of residential buildable acres where buildable acres are defined as those areas not including slopes and easements. Lot “A” includes 2.2 acres of open space with existing easements to be remained.

Proposed construction will include about 19,960 cubic yards of cut and 19,830 cubic yards of fill (long 130 cubic yards) to create 8 buildable pads, an entrance street (Playa Blanca) from Crown Valley Parkway, and 2 interior streets (Private Drive “A” and Private Drive “B”). Presently, no import or export soil is anticipated. The excess 130 cubic yards will likely be consumed via compaction-related soil shrinkage. Finished pad elevations will vary from 378.38 feet at Building 1 to 381.5 feet at Building 5. Pads will include Mechanically Stablized Earth (MSE) walls up to 15.5 feet tall (0 to 15.5 feet) along the west perimeter of Lot 1 and up to 6 feet high (3.5 to 6 feet) along the east perimeter of Lot 1, respectively. On the north perimeter of Lot 1, a two-teir retaining wall is proposed. The upper tier retaining wall is up to 5 feet (0 to 5 feet) high and the lower tier retaining wall is up to 6 feet (3.5 to 6 feet) high.

Up to 6-feet high radiant heat walls with or without retaining walls (up to 4.3 feet high) are also proposed surrounding Buildings 4 and 5 located on the south portion of Lot 1. An up to 6.5-foot high (3.5 to 6.5 feet) retaining wall is also proposed on the west side of Building 5. An up to 2-foot high (0 to 2 feet) retaining wall is proposed to be constructed along the 15-foot wide access road located on the southeast side of Lot “A” adjacent to the proposed MSE walls along the west perimeter of Lot 1. All of the proposed slope will have a slope ratio of 2:1 (horizontal:vertical). Additional work will include relocating the existing storm drain, removing and/or relocating eastments on Lot 1 and Lot “A”.

It is proposed to construct a two-story, multi-family building on each graded pad to house 22 separate condominium units at 8 buildings (6 triplex and 2 duplex).

The perimeter MSE walls bounding the west margin of the building pads and the 2:1 (horizontal:vertical) cut slope at the southwest margin of the building pads will be located at the toe of the compacted fill buttress built to stabilize the Via Estoril Landslide. MSE walls have been advised for these areas. Other portions of the proposed redevelopment are located east and outside of the landslide repair.

3.0 FIELD INVESTIGATION

3.1 SLOPE INCLINOMETER READINGS

On December 15, 2020, we read in inclinometers AG-26, AG-27, and AG-31 located within the Site. The locations of the inclinometers are shown on the Geotechnical/Geologic Map, **Plate 1**. Cumulative and incremental displacement plots for each inclinometer are presented in **Appendix C**.

Inclinometer AGI-26 was first measured on March 28, 2001. Maximum apparent cumulative displacement at AGI-26 location since 2001 readings is under 1 inch and reflects a combination of minor systematic instrument drift, very small movement of borehole backfill, and/or insignaifacnt creep over the monitoring period.

Inclinometer AGI-27 was first measured on May 2, 2002. Maximum apparent cumulative displacement at the AGI-27 location since the 2002 readings is slightly more than 2 inches. Maximum apparent cumulative displacement at the AGI-27 location from the previous 2016 to current 2020 readings is slightly more than 1 inch. The rate of movement at the AGI-27 location from 2016 to 2020 is about 0.25 inches per year and the overall rate of movement since the May 2, 2002 baseline readings is about 0.11 inchs per year. The rate of movement at the AGI-27 location reflects a typical creep movement of the highly expansive 1979 fill.

Inclinometer AGI-31 reflecting largely offsite conditions upslope was first measured on May 2, 2002. Maximum apparent cumulative displacement at the AGI-31 location since the 2002 readings is about 0.8 inches over a depth of 15 feet below ground surface (bgs) in the direction of the positive A-axis with a trend of S48E. Maximum apparent cumulative displacement at a depth of 57 feet bgs is about 0.6 inches. The incremental displacement plot shows a spike at a depth of about 57 feet bgs visible in both the positive A and B directions. Maximum incremental displacement is about 0.4 inch in the positive A direction and under 0.1 inch in the positive B direction. Incremental displacement at this depth (i.e., 57 feet bgs) has increased uniformly over the roughly 17.5 years since the baseline reading on May 2, 2002. We believe the small magnitude of cumulative and incremental displacements, the relative uniformity of incremental displacements over time, and the general shape of the displacement plots at the AGI-31 location indicate the movement at a depth of 57 feet may be related to a casing anomaly created during installation combined with minor instrument drift. Actual land movement near a depth of 57 feet cannot be ruled out, however, the magnitude of the total movement is very small, consistent with very slow creep which can be expected in any hillside area consisting of clayey earth materials. At a creep rate of only about 0.03 to 0.05

inches per year, no unusual behavior is indicated. Similar, at-depth creep movement was not observed in any of the further downslope inclinometers.

As can be seen from the inclinometer plots for AGI-26, AGI-27, and AGI-31 located within the subject property, no significant movement was detected when compared to previous readings. When minor movement has dictated, the rates of movement have been found to be of low magnitude and incredibly slow. Overall, minor movements detected are consistent with the landslide mass coming to equilibrium, mobilization of capacity within the buttress mass, and ordinary creep.

Based on the inclinometer plots, it is our opinion that no significant movement will occur during the construction life and only typical slope creep influence will continue over time. Our recommendations with regard to the slope creep influence are presented in the later section of this report.

3.2 PIEZOMETER READINGS

On December 15, 2020 we made readings in seven piezometer casings located on site, AGP-9, AGP-10, and AGPZ-1 to AGPZ-5. The piezometer readings are presented in **Appendix D**.

The long-term trend in groundwater elevation is relatively stable with no significant fluctuation. This is likely due to a combination of factors including the lack of nearby residential irrigation because of the landslide repair, reduced rainfall from the extended state wide drought, and proper functioning of surface and subsurface drainage devices constructed during the landslide repair. The in-place subdrain systems associated with the late 1990s mass grading remediation appear to be as expected and performing adequately.

3.3 EXISTING STORM DRAIN INSPECTION

The existing storm drain has been field verified by another consultant. Our review of the result of the storm drain video scoping indicated that all the storm drain and outlets are functioning well. Where observed, the storm drain joints appear to be in good condition; in other words, no signs of soil movement and no significant leakage were noted. Minor debris was noted in the 42-inch RCP which prevents a complete inspection. The debris could be jetted out to allow for a complete survey. Because most of the existing storm drains will be either removed or relocated in conjunction with the planned redevelopment, as such, it is not essential to redo the storm drain video scoping for a complete survey. The results of the storm drain video scoping report performed by Houston & Harris PCS, Inc., dated December 28, 2020 is included in

Appendix E. Furthermore, as mentioned earlier, monitored groundwater levels are consistent with properly functioning systems. The most significant conclusion drawn from the stormdrain videos from a geotechnical perspective is that no soil movement is indicated.

3.4 VISUAL OBSERVATION OF EXISTING SURFACE DRAINAGE FACILITIES

Visual observation of existing surface drainage facilities (e.g., terrace and down drains) have been conducted by other consultants. Similar to the existing storm drain conditions, the existing terrace and down drains are in good condition. No breaks or significant movement were noted at. As we understand, the property owner had a landscape gardener perform periodic maintenance and cleaning services to the surface drainage facilities and provide trimming services of onsite vegetation to prevent landscape debris (e.g., tree leaves, branches, etc.) from blocking the surface drainage facilities.

4.0 GEOLOGY

Geologic conditions on site and in the general area were discussed at length in our previous reports. The areal distribution of earth materials is shown on the Geotechnical/Geologic Map, **Plate 1**. Geologic cross-sections DR-DR’, E-E’, F-F’, G-G’, H-H’, I-I’, and J-J’ show interpreted subsurface conditions including the proposed site development and recommended removals are shown on Geotechnical Cross Sections, **Plates 2 and 3**, respectively.

4.1 ON-SITE GEOLOGIC UNITS

The bedrock formations, surface deposit units, and two distinct generations of artificial fill are each described in more detail below, listed in age from oldest to youngest. Bedrock units include the Middle-Miocene age San Onofre Breccia (Tso) and Monterey Formations (Tm), Undifferentiated Tso/Tm, the late Miocene to Pliocene age Capistrano Formation (Tc), Older Quaternary (Qlso) and Recent Landslides (Qlsr), and deposits of Engineered Fill (Ef1 and Ef2).

4.1.1 Bedrock

San Onofre Breccia (Map Symbol Tso): Regional maps depict the San Onofre Breccia as occurring west of a north-trending fault (CDMG, 1974) which transects the southwestern corner of the site (see **Plate 1**). Site-specific geotechnical reports revealed this unit previously outcropped along the original slope abutting the western site margin (Geosoils, Inc., 1979). Regional maps indicate bedding as striking northwesterly and dipping moderately to the northeast. The unit was not encountered in-situ within any of our borings. Indirect evidence supporting its westerly occurrence is interpreted based on the presence of breccia-type bedrock lithology encountered within the body of the residual Via Estoril Landslide deposit, penetrated in our boring AGLB-2. Its presence within this slide block is expected given its offsite upslope origin. Approximate structural relationships are depicted within our cross sections (**Plates 2 and 3**). The cross-sections are based on a compilation of available geologic mapping by other consultants (**Plate 1**).

Within AGLB-2 the unit consisted of a general crudely structured section of light gray to medium yellow brown to blue-gray sandstone, local gray claystone, and interbedded tan to light brown pebble to cobble breccia with clasts mostly composed of quartzite, quartz-schist, blue-green schist and gabbro. The sandstones and breccia are medium hard to hard and massive to crudely-bedded, while the clay stones are soft, plastic, sheared, and damp. Where breccia lithology was found to interfinger with the Monterey Formation on the site the units are grouped together as a single undifferentiated unit.

<u>Monterey Formation (Map Symbol Tm):</u> Regional maps depict the Monterey Formation as occurring generally east of the CDMG fault trace which transects the southwesterly site corner (see **Plate 1**). It is designated as undifferentiated with the San Onofre Breccia within the central portion of the site and as a stand-alone formation on the east where bounded between two fault traces (Geosoils, 1979; 2R Engineering, 1976). The structure of the unit is known from previous subsurface exploration on the site (Duco, 1976; Geosoils, 1979) and earthwork grading completed on the adjacent tract to the north (2R Engineering, 1977). This foundation tends to exhibit a moderate to severe degree of folding and variable bedding orientations, likely the result of its high ductility and greater influence by tectonic movement along the paleo fault system. The overall strike of fold axes and bedding is to the north with dips ranging from west to east. In closer proximity to faults, bedding tends to exhibit very high angles of dip and local overturning. Approximate structural relationships are depicted within our cross sections (see **Plates 2 and 3)**. The cross-sections are based on a compilation of available geologic mapping by other consultants (**Plate 1**).

Bedrock assigned to the Monterey Formation was encountered within our large-diameter exploratory boring AGLB-3 and small-diameter hollow-stem auger borings AGPZ-1, -2, and -5. The formation generally consists of a yellowish-gray to olive brown thinly bedded diatomaceous siltstone to shale that is soft to moderately firm and locally indurate. Where recovered by investigation sampling techniques, the locally diatomaceous nature of materials is indicated by low dry unit weight. Where breccia clasts interfinger with the San Onofre Breccia on the site the units are grouped together as a single undifferentiated unit.

<u>Undifferentiated San Onofre Breccia/Monterey Formation (Map Symbol Tso/Tm):</u> For the purpose of this report we assigned a fault-bounded area of bedrock to this undifferentiated formational category. The designation is made based on observation within our borings, where bedrock lithology, typical of these units, was noted to occur in an alternating and conformable interfingering stratigraphic relationship.

<u>Capistrano Formation (Map Symbol Tc):</u> Regional geologic maps depict the Capistrano Formation within areas to the north and south of the site. Its close proximity/relationship of the Monterey Formation within the area suggests that the contact between these two units is nearby. More detailed consultant investigations and grading reports depict this unit as occurring in fault contact with the Monterey Formation along a north-trending fault buried beneath the extreme easterly margin of the site.

Within the Capistrano Embayment, the Capistrano formation commonly overlies the Monterey Formation in stratigraphic Para conformity. In some cases, it was very difficult to assign a formational name to one rock type of another where lithologies were quite similar. In these cases, a conclusive classification was not possible visually. However, when extending map data onto the site from the adjacent tract on the north, it was possible to assign formational names to these units with a higher degree of confidence.

The Capistrano Formation is interpreted to cross only a small portion of the northwestern site corner. Based on an extrapolation of this unit from the north, we did not encounter it in any of our borings. From consultant and published reports, this unit generally consists of a medium olive-brown to dark grey clayey siltstone to silty claystone and local fine-to-medium grained sandstone. Deeper unoxidized parts of the formation tend to be black in color and omit a petroliferous odor. The rock is moderately firm, locally diatomaceous, and poorly bedded to massive. Available consultants' maps indicate a strike of bedding that is variable with dips that are typically shallow and gently towards the fault west.

4.1.2 Surficial Deposits

Older Quaternary Landslide Deposits (Map Symbol Qlso): Regional maps published in 1974 indicate the presence of several larger landslides within distal areas upslope from the site on the west as occurring within and underlain by the San Onofre Breccia. These slides were addressed long ago during development of the adjacent tract on the west. Of interest is that no slides were mapped within the area of the future 1998 Via Estoril slide. Through investigations conducted in association with the Via Estoril slide the presence of an older existing landslide surface, with a deeper rupture surface than that of the Via Estoril slide, was discovered. Minor movement along this deeper plane was inferred from slope inclinometer data within the area offsite to the northwest. The structural configuration of the deeper rupture surface was constrained and its parameters incorporated into remedial grading design for stabilization of the 1998 slide. The structural contours of this surface are depicted on **Plate 1**.

Recent Landslide Deposits (Map Symbol Qlsr): Recent landslide deposits associated with the Via Estoril Landslide lie buried beneath deposits of engineered fill along the westerly margins of the site (**Plate 1**). Where encountered in our boring AGLB-2, the slide deposits left in place following grading consist of competent bedrock derived from the San Onofre Breccia. The buried limits of the slide are depicted on **Plate 1** as a series of large gray dots. Also depicted on **Plate 1**, are structural contours along the base of the slide (prior to grading) established based on findings of the 1998 landslide investigation. Where encountered in AGLB-2, the landslide nature of the material was understood from the site history but absent from a historical perspective, these materials would have been difficult to identify as a landslide. The materials had the appearance of intact bedrock, apparently due to translational block sliding of a significant portion of the landslide mass. This finding is particularly significant because the finding greatly supports the conclusion that there is little or no settlement risk associated with the existence of the stabilized landslide material.

Engineered Fill Deposits (Map Symbol Ef1): Engineered fill deposits classified as Ef1 were placed during development of the original townhome development at the site in 1979. These fills were placed under the observation and testing of Geosoils, Inc., as documented in their report of final grading. The original limits of this fill were modified by subsequent grading associated with stabilization of the Via Estoril Landslide. In general, deposits of this fill remain in their original configuration within the eastern areas of the property, including the fill slope descending to Crown Valley Parkway (**Plate 1**). The fill in this area was originally placed to a maximum depth of 14 feet, not including removals. The deeper fills were concentrated toward the southeasterly corner of the property in the vicinity of the former tributary channel.

The Ef1 deposits were encountered along the eastern area of the site within our large-diameter borings AGLB-3 and -5 and small-diameter borings AGPZ-1, -2, and -5. The fill was reportedly derived from mixed on-site sources of surficial, bedrock and older undocumented fill units. The fill was found to thicken from around 5 feet on the north to 25 feet near the southeastern corner of the site. The differences in fill thickness can be explained by the presence of the tributary channels and deposits of older alluvium which are deeper on the south and required greater depths of removal, and the near-surface presence of bedrock on the north that required less removal. The approximate location of the existing buried alluvium/bedrock contact is noted on **Plate 1**. The approximate limits of the fill are also depicted on **Plate 1**. The approximate subsurface distribution of the fill is noted on our cross sections (**Plates 2 and 3**). As encountered in our borings, the fill consists of variable lifts of clay, silty clay, and gravelly sands that were found to be very competent, uniformly moist, and dense/stiff.

Engineered Fill Deposits (Map Symbol Ef2): Engineered fill deposits designated as Ef2 were placed during remedial grading for the Via Estoril Landslide. These fills were placed under the observation and testing of American Geotechnical, Inc., as documented in our report of the final landslide repair, dated 2000. Locally, low density and very high moisture indicate the diatomaceous nature of some fill. As depicted on **Plates 1 and 2** the eastern limits of this fill are generally coincident with the toe of an existing east-facing fill slope and a large gravity buttress. The western fill limits extend offsite into the Niguel

Summit property and abut the east side of Via Estoril. The north and south fill limits generally follow the boundaries of the site. The thickest fill on the site occurs within the area of the gravity buttress along the western site margin where it extends up to around 75 feet below existing grades.

The Ef2 fill deposits were encountered within the central and west areas of the site within our large diameter borings AGLB-1, -2 and -4 and small-diameter borings AGPZ-3 and -4. The fill was derived from both on-site and off-site/imported sources. The approximate location of the existing buried alluvium/bedrock contact is noted on **Plates 1 and 2**. The approximate limits of the fill are also depicted on **Plate 1**. The approximate subsurface distribution of the fill is noted on our cross sections (**Plates 2 and 3**). As encountered in our borings, the fill consists of variable lifts of clay, silty clay and gravelly sands that were found to be very competent, uniformly moist and dense/stiff. As aforementioned, some areas of lower density and very high moisture were interpreted to represent diatomaceous soil. These fill materials typically possess highly expansive and highly corrosive charactertics.

Engineered Fill Deposits (Map Symbol Ef): Engineered fill deposits classified as Ef represent fill occurring beyond the limits of the site that were placed as part of large offsite grading operations not reviewed in detail as part of this investigation. The general limits of this fill are noted on **Plate 1**. Although not specifically investigated due to their offsite location, these materials would be expected to be generally similar to those fill materials encountered on site.

4.2 GEOLOGIC HAZARDS

The potential impact of certain geologic hazards was assessed as part of our investigation to determine any threats they may pose to proposed development of the site. A discussion of these hazards is summarized below.

Tsunamis and Seiches: There are no large bodies of water located within close proximity to the site such as an ocean or lake that could pose an inundation hazard to the development in the form of a tsunami or seiche waves.

Based on the above, the potential threat to the site from this hazard is considered nil.

Surface Rupture: No known active faults are mapped as crossing the site or surrounding regions in close proximity to the site. Nor is the site and surrounding region depicted on any current Alquist-Priolo Earthquake Fault Zone Maps issued by the State of California. The nearest active fault to the site lies around 4.3 miles to the west, consisting of the Newport-Inglewood Structural Zone, a northwesterly trending strike-slip fault that crosses the western onshore margin of the Los Angeles Basin and extends offshore into the Continental Borderland.

Based on the above, the potential threat to the site from a surface rupture hazard is considered low.

Ground Shaking: Due to its location along the boundary between two global tectonic plates, the southern California region contains a wide variety of active faults. As a result, the occurrence of moderate and larger sized earthquakes is a common phenomenon within the region.

Ground shaking at the site generated by an earthquake on one or more of the active faults within the region will produce noticeable ground shaking at the subject site and other sites within the area. To mitigate this hazard, estimates of anticipated ground motion at the site are determined and suitable recommendations are developed for incorporation into the design of structures. Recommendations for seismic design in accordance with the 2019 Edition of the California Building Code (CBC) are included in a later section of this report.

It should be noted that the purpose of incorporating seismic parameters into site design is to safeguard against major structural failures and loss of life. Even after the incorporation of structural engineering into the design of the site structures, in accordance with applicable CBC codes, the potential for structural damage cannot be ruled out as a result of moderate to strong shaking. The structural objective of seismic designs is to limit risk of structural collapse.

Based on the above, we expect this hazard will be mitigated to levels that will have a typical, acceptable impact on the proposed development. Homes throughout most areas of southern California can be expected to experience moderate to strong ground shaking during the life of the project.

The nearest known active fault to the site is the Newport-Inglewood Fault, which lies approximately 4.3 miles to the west of the project. The maximum moment magnitude earthquake estimated for the Newport-Inglewood fault is 6.9 Mw. We used the U. S. Geological Survey (USGS) Uniform Hazard Tool and the 2014 dynamic fault model to estimate probabilistic seismic hazard conditions at the site. We assumed a 50 year exposure period and a risk of 2% probability of exceedance. This results in a return period of 2,475 years. We estimated the site class at the C/D boundary with a shear wave velocity of 360 meters per second in the upper 30 meters below ground surface. This analysis results in a peak ground acceleration of 0.658 g resulting from a moment magnitude event of 6.73 at a distance of 8.61 kilometers. A summary of this analysis is in **Appendix F**.

Landsliding: Of most obvious concern to the proposed redevelopment is the stability of the remediated landslide. As previously stated, it is our opinion that the 1998 landslide has been successfully remediated and therefore, not a threat to the proposed construction. Some minor low-level adjustment, creep of the remaining portion of the central landslide, located offsite to the west, has occurred and is possible in the future. Such movements are expected to be of low magnitude and not detrimental to the development of this project. No significant lateral movement or adjustments have been noted at the site and are not expected within the area underlying the areas of proposed construction.

According to the current seismic hazard zone maps issued by the State of California, a majority of the site is located outside the boundaries designated for investigation of earthquake-induced landsliding.

Based on the above, the potential impact to the site from a gross slope instability hazard standpoint is considered very low.

Debris Flow and Other Surficial Slope Failures: A debris flow is a type of slope failure described as the relatively rapid downslope movement of saturated sediment and other debris that eventually spreads into areas of lower elevation or flatter terrain before they come to rest. The materials involved in the flow are powerful enough to cause significant damage to structures in their path if not designed to retard them. The source of this hazard tends to be sediments infilling narrow swales or tributary channels on natural hillsides. They are mainly generated by significant storm events following periods of prolonged rainfall.

Slopes surrounding the site have been mass graded, constructed at slope ratios around 2:1 (horizontal:vertical), have a thick stabilizing cover of vegetation, and are equipped with a network of surface drains to prevent concentrated erosion. No tributary canyons or natural hillside areas remain on the site or within the adjacent areas. Given the above, threat of debris flow to the site development is considered to be low.

Liquefaction: Liquefaction is defined as the loss of shear strength due to an increase in pore water pressure typically within a poorly graded silty to sandy soil that is cohesionless, has relatively low density, is saturated, and falls within a rather specific grain-size distribution. The phenomenon is often triggered by ground shaking generated by strong earthquakes. Settlement of the soil mass may occur as the excess pore pressure dissipates. In relatively dry sands, settlement occurs during the earthquake shaking under conditions of constant effective vertical stress. It should be noted that liquefaction can only occur in the presence of a well-developed groundwater aquifer and peizometric surface shallower than a depth of around 50 feet.

According to the current seismic hazard zone maps issued by the State of California, the site is not located within the boundaries designated for investigation of earthquake-induced liquefaction. The site earth materials consisting of predominantly clayey soil and rock types mitigate against any national risk of liquefaction.

The presence of groundwater beneath the site is largely controlled and influenced by subsurface conditions existing up gradient of the site with respect to groundwater flow, within the Niguel Summit property to the west. This area underwent significant grading modifications in 2000 as part of the Via Estoril Landslide repair. Grading activities included installation of an extensive network of subdrains to control groundwater conditions. The monitoring of water levels in piezometers constructed for this report has established that these drains continue to perform as designed, minimizing the potential for rising groundwater.

As depicted within our cross sections, present groundwater conditions beneath offsite areas to the west largely occur as a narrow-perched zone above removal bottoms at the base of the fill. On the western margins of the site, the thickness of fills and depth to groundwater is on the order of approximately 65 to 80 feet below existing grades.

Based on the results of drilling and groundwater monitoring performed during the present study, within the boundaries of the site, a similar groundwater condition exists in this area consisting of a narrow zone of perched water above the natural bedrock contact at the base of residual alluvium deposits. The depths of water beneath the central and eastern areas of the site are around 35 to 40 feet below existing grades.

Based on the above, the potential for the occurrence of liquefaction beneath the site with impact to the development is considered negligible.

5.0 SLOPE STABILITY ANALYSES

5.1 SHEAR STRENGTH PARAMETERS

The shear strength parameters used in the stability analyses are based on the shear strength parameters obtained from laboratory testing and compared with the shear strength parameters shown in our referenced report. The conservative shear strength parameters based on the lowest bound shear strength for all types of soils and/or bedrock materials were utilized in the subsequent slope stability analyses and are presented in **Table 2** below. For the fill soil material, the lowest bound shear strength parameters based on the laboratory direct shear test results is zero cohesion and 32.5 degree of friction angle. However, to be conservative, shear strength parameters of c = 0 and a friction angle of 30 degrees were adopted for the fill soil material in the subsequent stability analyses. While some might argue that the clay soil could possess a moderate degree of cohesion, considering creep influence and being reasonably conservative resulted in the zero cohesion parameters. Based on the previous report, the landslide debris material has anisotropic shear strength parameters of c = 0 and a friction angle of 30 degrees for across bedding and c = 0 and a friction angle of 13 degrees for along bedding. The landslide plane has strength parameters of c = 0 and a friction angle of 13 degrees. These shear strength parameters for the landslide debris and slide plane were adopted for the subsequent slope stability analyses. The lowest bound of the bedrock material based on the laboratory testing results has zero cohesion and a 30-degree friction angle. Again, to be conservative, the most conservative shear strength value of zero cohesion and a 30-degree friction angle for the underlying bedrock material was adopted in the subsequent stability analyses. A review of the shear strength test data within Appendix C of our July 17, 2013 report will reveal how typical strength values are well above the lower-bound parameters used for the purposes of analysis.

TABLE 2 – SHEAR STRENGTH PARAMETERS

MATERIAL	DENSITY (pcf)	FRICTION ANGLE	COHESION
Fill	120	30	0
Slide Debris (Along Bedding)	120	13 (residual)	0
Slide Debris (Across Bedding)	120	30	0
Slide Plane	120	13 (residual)	0
Bedrock	120	30	0

5.2 SLOPE STABILITY ANALYSES

Slope stability analyses were performed on Cross-Section DR-DR'. A computer program, GSTABL7 Version 2.005.3 with STEDwin 3.59, was utilized to conduct the stability analyses. Stability analyses were conducted using Circular or Block Search as well as the Spencer Method to determine the factors-of-safety at different conditions. Both upper and lower slide planes shown in Cross-Section DR-DR' were analyzed as well. Groundwater conditions based on the subsurface exploration were also considered in the slope stability analyses. Both long-term (gross) and short-term (seismic) analyses were conducted.

5.2.1 Long-Term Stability (Gross) Analyses

Long-term stability (gross) analyses were conducted utilizing Geologic Cross-Section DR-DR'. The factor-of-safety based on the Circular Search with groundwater conditions was determined to be 2.308. The factor-of-safety value is well above the minimum required factor-of-safety of 1.50 for static analysis. Long-term stability (gross) analyses were also conducted using a shear strength of c = 0 and a friction angle of 13 degrees for both upper and lower slide planes and based on Block Search and the Spencer Method to evaluate the factors-of-safety for these conditions. The factors-of-safety for upper and lower slide planes using the Block Search and Spencer Method were determined to be about 4.350 and 5.754, respectively. These factor-of-safety values are well above the generally accepted minimum factor-of-safety of 1.50 for static analysis. Furthermore, it is our belief that the static factors-of-safety are sufficiently high to render subsequent pseudostatic analyses academic.

5.2.2 Short-Term Stability Pseudostatic (Seismic) Analyses

We have performed short-term stability (pseudostatic/seismic) analyses following the *Guidelines for Evaluating and Mitigating Seismic Hazards in California*, Special Publication 117A, September 11, 2008. Short-term stability (pseudostatic/seismic) analyses were performed using Geotechnical Cross-Section DR-DR' (see **Plate 2**). The following sections discuss our short-term (pseudostatic/seismic) stability analyses.

The seismic coefficient, K_{eq}, was calculated using the formula presented in that report and is taken as:

$$K_{eq} = f_{eq} \times (MHA_r/g)$$

Where MHA_r = the maximum horizontal acceleration at the site for a soft rock site condition;

g = acceleration of gravity;

f_{eq} = a factor related to the seismicity of the site.

Based on the 2019 California Building Code (CBC), the maximum horizontal acceleration at the site for a soft rock site condition (MHA_r) is determined to be 0.549g. The factor, f_{eq}, can be estimated to be 0.48 based on the 5 cm threshold displacement and Magnitude of 6.7 for r ≤ 10 km illustrated in the *Guidelines for Evaluating and Mitigating Seismic Hazards in California*, Special Publication 117A, September, 2008. The seismic coefficient, K_{eq}, using the equation, $K_{eq} = f_{eq} \times (MHA_r/g)$, can be estimated to be (0.48)(0.549g) = 0.263g. This value was utilized for the subsequent pseudo-static/seismic slope stability analyses. For transit loads, such as wind and earthquake, the shear strength parameters can be increased by one-third. Therefore, the shear strength parameters (i.e., cohesion and friction angle) used in the subsequent pseudostatic/seismic slope stability analyses were increased as is convention by one-third from the static values.

According to the Special Publication 117A, if the factor-of-safety (FS) based on the seismic coefficient, K_{eq}, is greater than 1.0, the site passes the screen analysis and is considered stable. No further seismic deformation analyses are needed. However, if the factor-of-safety (FS) based on the seismic coefficient, K_{eq}, is less than 1.0, then the site failed the screen analysis and further seismic displacement analysis should be conducted to evaluate the seismic effect to the project site.

Results of our short-term stability (pseudostatic/seismic) analysis for Cross-Section DR-DR' based on the seismic factor obtained from the Special Publication 117A indicated that the factor-of-safety based on the Circular Search with groundwater conditions was determined to be 1.264. The result passes the screen analysis of factor-of-safety of 1.0. Short-term stability (pseudostatic/seismic) analyses were also conducted on Cross-Section DR-DR' for both upper and lower slide planes and based on the Block Search and Spencer Method to evaluate the factors-of-safety for these conditions. The factors-of-safety for upper and lower slide planes using Block Search and the Spencer Method were determined to be about 2.066 and 3.813, respectively. Again, the results pass the screen analysis of factor-of-safety of 1.0.

As can be seen from the above results, the site passes the screen analysis for both "Circular" and Block" Search methods (i.e., FS > 1.0). Therefore, the site is considered to be stable under seismic conditions. No further seismic deformation analyses are required. Results of our short-term (pseudostatic/seismic) analyses based on the 2019 CBC seismic design criteria and the procedure outlined in Special Publication 117A are included in **Appendix G**.

As a person who was in detail involved with the development of the seismic slope deformation analyses in the late 1970s with former Professor Kenneth Lee of UCLA, the undersigned principal engineer (Gregory Axten) puts relatively little faith in the results of pseudo-static based seismic slope deformation analyses. To our knowledge, there is no database of information which compares predicted seismic slope deformation to actual performance. As such, adoption of the aforementioned methodologies is largely academic. As a practical matter, the proposed slopes, as conceived and analyzed to date, are expected to perform reasonably well in response to a regional earthquake of major magnitude. The site slopes can be expected to behave similarly to most 2:1 slopes of similar height in the general vicinity.

5.2.3 Summary of Long-term (Gross), Short-Term (Pseudo/seismic) Stability Analyses

Summaries of the results of the long-term (gross), short-term (pseudostatic/seismic), and temporary backcut stability analyses for the proposed site development are presented in **Table 3**. Results of our long-term (gross), short-term (pseudostatic/seismic), and temporary backcut stability analyses for the proposed site development are presented in **Appendix G**.

TABLE 3 – RESULTS OF SLOPE STABILITY ANALYSES

CROSS-SECTION	STATIC/ PSEUDO-STATIC	G.W.	SEARCH METHOD	STRENGTH PARAMETER	FACTOR-OF-SAFETY
DR-DR’	Long-term (Gross-Static)	Yes	Circular	C = 0, Ø = 13º (Along Bedding) C = 0, Ø = 30º (Across Bedding)	2.308
DR-DR’	Short-term (Seismic)	Yes	Circular	C = 0, Ø = 17.33º (Along Bedding) C = 0, Ø = 40º (Across Bedding)	1.264
DR-DR’	Long-term (Gross-Static)	Yes	Block (Upper Slide)	C = 0, Ø = 13º (Along Bedding) C = 0, Ø = 30º (Across Bedding)	5.754
DR-DR’	Short-term (Seismic)	Yes	Block (Upper Slide)	C = 0, Ø = 17.33º (Along Bedding) C = 0, Ø = 40º (Across Bedding)	2.066
DR-DR’	Long-term (Gross-Static)	Yes	Block (Lower Slide)	C = 0, Ø = 13º (Along Bedding) C = 0, Ø = 30º (Across Bedding)	4.350
DR-DR’	Short-term (Seismic)	Yes	Block (Lower Slide)	C = 0, Ø = 17.33º (Along Bedding) C = 0, Ø = 40º (Across Bedding)	3.813

5.2.4 Surficial Slope Stability Analyses

Surficial slope stability analyses were conducted for the proposed and/or existing 2:1 (horizontal:vertical) slopes. As indicated in the direct shear plots presented in our July 17, 2013 report, the existing fill soil material possess nonlinear curve pattern. To be conservative, cohesion of 250 pounds per square feet and a friction angle of 28 degrees are adopted for the subsequent surficial slope stability analyses. The results of the surficial slope stability analyses indicated that the fill slopes with 2:1 (horizontal:vertical) are stable for a depth of up to 5 feet. Realistically, slope influence in the areas at 2:1 (horizontal:vertical) and flatter has demonstrated adequate surficial stability.

5.2.5 Stability Analysis of 15.5-Foot High Mechanically Stabilized Earth (MSE) Wall

Mechanically Stabilized Earth (MSE) retaining walls up to 15.5 feet is proposed along the west portion of the project. Another MSE retaining wall up to 6 feet is proposed on the east side of the project (see **Plate 1**). The stability analyses for the proposed 15.5-foot high MSE retaining wall were conducted utilizing both MSEW 3.0 and ReSSA 3.0 software programs developed by ADAMA Engineering, Inc. MSE3.0 is a sowfware program for designing and/or analyzing MSE walls. MSEW 3.0 follows the design guidelines utilizing AASHTO and NCMA methods published in FHWA and NCMA publications. ReSSA 3.0 is an interactive software program designed to analyze the rotational and translational stability of the reinforced slope including mechanically stabilized earth slopes.

Cross-Section J-J' (**Plate 3**) was utilized to evaluate the proposed MSE retaining wall stability. Along this section the proposed buildings in front of the MSE wall are set back at least 15 feet from the wall and have a wall height of up to 15.5 feet. This represents the maximuim wall height depicted for the proposed site redevelopment. Therefore, a stability analysis of the proposed MSE retaining wall was based on the 15.5-foot high MSE wall.

Our stability analyses of the proposed 15.5-foot high MSE wall utilizing ReSSA 3.0 and MSEW 3.0 supports our recommendation that a 23.25-foot wide (i.e., 1.5H, where H is the height of the MSE wall measured from top of the wall to finish grade) Miragrid 20XT geogrid (or equivalent) with 1.5-foot (18-inch) vertical spacing is suitable for the proposed 15.5-foot high MSE retaining wall to satisfy rotational and translational stability for both static and seismic conditions. Geogrid product Miragrid 20XT manufactured by Tencate Geosynthetic with Long-Term Design Strength (LTDS) of 8,240 pounds per foot was utilized in our design. Other geogrid products with equivalent or higher LTDS can also be used subject to the review and approval by the project geotechnical engineer. Results of the stability analyses of the 15.5-foot high MSE wall using ReSSA 3.0 and MSEW 3.0 are included in **Appendix H**.

From a practical perspective, MSE walls, like other gravity wall systems, such as crib walls, are flexible systems compared to conventional retaining walls and other mechanically restrained walls, such as tieback walls. For MSE wall systems with predominantly sandy, non or low expansion potential backfill, little or no wall movement will occur under normal static conditions. With expansive clay soil, backfill flexibility in the wall systems will manifest. As such, onsite soil with moderate to high expansion potential should not be used as retaining wall and/or MSE wall backfill material. Any flatwork in areas above the MSE wall should also be designed to accommodate yielding/creep. A detailed discussion of the building foundations for the building above the proposed MSE retaining wall (e.g., above Crowne Valley Parkway) will be discussed later in this report.

6.0 PRIMARY GEOTECHNICAL CONSIDERATIONS

6.1 GENERAL

In addition to the potential geologic hazards described earlier, the primary geotechnical considerations at the site are expansive soil activity, slope creep phenomenon of the existing stabilizing fill, and proposed new fill soil and settlement risk associated with fill and landslide debris. Expansive soil activity, settlement, and slope creep phenomenon are discussed below. Settlement is worthy of mention but is of no significant concern.

6.2 EXPANSIVE SOIL

Soil with a significant clay fraction tends to possess expansive characteristics. Expansive soil heaves when water is introduced and shrinks as it dries. Pressures produced by heaving soil can be large enough to lift most buildings. Slabs over expansive soil are often said to "walk" as a result of expansive soil movement. This process generally tends to increase separation of slab joints and/or cause exterior improvements such as patios, originally abutting structures, to separate. Expansive soil can also cause cracking of slabs and foundations. Relatively light slab and foundation systems on expansive soil will tend to curl up at the building perimeter; a process sometimes referred to as edge lift or dishing. Over the long-term, as moisture migrates further under the building and in response to drying along the building perimeter, the process reverses and central doming sometimes referred to as a heave pattern results. This doming process is also alternatively described as edge drop and can be further aggravated near descending slopes by ordinary slope creep by long-term clay desiccation of the slope areas. Relatively stiff foundations are needed to resist the adverse influence of expansive soil.

Expansive soil tends to be active near the ground surface. The actual depth varies with specific material types and environmental differences. To reduce the effect of expansive soil on surface structures, foundation systems are usually deepened. Slabs and foundations are usually reinforced to increase their resistance to differential movement, such as relatively stiff, essentially uniform thickness mat foundations. Where buildings are not proposed next to descending slopes, simple mat foundations are advised. Where buildings are proposed next to descending slopes, deepening house foundations and/or increased stiffening are advised. Given the generally highly expansive conditions, vertical and lateral support for new foundatons should be ignored within a horizontal distance to daylight (HDTDL) of 25 feet unless otherwise supported by an adjacent retaining wall or MSE wall system designed with “at-rest” (i.e., no movement) soil assumptions.

It is usually suggested in planning yard improvements and a landscape theme that maintaining uniform moisture conditions as much practical around isolated individual structures is desirable. Preferably, soil should be kept on the moist side without allowing ponding. Since water tends to migrate under slab areas, saturation of the slab subgrade for conventional construction is usually recommended prior to placement of slab concrete. This process, presaturation, activates the soil expansion prior to slab placement and thereby limits both the tendency toward edge lift and long-term heave. Deepened exterior footings and/or membrane barriers for a conventional foundation system also serve to limit the degree to which drying can occur and thereby limit edge drop as well. While adopting these practices plus generally good drainage can modulate moisture changes to some degree, preventing moisture changes is generally thought of as impossible.

In the absence of mitigating measures, placing trees within about 10 to 20 feet of the structures' foundations is not desirable because they tend to extract water and contribute to edge drop. Unless a conservative approach to the design is taken, actual tree/soil/structure interaction should consider actual tree species. Large trees should be kept at an even greater distance unless special precautions are taken.

Similarly, structures too close to descending slopes can be subject to expansive soil-related creep influence. In such cases, deepening and stiffening of foundations and designing foundations as retaining walls are necessary to limit the risk of damage. Where mat or conventional foundations are constructed with relatively deep membrane cut-offs, which also serve as root barriers, no special planting prohibitions are necessary but landscape can influence appurtenant improvements, such as flatwork.

As discussed in our July 17, 2013 report, results of our laboratory testing revealed that the onsite soil has expansion potential with Expansion Index (EI) ranges from 11 (Very Low) to 82 (Medium). The sample of old fill collected from a depth of 30 feet within our Boring AGPZ-3 in the eastern area of the site was found to possess generally medium expansion potential (EI = 59). The results of EI testing conducted at the site by other consultants describe the soil as having a critically expansive (Duco, 1976). For design purposes, we would consider the site soil as having a 'High' expansion potential. Further testing is advised in conjunction with remedial grading and general preparation of the site for home construction.

In recent years, thicker (e.g., 12 to 18 inches) essentially uniform thickness slabs have gained popularity over the concept of a thinner slab over a waffle-like system of grade beam stiffeners. The mat slabs are much simpler in design and, as such, can be constructed with relative ease at reasonable cost. The uniform slabs also provide far less restraint to shortening, and accordingly, can be constructed with far less risk of crack development. Specific foundation design citeria will be given in later report sections.

6.3 SLOPE CREEP

As a result of weathering of the faces of slopes, gravitational forces, seasonal moisture variations, and expansive soil conditions, the near surface soil on a slope tends to gradually move downslope. Generally, the more expansive the soil, the deeper the influence will be. This weathering process is usually most apparent in the topsoil profile of a natural slope. The underlying formational material can also be affected to a more limited extent. Compacted fill slopes, particularly those composed of expansive clayey soil, tend to be most impacted by creep. While young projects have creep occurring in response to wetting of soil, more mature projects tend to also suffer from long-term clay desiccation. Slope planting since the late 1970s has tended to favor deep rooting, drought-tolerant plant and trees. While in the early years, irrigation to promote plant growth tends to increase soil moisture, the maturing plants eventually reach the point that water demand exceeds that provided by surface irrigation. As such, long-term moisture depletion occurs. Shrinkage in the soil results in clay desiccation creep. This process has been further aggravated by water conservation efforts. As such, an improvement within about 30 feet or so from an unconfined descending slope (e.g., no retained walls or deep foundations) can display creep influence over the years. For practical design purposes, we have recommended that support be ignored within 25 feet HDTDL. Features like adjacent restrained walls and/or deep foundations tend to substantially reduce creep effects. Critical structures should be designed with provisions to isolate them from creep influence. Non-critical structures (e.g. flatwork) should accommodate creep to the extent practical.

For this project, an MSE wall was planned to be placed at the top of the existing slope along Crown Valley Parkway, a relatively mature slope, where significant creep resistance would be achieved. The project architect and structural engineer must work closely with the owner and our office in deciding which structures will be designed to resist creep influence and which structures will be designed with flexibility in mind to accommodate creep influence. Based upon our experience with creep in compacted fill soil throughout Orange County, California for a period of decades, the most active form of creep, long-term, environmental creep induced by seasonal moisture cycles can exceed 10-feet vertical. Criteria have been developed for significant creep impacted soil extending to a depth of about 12.5 feet (or 25 feet Horizontal Distance To Daylight, HDTDL). Althouth creep influence can extend deeper, assuming complete loss of support for 25 feet HDTDL, is considered both appropriate and practical. Due to the highly expansive soil conditions, this criteria is more conservative than the current Code criteria for minimum foundation setbacks (horizontal distance to daylight of H/3; 40-feet maximum) for the site slopes.

6.4 SETTLEMENT

To some degree, all materials are compressible. Settlement or subsidence of the ground surface can also occur on a regional scale and manifest as surface cracking in response to deep groundwater or petroleum withdrawal. More locally, this phenomenon can occur where loose and/or porous and compressible earth materials are surcharged by a body of overlying fill, in areas underlain by artificial fills having low relative densities and/or dryer than optimum moisture contents. Settlement occurs as a result of the stresses imposed, and most significant stresses usually result from the weight of a structure, as well as the weight of the earth materials. Settlement can, in some cases, be aggravated by the introduction of water to the subsoil. Fill material and natural soil tend to be more compressible than bedrock materials (e.g., many "cut" areas). Accordingly, settlement potential usually increases with increases in the depth of fill and natural soil (e.g., compaction). Where the depth of fill and natural soil vary, such as in areas where transitions/contacts exist between earth units having differing settlement potentials, the potential for differential settlement increases. The amount of differential settlement is of most concern since differential movements can result in distress. New fills also commonly undergo a certain degree of settlement both during and for a period of time following its placement. The amount of settlement is generally based on its thickness and other conditions of placement.

A substantial amount of fill was used to construct the 1998 landslide gravity buttress. The remedial fill within the buttress is as much as 70-feet thick below the building locations of some of the proposed new condominiums. Given the timeframe in which the fill was placed and given the inclinometer behavior which is not consistent with settling fill, no problematic settlement is believed to have occurred and none is expected. Proposed construction includes placement of only nominal additional fill. Risk associated with the expansive nature of the soil far outweights any settlement risk. As depicted on our cross sections, the maximum thickness of proposed fill is on the order of 10 feet. As briefly described earlier, an initial area of interest in our investigation was the suitability of existing fill and landslide debris over which the fill was placed. Investigation has revealed that the remaining landslide debris has a character similar to undisturbed, weathered bedrock. As stated in our December 9, 2016 report, our down-hole logging of the fill revealed what appeared to be very competent material. In fact, the fill seemed so well compacted in general, that some concern arose over our ability to obtain truly representative, undisturbed samples for testing. As such, the undersigned, president/CEO/principal engineer, carefully examined the ring samples at our laboratory for their condition and possible disturbance. Some clear dilation, break-up of the upper portion of samples (away from the tip) was apparent. Samples close to the tip end were generally very stiff to hard. An effort was made to trim relatively undisturbed samples and test them for response to moisture introduction. The test results revealed generally low-level collapse responses, most of which was believed to reflect minor sample disturbance. Nonetheless, the overall results are consistent with good quality fill and very limited settlement potential.

The site is not located within an area known for historical groundwater or petroleum withdrawal. It is underlain entirely by engineered fill placed during mass grading under the observation and testing control of geotechnical engineers. Given the significant amount of time since the fills were placed, any significant post-construction settlement of these deposits has long since occurred. Similarly, the deposit of Older Quaternary Alluvium buried by the fill is of such a significant age that it is very dense and possesses very low compressibility potential.

Based on the above, the characteristics of the existing earth materials, their distribution beneath the site, and the fact that no significant changes in grade are proposed, no significant adverse settlement or subsidence is anticipated. The effect of differential settlement above the contact between bedrock and alluvium on the site is also considered negligible. Since most project fill will have been in place for more than a decade prior to building homes, and considering the fill is expansive, settlement will be of little concern compared to that of expansive soil influences and slope creep. Settlement should be within tolerable limits (usually less than about 1-inch total with differential between adjacent similar foundations not typically greater than 1/2-inch).

7.0 CONCLUSIONS AND DISCUSSIONS

Based upon the results of our geotechnical evaluation, it is our opinion that the proposed The Cove at El Niguel redevelopment at the site is feasible from a geotechnical standpoint. However, the site possesses some adverse geotechnical/geologic conditions that will impact the site development and will require special attention. Development of the project can be accomplished without risk of landsliding or subsidence adversely impacting adjacent properties. The recommendations presented in Section 8.0 should be incorporated into the project plans and specifications for redevelopment of the site.

7.1 IMPACT OF THE PROPOSED SITE DEVELOPMENT ON OFF-SITE PROPERTIES

As can be seen from the geotechnical/geologic map (**Plate 1**) and cross sections (**Plates 2 and 3**), no immediate grading operations will be conducted adjacent to off-site improved properties. American Geotechnical has since the time of the landslide provided periodic consulting to the upslope homeowner's association. Similarly, we have provided periodic consulting, even recently, to the southerly adjacent, La Vista, homeowners association. As such, we have great familiarity with those projects. While we have not provided consulting to the northerly adjacent property, the geotechnical conditions in that area are reasonably understood and the least construction impact will occur along the north boundary. Based upon all our knowledge, and in consideration of only relatively minor to moderate grading for the proposed development, we cannot reasonably anticipate any adverse geotechnical impact on the adjoining properties. As such, it is our further opinion that no special monitoring adjacent to off-site properties is required during construction. If in the opinion of the City reviewers, boundary monitoring should be provided, vibration and sound monitoring could easily be established at the project boundaries. Inclinometers could be installed. More sophisticated ground monitoring is also available (e.g., tiltmeters) and could be considered if dictated by the City. If vibration, sound, or other monitoring is required by the City, the monitoring devices should be in place at least one month prior to grading so that typical background noise/movement thresholds can be identified.

American Geotechnical is very familiar with past claims by the La Vista HOA, the condominium complex to the south of the proposed site development. Previously, we have periodically inspected conditions at the La Vista HOA since the time of the original landslide and we have even consulted on behalf of that HOA. For example, it is our understanding that the condominium complex to the south of the project has water/moisture leakage through at least one low-level retaining wall at the back of a garage. The street pavements have some cracking in various pavement areas. It appears that a seal coat was put over the AC pavement with little attempt to repair underlying conditions resulting in cracking in the AC pavement. It is our opinion that such stress features were primarily related to original pavement designs and long-term maintenance issues. There was no adverse impact on that project by the original landslide and/or stabilization efforts, nor is any adverse impact anticipated by the proposed development. We are confident in opining that to the extent that geotechnical problems may exist at that HOA, no issues related to the past landslide, or past remediation, and no future issues can be expected from the currently proposed grading and redevelopment plan. Presently, we also believe that no boundary monitoring is needed during construction. However, as aforementioned, if the City desires monitoring at the La Vista HOA property boundary at the time of construction, monitoring can be provided. Similarly, this office has not recognized for any reasonably foreseeable risks to the existing residential properties to the west and north.

8.0 RECOMMENDATIONS

8.1 GENERAL

Conclusions and recommendations contained in this report are based upon the information provided, information gathered, engineering and geologic evaluations, experience, and professional judgment. Recommendations contained herein are consistent with industry practices. Other alternatives exist and can be discussed on request. Regardless of the approach taken, some risk will remain, as is always the case.

8.2 EARTHWORK

Except as noted in the text of this report, all demolition, site preparation, and grading operations should be conducted as outlined in the Standard Guidelines for Grading Projects included as **Appendix I**, and should also be in conformance with the latest edition of applicable building codes and the City of Laguna Niguel's grading requirements. Generally speaking, there are no significant problems anticipated in conjunction with the proposed grading. Due to the compact, clayey nature of the site soil, temporary cuts at 1:1 (horizontal:vertical) should be adequately stable. For safety reasons, vertical temporary cuts should be limited to about 4 feet high.

8.2.1 Site Preparation

Grading is planned to some extent throughout the project to create level building pads for the planned structures and to create roads and other improvements for the planned redevelopment. Prior to grading, the site should be cleared of all surface and subsurface obstructions including things such as existing structures, fill, debris, buried utilities, and should be stripped of vegetation within the proposed pad areas. Vegetation and debris should be disposed of off-site. Any holes or excavations made which extend below finished grade should be filled with compacted soil. Due to on-site fill weathering over time, 5 feet minimum of over-excavation and recompaction should be anticipated for improvement areas and areas of new fill. Where relatively shallow cuts are proposed, the amount of over-excavation will be reduced accordingly. The actual amount of over-excavation in any particular area will be subject to review of the conditions at the time of grading but will likely be about 5 feet but could range to about 10 feet maximum.

8.2.2 Cuts and Excavations

It is our opinion that the material present on-site can be excavated with conventional earth moving equipment. Excavating within the fill soil will be relatively easy. Cut pads and slope excavations are expected to encounter engineered fill material which were placed during the landslide repair. It is recommended that the earthwork contractor(s) visit the site and review this report in detail prior to submitting bids. Past swell and weathering of existing, upper 5 feet of site fill could result in on the order of 7.5 to 12.5 percent shrinkage when stripped of vegetation, excavated, then recompacted.

All permanent cut slopes planned for the development shall not be steeper than 2:1 (horizontal: vertical) unless approved by the geotechnical consultant. Although we anticipate all cuts will expose existing fill, any cut slopes exposing bedrock and adverse bedding, unstable, loose, significantly fractured, or otherwise unsuitable materials may require over-excavation and replacement with a compacted stabilization fill as recommended by the geotechnical consultant. A member of our firm should be notified and present to review all cut slopes and excavations prior to placing fill and/or foundation steel or concrete.

8.2.3 Fills

The site fill soil possesses expansive soil characteristics. Without fairly massive and impractical import and export, selective grading to lower expansivity is not possible. Blending desirable chemical constituents into the soil is also possible for lowering expansive soil risk, but such procedures are commonly considered somewhat impractical and are seldom adopted. Soil improvement techniques are not expected but could be given further consideration, if desired. The on-site soil is considered suitable for use as compacted fill provided it is free of organic material and debris and the moisture content is adjusted to within a compactable range.

Although not currently anticipated, if imported soil is used for structural fill to achieve design grades it must first be submitted to the geotechnical consultant for approval and additional testing. In addition, the soil should generally conform to the following criteria. If such material is not available, further remedial measures can be delineated.

- Should be free of environmentally hazardous materials.
- Should preferably be non-expansive or of low expansivity.
- Should be free of oversized material (material greater than 8 inches).
- Should be free of organic material and debris.
- Should preferably have less than 0.1 percent sulfate content.
- Should preferably have less than 25 ppm chlorides and greater than 6,000 ohm-cm soil resistivity.

The above criteria should be considered ideal. Good quality material may not be available within reasonable proximity to the project. In general, we expect that import soil should be at least comparable to the on-site soil. Final designs for foundations and other improvements should be based on as-graded conditions. All fill to be placed should be moisture conditioned and compacted to a minimum of ninety (90) percent of the maximum dry density per the latest ASTM Standard Test Method D-1557. Fill should be placed in thin, uniform lifts (6 to 8-inches loose thickness). Fills placed on natural slopes or cut slopes should be keyed and benched into firm, competent material and inspected by the geotechnical consultant prior to the placement of fill. Any permanently graded site slopes should be constructed at inclinations no steeper than 2:1 (horizontal:vertical). The geotechnical consultant should be on-site to test all fills/removals during grading operations.

8.2.4 Cut/Fill Transitions and Lot Capping

Based on our review of site conditions and the latest proposed building layout plan, cut/fill transitions and lot capping are not expected. The need for capping can be further evaluated during grading. If, for example, a building area with new and old fill were to expose significantly varying soil moisture, local remedial grading could be advised. If needed, over-excavate to a minimum depth of 5 feet below finish grade and at least 5 feet below the base of the mat foundation and footings for appurtenances. A properly compacted structural fill blanket should then be placed in the resulting excavation. The limits of this work should extend a minimum of 5 feet beyond the aerial extent of proposed improvements (i.e., beyond the limits of the building and appurtenances). The geotechnical consultant should observe the bottom of the excavated areas at the time of grading to assess the quality of exposed material and to evaluate if additional removals or recommendations are required.

8.2.5 Material Volume Changes

It is anticipated that the grading process will create a volume change that will vary with material type and conditions created. It is expected that the excavation and recompaction in soil areas will create a net loss or shrinkage. Although grading is likely to be contained within compacted fill, the following criteria can be used as estimates for earthwork calculations:

Weathered artificial fill .... 7.5% to 12.5% shrinkage
Loose natural soil .... not expected
Moderately dense natural soil .... not expected
Landslide debris... not expected
Weathered bedrock or bedrock .... not expected

8.3 SLOPES AND SLOPE STABILITY

Any permanently graded site slopes should be constructed at inclinations no steeper than 2:1 (horizontal:vertical) unless approved by the geotechnical consultant. As for the new 2:1 (horizontal:vertical) slopes, history has demonstrated throughout southern California that 2:1 slopes are far less likely to experience surficial instability, particularly if the compacted fill slopes are constructed by overfilling and cutting back to the compacted inner core and where slope construction is followed by landscaping with generally deep rooting plants which are somewhat drought-tolerant. Existing 2:1 slopes are considered acceptable but are at greater risk of superficial instability (e.g., erosion and slumping/pop-outs) in response to such conditions like prolonged heavy rainfall and sprinkler malfunctions. If occasional, superficial instabilities do occur they are typically localized and have been triggered by severe moisturization either by a pipe or irrigation problem or an unusually prolonged and heavy rainfall. Any reinforced slope should be essentially free from any risk of surficial slumping. The slope plants at the project consist of prostrate acacia which has worked very well in mitigating risk of surficial instability. The landscape architect should be aware that plants like prostrate acacia and most eucalyptus varieties high in suction available and accordingly can serve to significantly dry back the soil long-term. Such species and similar very drought-tolerant species should be separated from improvements to the extent practical. Root barriers should be considered for the protection of improvements such as those advised for mat foundations.

8.3.1 Fill Slopes

The planned fill slopes will be located on the lower pad area near Crowne Valley Parkway. The proposed slopes should be constructed in accordance with the recommendations of this report and the City of Laguna Niguel's grading requirements.

Presently, no signs of surficial instability have been observed. Nonetheless, if surficial instabilities (i.e., shallow failures and erosional features) are encountered, they should be addressed during design and construction of the proposed slopes. Surficial instabilities should be excavated, a key established (1 meter deep by 4 meter wide), and the soil be replaced with compacted fill reinforced with 4-meter wide (Synteen SF35, Tencate Mirgrad 3XT, or better) spaced vertically at 18 inches. It is recommended that slopes be planted immediately following grading to help mitigate the potential for surficial instability and erosion. For existing slopes at 2:1 and flatter, risk of surficial instability is low. For reinforced 2:1 slopes, surficial instability risk is virtually non-existent.

Erosion resistant plants should be used in slopes, preferably deep-rooted varieties adapted to semi-arid climates. Heavy bodied, shallow-rooted varieties such as ice plant should be avoided. In addition, drainage should not be allowed to flow over slopes. Surface water should be directed away from the top-of-slopes, although rear yard drainage is typically recommended to be to the street. Rear yard drainage could be collected by a system of inlets and pipes and directed to the nearest terrace drain in the slope below or to the street below the rear yards. It is recommended that the new HOA authorize annual geo-reviews of all slopes, pavements, and other common area improvements.

8.3.2 Cut Slopes

Any planned cut slopes will require review/monitoring by a geologist. Since only fill should be exposed, no adverse conditions are expected. Although not currently expected, if seepage is exposed, local drainage systems will be recommended as field conditions dictate.

8.3.3 Setback Criteria

Any footings near slopes should satisfy a minimum horizontal setback as indicated in the 2019 California Building Code, Chapter 18, Section 1808A.7, Foundations on or adjacent to Slopes and Figure 1808A.7.1, Foundation Clearances from Slopes. This distance should be measured from the lower leading edge of the footing to the slope face. For footings near the top-of-slope, the code dictates that at least the smaller of H/3 or 40-foot setback should be provided, where H is the height of the slope. For this project, we recommend that foundations located near top of slopes be designed to penetrate and resist a loss-of-support zone of about 12.5 feet (or 25 feet Horizontal Distance To Daylight, HDTDL).

8.4 PRELIMINARY FOUNDATION DESIGN

Potential slope influence and expansive soil influence are considered to be of greatest potential risk to the project and should be given careful consideration by the project architect and structural engineer. As design aids, this report includes some minimum criteria. If either the architect or structural engineer requires more information or assistance, they should contact the American Geotechnical project engineer. Following grading, at the time that project details and final designs are needed, American Geotechnical can provide the actual foundation designs on building by building basis. For American Geotechnical to provide this supplemental service, the structural engineer will need to provide to us a load plan for each unit type along with proposed hold-down locations and types of hold-downs.

Based on the latest building layout plans, recommendations for building foundations utilizing mat slab foundation system have been developed. For expansive soil influence consideration, all buildings have been preliminarily recommended to have a 16-inch thick mat slab with construction details illustrated in **Figure 2**. At the south end of this project some units may be above a 1:1 projection from existing or proposed storm drains. For such buildings, the project civil engineer should anticipate the addition of piers to extend support below any such existing or planned utilities or relocate the existing utilities. According to the preliminary grading plan by Hunsaker and Associates, all existing storm drain utilities within the proposed site redevelopment area in Lot 1 will either be removed or relocated. As such, piers are not anticipated if the storm drain utilities are to be removed and/or relocated. If cast-in-place piers are planned, for preliminary planning purposed, anticipate 24-inch diameter piles to 5 feet below any utility of interest. For preliminary design purposes, plan 8#8 bars with #4 ties at 9 inches on center reducing to 4.5 inches on center within 24 inches of the bottom of the mat slab. The basic mat slab detail will remain the same as presented in **Figure 2**. To tie the piles to the mat, plan 8#8 bent dowels, 48 inches by 48 inches, extended into the mat around each pile. The dowels should be positioned within the middle one-third of the mat.

8.4.1 Mat Slab

It is recommended that all buildings constructed at the site be designed with structural slabs/mat slabs to account for expansive and other soil influences. The mat slab foundation system should be capable of spanning a minimum unsupported, continuous cantilevered distance of 8 feet. It should be realized that the critical design conditions are the outer edges and the end corners of the mat slab. The allowable bearing pressure and coefficient of friction can be used as 1,500 pounds per square feet and 0.3, respectively. No passive resistance within the soil medium should be assumed in the upper 2 feet in a non-slope direction. When considering concrete restraint to shortening a friction value of 2.0 and a passive soil pressure value of 1,000 pcf for the first cycle friction and passive restraint. Isolated foundations for site walls and retaiing walls should ignore vertical bearing to depth of 3 feet. The mat foundation design should be performed by a separate structural engineer or by our office. It is recommended that a minimum 16-inch thick mat slab system be considered for the project (see **Figure 2**). Our office can work with the developer and provide final foundation designs, if desired.

As previously discussed with respect to foundations, the mat slab foundation system should be supported on a minimum of 5 feet of compacted fill. All building areas are expected to be underlain by at least 5 feet of fill. The fill subgrade beneath the slab-on-grade floors/mat foundations should be compacted by rolling with a smooth-wheeled, rubber-tired, or vibrating roller to produce a uniformly dense, non-yielding surface.

Mat Slab Detail

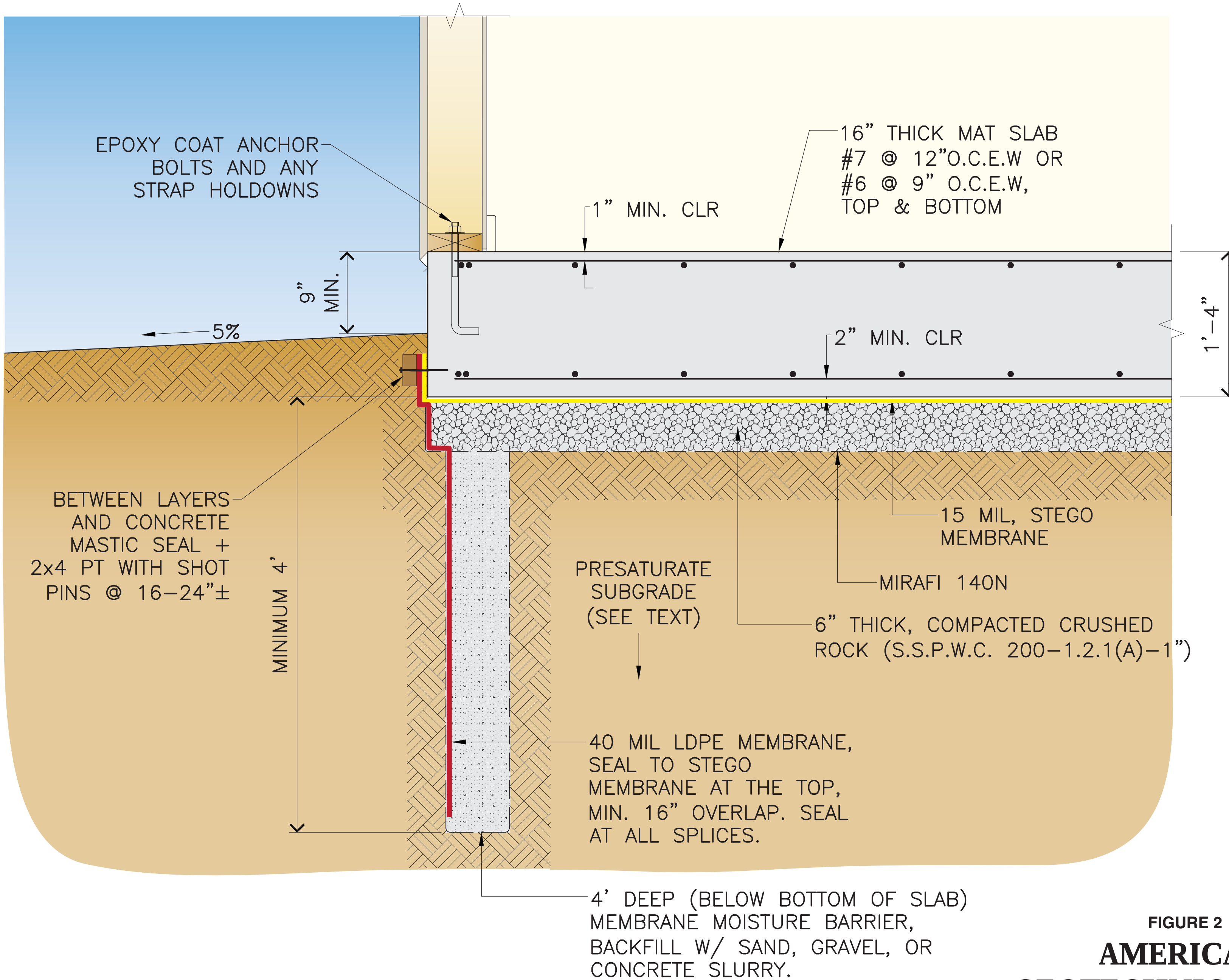

FIGURE 2

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Foundation excavation debris should not be cast into any slab-on-grade areas without proper compaction. For slab areas, presaturation (100 percent of field capacity) to at least a depth of 36 inches should be accomplished prior to slab construction. To facilitate deep presaturation, we recommend that 4 to 6-inch diameter holes be drilled within the slab area at a maximum grid spacing of 8 feet. Excavation spoil should be removed from the slab area. Each hole should be backfilled with clean crushed rock or gravel. A layer of free-draining crushed rock at least 8 inches thick should be underlain by a layer of Mirafi 140. Only about 4 to 6 inches of the rock should be placed initially. The rock should be thoroughly flooded for about two weeks to facilitate the presaturation. Our representatives should verify satisfaction of the presaturation requirement. Without the benefit of the gravel filled grid of auger holes, presaturation could take two to three months or longer. The gravel or crushed rock should have no more than 10 percent passing the 3/4-inch sieve nor more than 3 percent passing the No. 200 sieve. Upon completion of presaturation, the final gravel can be added as needed. A light non-woven geofabric (e.g., Mirafi 140N) layer should be placed above and below the compacted rock.

A vapor barrier over the crushed rock should be considered in areas where the migration of moisture through the floor slab would be detrimental. The vapor barrier is advised for the garage slab in addition to living spaces. The vapor barrier should be at least 20 mil virgin plastic LDPE, such as 20-mil Stego plastic, and should be sealed at all splices, around plumbing and other protrusions, and at the perimeter of slab areas. Every effort should be made to provide a continuous barrier, and care should be taken not to puncture the membrane. Sealing should only be accomplished with the membrane manufacturer's approved materials (e.g., Stego Mastic and Stego type). All detailing should be in accordance with the manufacturer's specifications. Proposed materials and methods for subgrade membrane placement, sealing, and protection should be provided by the contractor for our review and acceptance. The subgrade membrane should be sealed to the cut-off membrane. The subgrade membrane should be wrapped up to perimeter form boards and temporarily attached to the top of the form boards. The actual membrane selected, like Stego recommended above, should conform to the most rigorous specifications within ASTM E1745 (0.1 perm). Installation should conform to the continuity indicated in ASTM E1643. The manufacturer's special mastic and tape should be exclusively used to seal joints, other splices, protrusions, and perimeters. As an alternative to the slab subgrade membrane, the slab concrete can be cast incorporating "Hycrete" concrete waterproofing additive. If the membrane is omitted, the bottom steel clearance should be at least 3 inches. Placing the membrane is considered the most practical approach. A detail expected to closely resemble the final mat slab detail has been presented in **Figure 2**. Once final building concepts are approved and structural loads determined, final mat slab details can be determined.

8.4.2 2019 California Building Code (CBC) Seismic Design Parameters

Based on the available information about the fault zone closest to the site and the soil conditions, the following seismic design parameters are recommended according to the 2019 California Building Code (CBC) and ASCE 7-16. Final selection of the appropriate seismic design coefficients should be made by the project structural engineer based on the local laws and ordinances, expected response of the proposed structure and desired level of conservatism.

TABLE 1
Seismic Hazard Response Parameters and Design Parameters

Latitude: 33.5189° - Longitude: -117.7189° Seismic Design Parameters	**Symbol**	**Value**
Site Class	-	D
Risk Category	-	II
Site Amplification Factor at 0.2 Second	F_a	1.0
Site Amplification Factor at 1.0 Second	F_v	2.5
Spectral response acceleration parameter at short periods adjusted for site class effects	S_{MS}	1.267 g
Spectral response acceleration parameter at a period of 1 s adjusted for site class effects	S_{M1}	1.130 g
Design spectral response acceleration parameter at short periods	S_{DS}	0.845 g
Design spectral response acceleration parameter at a period of 1 s	S_{D1}	0.753 g
MCE_G Peak Ground Acceleration	PGA	0.549 g
Site Amplification Factor at PGA	F_{PGA}	1.1
Site Modified Peak Ground Acceleration	PGA_M	0.604 g

It should be noted that the above seismic parameters are based on Site Class D and using SEAOC/OSHPD web tool as well as simplified procedure outlined in a paper titled, "*New Site-Specific Ground Motion Requirements of ASCE 7-16*," by Charles A. Kircher, published in 2017 SEAOC Convention Proceedings. Per ASCE 7-16, Section 20.3, Site Class D can be used for structures that have fundamental period of vibration equal to or less than 0.5 seconds.

It should be realized that the purpose of the seismic design utilizing the above parameters is to safeguard against major structural failures and loss of life, but not to prevent damage altogether. Even if the structural engineer provides designs in accordance with the applicable codes for seismic design, the possibility of damage cannot be ruled out if moderate to strong shaking occurs as a result of a large earthquake. This is the case for essentially all structures in Southern California.

8.4.3 Concrete

Experience and research has shown that concrete with a high water/cement ratio can experience problems, such as excessive shrinkage cracking, moisture intrusion, and high vapor emissions, among other things. Generally speaking, the higher the water/cement ratio, the higher the porosity and permeability of the concrete, and the lower the strength. Concrete designed for minimum compressive strengths on the order of 2,000-3,000 psi usually have relatively high levels of mixing water and, correspondingly, a high water/cement ratio, relatively high permeability, and greater risk of shrinkage cracking as well as deterioration after concrete itself and corrosion of the embedded steel components.

Consideration should be given to using the lowest possible water/cement ratio while still maintaining workability. If necessary, water reducing agents can be used to increase workability. Because of the high levels of sulfate present on-site, it is recommended that concrete used for footings and slab areas conform to the 2013 California Building Code Section 1904A.3 Concrete Properties, ACI 318 Table 4.2.1 Exposures and Classes, and Table 4.3.1 Requirements for Concrete by Exposure Classes, for maximum water/cement ratio and other concrete design elements. All steel and concrete materials, details, placement procedures, and curing should be performed strictly in accordance with ACI specifications and guidelines. The slab design by the structural engineer and/or architect should consider shrinkage of the concrete to limit cracking to the slab and overlying floor coverings. Type V, 0.45 maximum water/cement ratio concrete (0.42 preferred) should be planned. Our experience with 0.42 to 0.45 water-to-cement ratio indicates that 5,000 psi in 28 days is likely even though 4,500 psi minimum would be specified. For any slab cast directly on the subgrade membrane, a shrinkage reducing admixture (e.g., Grace Eclipse) should be provided and dosed for maximum benefit. Alternatively, shrinkage compensating concrete should be used for mats. Shrinkage neutral concrete can generally be achieved by substituting about one sack of CTS Komponent for one sack of cement in each yard of concrete. The concrete supplier and CTS can be consulted for an actual mix design. Another approach that could be considered would be to provide corrosion resistance for concrete and steel by adding a waterproofing admixture to the concrete such as

Hycrete. This approach could prove more costly for slab-on-grade concrete, but is worth consideration where any split-level retaining walls are proposed.

Care should be taken to properly cure all exposed concrete. At least seven days of continuous moist curing is typically sufficient. More detailed curing recommendations can be provided as construction approaches.

8.4.4 Reinforcement Placement

Care should be taken when placing foundation and slab reinforcement. Placement details should be in conformance with ACI specifications (U.N.O.). The bottom foundation steel should not be closer than 3 inches to the underlying excavation where no subgrade membrane has been provided but preferably 4 inches due to the corrosive nature of the soil. For only 3 inches of coverage epoxy coated steel bars is advised otherwise the clearance should be increased to 4 inches unless Hycrete Admixture can be provided. Appurtenant slab reinforcement should be placed in a positive fashion between the midpoint and upper one-third portion of the slab section. “Lifting” slab steel into place following concrete placement is not recommended.

8.5 APPURTENANT STRUCTURES

For appurtenant conventional slabs-on-grade, the minimum slab thickness should be 6 inches. Appurtenant slab reinforcement should be at least No. 4 bars centrally positioned and spaced at 12 inches on center each way or No. 5 bars at 18 inches on center each way. For non-critical flatwork, neither epoxy coated re-bar or Hycrete are required but should be considered. Hycrete is specifically recommended as an additive for curing pool and spa decking.

Special consideration should be given to slope creep and expansive soil influences. However, the recommendations for slab thickness and reinforcement for exterior flatwork still pertain to help reduce the potential for cracking and separation. Thickened edges (at least 4 inches) should be planned for the outer 8 inches of appurtenant slabs. Deeper cut-off walls or root barriers should be considered where flatwork is proposed near trees. Generally, trees can be expected to have roots extend out from the trunk a distance of about equivalent to the height of the tree. Root barriers should be planned where tree roots can adversely impact structures. Proper jointing should be used to control cracking. The contractor should take care in coordinating joint spacing, steel lay-out, and doweling as needed. Joint spacing should not exceed 10 feet or 1.25 times the narrow width (i.e., a 5-foot spacing for a 4-foot wide walkway), whichever is less. Joints should be one-third to one-fourth the slab thickness taking care not to cut any steel. Minimum 3/8-inch expansion joint material should be placed wherever new flatwork must adjoin existing concrete, foundations, and/or retaining walls. Closed-cell foam is considered the best material for use in expansion joints. As with interior concrete, all steel and concrete materials, details, placement procedures, and curing should be performed strictly in accordance with ACI specifications and guidelines.

Special detailing may be necessary to limit unsightly cracking at structural interfaces, such as between foundations and adjacent slabs. Appurtenant structures placed near slope tops could creep over time in response to slope movement. Special detailing can be provided to help minimize the effects of slope influence. This might include structurally tying exterior slabs to the foundation and/or providing a thicker, heavily reinforced section. Architectural separations (e.g., planting strips) can also be provided around appurtenances to reduce cracking and unsightly separations. The project architect and/or structural engineer should develop the actual details.

Because of the substantial pressures that expansive soil can exert, it is not possible to prevent flatwork from experiencing differential movement (uplift during swelling and dropping during shrinkage) when exposed to changes in moisture content. As such, enhanced drainage measures and special detailing and reinforcing plus architectural separations should be provided to limit apparent damage. Between appurtenant walls and the main buildings, at least 2 inches of architectural separation should be planned. Two inches allows for stucco finish or paint to be added within the joint.

8.6 RETAINING WALL DESIGN CRITERIA

Where cantilever, free to rotate (unrestrained) retaining walls are planned, they should be designed utilizing the following design criteria:

Cantilever Walls (Unrestrained):

Active Soil Pressure (Level Backfill)...65 pcf EFP*
Active Soil Pressure (2:1 Backfill)...80 pcf EFP*
Expansive Soil Surcharge in Upper 4 feet..60 pcf EFP*
Passive Soil Resistance ....250 pcf EFP*
(Ignore lateral support over upper 2 feet and 25 feet HDTDL)
Allowable Coefficient of Friction..0.3
Allowable Bearing Pressure.. 2500 psf

(3 feet minimum embedment)

At-rest earth pressure – level backfill .... 125 pcf

At-rest earth pressure – 2:1 ascending slope... 140 pcf

Note: * EFP: Equivalent Fluid Pressure

Note: ** Values given above are for active earth pressure conditions (i.e., expect up to about 1% wall rotation). For At-Rest (restrained) conditions, use combined 125 pcf EFP with pressure over effective wall entire height.

For passive earth pressure, 250 pcf should be considered if the proposed retaining walls provided lateral foundation support adjacent to a descending slope is ignored over 2 feet and within 25 feet HDTDL. The 250 pcf EFP should be extended down as a stress distribution below the ground surface with actual support over the distance specified above. Allowable coefficient of friction should be 0.3; reduce lateral bearing by one-third when combining with friction. Maximum allowable vertical bearing should be 2,500 psf at 36-inch minimum embedment.

The allowable bearing pressures are for dead plus long-term live loads and include a factor-of-safety of at least 3.0. The coefficient of friction should be applied to dead load forces only. Footings can be designed to resist lateral loads by using a combination of sliding friction and passive resistance.

Retaining wall design criteria have been provided utilizing Sections B-B', I-I', and K-K' of the Tentative Tract Map plans and are illustrated in **Plates 4, 5, and 6**, respectively, as well as within report Section 8.6 herein.

Per 2019 California Building Code (CBC), seismic earth pressure should be included in the design for retaining walls exceeding 6 feet in height. A minimum horizontal seismic coefficient corresponding to one-half of two-thirds of the PGA_M (Maximum Considered Earthquake-Geometric Mean, MCE_G adjusted for Site Class effects) was used to determine the seismic active pressure (i.e., horizontal seismic coefficient = 0.5*(2/3 of PGA_M)). A seismic earth pressure of 18 pounds per cubic foot (pcf) should be included in the wall design. For simplicity, the dynamic (seismic) earth pressure can be applied as a uniform pressure (i.e., rectangular distribution) equal to 9 times the retaining height of the wall (i.e., 9H, where H is the height of the basement walls in feet). This pressure is in addition to the static design wall load and is for level backfill behind the wall. The allowable passive pressure and bearing capacity can be increased by one-third for seismic conditions.

Appropriate allowances should be made for anticipated surcharge loading. It is assumed that the project design engineer will incorporate an appropriately designed wall backdrain system for the purpose of mitigating potential for hydrostatic and/or seepage forces. The minimum width for granular backfill is 1.0 foot. Backdrain pipe should be positioned at the heel of the retaining wall foundatons with at least a 1 percent drainage gradient to suitable disposal areas wherever possible.

It should be pointed out that the use of heavy compaction equipment in close proximity to retaining walls can result in excess wall movement and/or soil loadings exceeding design values. In this regard, care should be taken during backfilling operations.

In order for these soil design parameters to be valid, all planned retaining walls should be properly designed and detailed. All walls should be provided with an adequate backdrain system and a clean, coarse grained, non-expansive backfill for a width of at least 12 inches or suitable alternative such as continuous Miradrain.

All retaining walls should be waterproofed from above the highest point of earth retained to the heel of the foundation or pile grade beam. <u>The architect should provide details for waterproofing including termination details and provisions for protecting the waterproofing (i.e., protection board and corrosion resistant flashing)</u>. Each retaining wall should be provided with an appropriate backdrain system designed by the project architect or civil engineer. It is recommended that the backdrain system extend to the heel of the foundation and at least 1-foot below any of the interior slab elevation (where applicable). Water collected in the backdrain system should ideally be recovered in a perforated SDR35 or Schedule 40 PVC plastic pipe (perforations down) and directed to a suitable disposal area of at least one percent gradient unless otherwise specified by the project civil engineer. If desired, our office can design the walls, back drainage, water proofing protection board and termination details.

Retaining wall backfill should be placed in thin lifts (6 to 8 inches) and compacted by mechanical means. Care should be taken not to utilize heavy compaction equipment in close proximity to the walls to help reduce the possibility of damage to the wall and an increase in the above recommended earth pressures.

Unless walls are designed for expansive soil swelling pressure, walls should be provided with non-expansive, free draining backfill for a width equal to the height of the wall.

8.7 MECHANICALLY STBILIZED EARTH (MSE) RETAINING WALL

The proposed Mechanically Stabilized Earth (MSE) gravity retaining walls should be designed in accordance with the criteria provided below. The MSE wall using import, non-expansive, free-draining, granular soil with Expansion Index (EI) less than 20 for backfill material in reinforced zone may be designed as gravity retaining wall using the following design parameters:

Wall Batter (vertical:horizontal)..1:6 max.
Design Wall Height (from Top of Wall to Finish Grade) .... 15.5 feet max.
Minimum footing embedment depth ....3 feet
Active Soil Pressure (Level Backfill) ....65 pcf EFP*
Active Soil Pressure (2:1 Backfill)..80 pcf EFP*
Active Soil Pressure (2:1 Existing Buttress Slope)............................140 pcf EFP*
Expansive Soil Surcharge in Upper 4 feet ....60 pcf EFP*
Passive Soil Resistance .... 250 PCF EFP*
(Ignore over upper 2 feet for no lateral support and 25 feet HDTDL)
Allowable Coeffifient of Friction 0.3
Allowable bearing pressure
    a) Sustained loads...2500 psf
    b) Total loads (including wind or seismic) ....3325 psf
Reinforced Zone Soil:
        Cohesion .... 0 pounds per square feet (psf)
        Friction Angle ....30 degrees
        Total Unit Weight ....120 psf
Foundation Soil:
        Cohesion .... 0 pounds per square feet (psf)
        Friction Angle ....30 degrees
        Total Unit Weight ....120 psf
Minimum Width of Reinforced Zone:
        Level Backfill ....Reinforced Zone Height, H
        2:1 Backfill ....1.5 times Reinforced Zone Height, 1.5H

Note: * EFP = Effective Fluid Pressure

The allowable bearing pressures are for dead plus long-term live loads and include a factor-of-safety of at least 3.0.

MSE retaining wall design criteria recommended above have been provided utilizing Section H-H', J-J', and L-L' of Tentative Tract Map plan and are illustrated in **Plates 7, 8, and 9**, respectively, as well as within report Section 8.7 herein.

The import soil used as reinforced zone backfill soil material should be tested and verified by the project geotechnical engineer and compared with the design shear strength parameters for reinforced zone soil material at the time of construction. The wall designer should also consider slope creep influence and expansive soil influence of both slope and foundation soils into wall design.

8.8 POST CONSTRUCTION MOVEMENT

Once a project is complete, soil movements can occur for various reasons including minor settlement, expansive soil influence, and creep. As mentioned earlier, expansive soil influence is considered the most significant influence impacting the design of slab and foundation systems at the project. Settlement is not expected to be a significant factor, however soil subsidence associated with drying can occur. To mitigate against problems associated with swelling and shrinking, mat slab foundation systems have been recommended. The designer should give careful consideration to the design of appurtenant improvements near or immediately adjacent to the main buildings and near slopes. Because appurtenant structures and the main building will commonly behave differently, even in the face of similar soil conditions, it is strongly recommended that no appurtenant walls, slabs, or otherwise be tied directly to the main buildings. If privacy walls are proposed, they should be placed on foundation systems specifically designed for their purpose, but the walls themselves should be separated from the main structure a nominal distance of about 2 inches which would allow for painting and stucco applications. This separation will form an architectural gap which with movement in the future would not produce unsightly damage. The same concept is true with respect to slabs-on-grade. It is not recommended patio slabs or stoops be placed directly adjacent to the building concrete. Patios should be separated from the main building by approximately 4 to 6 inches in which river rock, wood chips, low-growing vegetation, or similar would be placed. Such patios should not extend to adjacent masonry site walls, retaining walls, or otherwise. Similarly, a gap should be created between walls and flatwork. In a typical rear patio, the homeowner would probably want about 1-foot separation in order to accommodate minor landscape in the patio perimeter area. Designing appurtenant structures with this style of flexibility allows for post-construction soil movement without having unsightly stucco cracking, patio separations, or other stress features which would normally alarm homeowners. Minor movement can be expected. Creep at the site, as measured via inclinometers, has been on the order of .05 inches per year or less, well below thresholds generally worthy of concern. Nonetheless, that kind of creep over sufficient years could result in stucco cracking or patio separations which are unsightly and to some people alarming.

Whether via drought, wet years, or normal rainfall years, variations will occur in the soil supporting appurtenances over the course of any year or years. Minor movements associated with them can be expected to cause relative movement between adjacent structures and as such the concept of adding flexibility is strongly encouraged. Instead of “fighting Mother Nature,” we advise the client and future homeowners to “work with Mother Nature” to provide a little give, “wiggle room” in the appurtenant improvements.

In recent years, the thing most impacting projects in Orange County, California, as viewed by this consultant has been drought-related soil shrinkage. Simply stated, as moisture gradually is lost in soil supporting improvements, subsidence will occur when that soil is significantly expansive. The site soil is typically, significantly expansive. As such, moisture loss can result in subsidence which results from the shrinkage of the material. At this site, the project has been sitting in place for over 15 years, and as such significant drying has likely occurred. In this case, we will be adding fill to the site, on the order of 5 feet, and typically less than 10 feet. This new material will be more prone to drying than the older materials left in place. To mitigate against significant post-construction related soil movements including drying, care should be taken to plan irrigations systems that do not overwater and yet do not underwater either. Consideration should be given where any large tree is proposed near improvements. While the buildings themselves have been recommended to contain a perimeter moisture barrier that will serve as a root barrier, large trees adjacent pavement, curbs, flatwork, and other such improvements could cause long-term heaving from root flare disruption or subsidence associated with moisture extraction. The Association should be provided guidance with regard to maintenance provisions. It is likely that over the years, appurtenant flatwork sections will require removal and replacement, possibly curbs and gutters would as well. The processes impacting these appurtenances will be very long-term and as such, insidious.
Cases arise where the architect desires to put an appurtenant piece of flatwork immediately adjacent the building, such as at a garage side door stoop. Such construction could proceed; however, it would be

recommended that a flexible joint be created between the building and the stoop concrete. This type of joint can be best accomplished by providing isolation joint/expansion joint material between the stoop and the main structure. If it is desired that a connection be made to limit separation due to expansive soil and/or other soil-related influences, dowels could extend from the main foundation into the stoop concrete. To care for corrosion at the interface, we suggest the use of epoxy coated dowels plus a polymer sheathing could be placed over the dowel in interface area. We recommend the sheathing extend into each concrete element at least 3 inches. For a typical 1/2-inch expansion joint material example, it might be useful to simply cut 8-inch nominal pieces of vinyl or similar UV resistant hose to form that central slip connection. Care should be taken to provide expansion joint material wherever new concrete must be adjacent to existing concrete. As aforementioned, separating the concrete plus providing river rock, wood chips, or minor landscape strips is the best, but in some cases like at garage entries, a close connection to the concrete can be provided. In those situations, the construction should be handled with the kind of doweling aforementioned. Flatwork concrete should be adequately jointed. Typically, joints should be spaced less than 10 feet and not more than 1.25 times the minimum dimension of any concrete panel. Control joints should extend at least one-quarter to one-third of the concrete section taking care not to interrupt any embedded steel.

Expansive soil in hillside areas can be considered somewhat of a dynamic construction environment. Designing improvements that absolutely stop, resist soils movements are inordinately expensive and impractical. Designing improvements with separations and flexibility, as described above, is considered a positive and practical approach. Since backyards are going to be relatively shallow, no special considerations need to be made with respect to drainage. For conventional construction, with very large backyards, long-term creep and soil-related movements as well as drying and shrinkage can adversely impact long-term drainage gradients to suitable disposal areas, however as aforementioned backyards are relatively small and drainage in those areas should be fairly efficiently collected and directed to a suitable disposal area such as the street in front of the buildings. This consultant is not adverse to directing drainage to areas below slopes as determined practical by the project civil engineer. This office should review fine-grade plans in relation to the location of probably appurtenances when such plans are available. All homeowners should be made aware that to some extent future soil movement can occur with proper design and construction. Adverse impacts should be minimal. The landscape architect should plan on ample yard drains. Around unit patios, 4 to 6-inch atrium grates at about 8 feet o.c. should be planned.

8.9 PRELIMINARY PAVEMENT DESIGN

Based on our experience on similar projects with similar soil conditions, the pavement section given on **Table 4** can be used for determining preliminary cost estimates. A Traffic Index (TI) of 5.0 for light traffic and an R-Value less than 10 was assumed for the on-site roadways and alleys.

TABLE 4 – PAVEMENT DESIGN

TI	Asphalt Concrete (in)	Class 2 Aggregate Base (in)
5.0	4[(1)]	10

(1) Preferably 8-inch net reinforced concrete sections could be used in trash pick-up areas and down-sloping vehicle braking areas. The concrete should be provided with at least 6 inches of compacted aggregate base material (e.g., S.S.P.W.C.200.2.2).

The aggregate base (AB) and upper 12 inches of subgrade should be compacted to 95 percent of the maximum dry density as determined by ASTM D-1557. For critical areas such as the entry and trash pick-up areas, more detailed rigid pavement (concrete) designs can be provided upon request. Preliminarily, for planning purposes, use 4500 psi, Type V concrete (0.45 max w/c) 8 inches net thickness, containing No. 6 bars at 12 inches o.c.e.w, and underlain by 6 inches of compacted crushed rock base. Mirafi 140 or similar overlapped at least 6 inches at splices should be placed on the clean, compacted subgrade before base rock is placed.

All fill beneath the subgrade (including trench backfills) should be compacted to a minimum of 90 percent of the maximum dry density per ASTM D-1557. Our firm should observe and test the compaction of subgrade and base material. Final pavement design can be provided once rough grading is completed and near surface soil is tested.

8.10 SITE DRAINAGE

Proper surface drainage should be incorporated into the design for the proposed project. Because of potential problems associated with poor drainage conditions, proper surface drainage should be maintained at all times. As a minimum, the following standard drainage guidelines are recommended and should be considered by the builder/developer and civil engineer during final plan preparation:

A. Roof drains should be installed on all structures and tied via a "tight line" to a drain system that empties to a storm drain, terrace drain, or other suitable disposal area.

B. Surface water should flow away from structures and slopes and be directed to suitable (maintained) disposal systems, such as yard drains, drainage swales, street gutters, etc. Five percent drainage directed away from structures is recommended. Five percent is also advised for planter areas.

C. At least two (2) percent drainage is recommended for sheet flow in any soil areas.

D. No drains should be allowed to empty adjacent foundations or over slopes.

E. PVC Schedule 40 sealed at all joints or equivalent is preferred for yard drains. A corrugated plastic yard drain pipe is not recommended. SDR35 pipe could be used with all joints sealed, provided inspections are conducted, pipes regularly cleaned and repaired, and provided it is recognized by all concerned parties that SDR35 pipe is at far more risk of root damage than Schedule 40 PVC.

8.11 MAINTENANCE

Any on-site or off-site surface drainage devices (e.g., terrace drains, downdrains, etc.) should be maintained on a regular basis. Any debris which accumulates within the drainage devices should be periodically cleaned (e.g., twice a year) to provide that all on-site or off-site surface drainage devices will always function as intended. In addition to periodic cleaning of drainage swales, local crack filling with durable resilient mastic is advised and can be expected. Long-term, local areas of swale replacement may be necessary. Long-term local replacements tend to be dictated by long-term tree influences and slope creep. Although large scale slope instability is not considered likely, even in the event of poor maintenance, local swale undermining, erosion gullies, and superficial instability could occur and will require maintenance. In addition to swale maintenance, burrowing animals should also be controlled.

The streets should be maintained and inspected on a regular basis (e.g., every two years). Annual reviews by our office on behalf of the new HOA are encouraged. The future homeowner association should seal the street pavements on an as-needed basis. If the association desires an attractive black pavement, a slurry seal should be anticipated every two to three years. A more effective, but less attractive Type I seal (contains fine aggregate to about 1/8-inch and heavier, more viscous asphalt oil) could be planned every 4 to 6 years. A reasonable combination of seals could be estimated by two cycles of slurry seal (oil only) at the above-stated interval, followed by a Type I seal. As a pavement ages, a Type I seal will tend to better fill the voids associated with a long-term roughening of the pavement. By ten years, some areas of patching can be expected. By twenty years, larger areas of patching, repaving, and probably an overlay can be expected. All irrigation utilities and surface drainage should be periodically inspected, cleaned, and repaired, as needed.

8.12 UTILITIES

It is not recommended that utilities be planned below a 1:1 projection extending down from the outer edge of foundations. Footings should be deepened to satisfy the foregoing recommendation. Backfill for all utilities should be placed by mechanical compaction methods. Flooding and/or jetting of utility or other trench backfill are not recommended.

8.13 CORROSIVITY

Site soil is recognized to be corrosive to concrete and metal. This type of corrosive soil is common to the area. Consideration will need to be given to the durability of concrete and steel (embedded in concrete and not embedded). For example, vinyl weep screed and epoxy coated anchor bolts are better alternatives to galvanized metal and increasing concrete coverage for improved corrosion protection. The use of relatively good quality concrete consistent with the corrosive conditions is advised (e.g. Type V cement, 0.45 maximum water-to-cement ratio (0.42 preferred) and 4,500 psi minimum (5,000 psi preferred)).

With respect to buried piping, it is considered good construction practice to provide corrosion protection by means of a suitable coating. Placing pressure-plumbing overhead instead of under slabs is desirable. Laboratory tests performed on soil obtained from the site indicate that concrete should be designed for sulfate in accordance with the “Severe” category of ACI 318 Table 4.2.1 and the requirement of the 2010 California Building Code. Test results also indicate a “high” corrosion potential to buried metals. Even embedded metal is at risk. The Code requirement for embedded metal is to increase coverage, increase concrete density, and/or provide some other approach. As mentioned earlier, providing epoxy-coated anchor bolts with specified minimum coverage (i.e., 2.5-inch coverage for 5/8-inch anchor bolts; 6x exterior framing) is considered a suitable approach for anchor bolts. The use of epoxy-coated reinforcing in conjunction with Code minimum bar clearances would also be appropriate (i.e., with 2x4 exterior framing). Recommendations should be provided by the project designers to address possible problems related to these findings. Other approaches include lower concrete w/c ratio (e.g., 0.42 maximum) and using vinyl weep screed. The use of 20d stainless steel nails for anchoring PT hardware is also recommended. Alternatively, “no-nail” PT hardware assemblies can be used. Where PT detailing is adopted, PT hardware at live ends should be cleaned and epoxy bonded prior to application of a stiff, low water-to-cement ratio structural mortar cap. It is also recommended that unless the PT hardware is encapsulated (e.g., fully protected by durable, water tight plastic coating including a sealed plastic cap over the cable ends. Grout pockets should be cleaned, epoxy bonded and filled with a chloride free low permeability, high-strength grout (e.g., SikaGrout 212).

Simpson-style STHD hold-downs should only be used if specifed in stainless steel. Anchor bolts should be stainless steel or exposy coated, mild steel below the top, 1 inch of threaded sections.

8.14 GEOTECHNICAL SERVICES DURING CONSTRUCTION

A representative of this office should be on hand during construction to provide observation and testing services. As previously recommended, our representatives should be on-site full-time during grading. Full-time monitoring during excavations related to remedial grading is also recommended and should be carried out in accordance with the accompanying report. We recommend providing observation of all foundation excavations prior to the placement of forms, reinforcement, or concrete. Furthermore, it is recommended that our office be requested to review the slab subgrade areas and perform any necessary testing prior to the placement of concrete. Concrete suppliers should provide long-form delivery tickets for all concrete delivered. The long-form tickets should clearly indicate the mix proportions, the design water-to-cement ration, and concrete strength at 28 days. The delivery tickets should also clearly indicate the amount of water, if any, that can be added to the mix at the site.

8.15 LANDSCAPE

Generally, native species requiring low to moderate irrigation are preferred over non-native species requiring high irrigation, but special care or special detailing should be provided for high shrubs or trees within 20 feet of foundations. Very drought-tolerant species such as prostrate acacia and eucalyptus trees should be avoided or used with caution since they have high suction capacity and tend to over-dry soil masses. Since planning of similar projects requires selected tree species relatively close to buildings, tree roots will represent a significant long-term risk of root-related heave and expansive soil-related subsidence. At least 4-foot deep polymer root barriers should be installed around buildings (see Figure 4). Membrane cut-off walls/root barriers have been recommended for the main buildings. Similar details can be adapted for more critical appurtenances. Details can be provided following the development of preliminary foundation details. Other details to reduce risk to appurtenances can be developed once preliminary landscape plans are available. The landscape architect should take care in considering the primary long-term geotechnical influences, creep and expansive soil. We would encourage the project landscape architect to arrange a meeting with our geotechnical team in order to discuss geotechnical issues.

8.16 PLAN REVIEW

When final plans are available they should be forwarded to our office for review and comment. When acceptable, we will add an engineer's signature and professional stamp indicating compliance with the intentions of the recommendations contained in this report. If any of the other design professionals or construction members have any questions regarding the site geotechnical conditions or the recommendations of this report, our office should be contacted.

8.17 PROJECT SAFETY

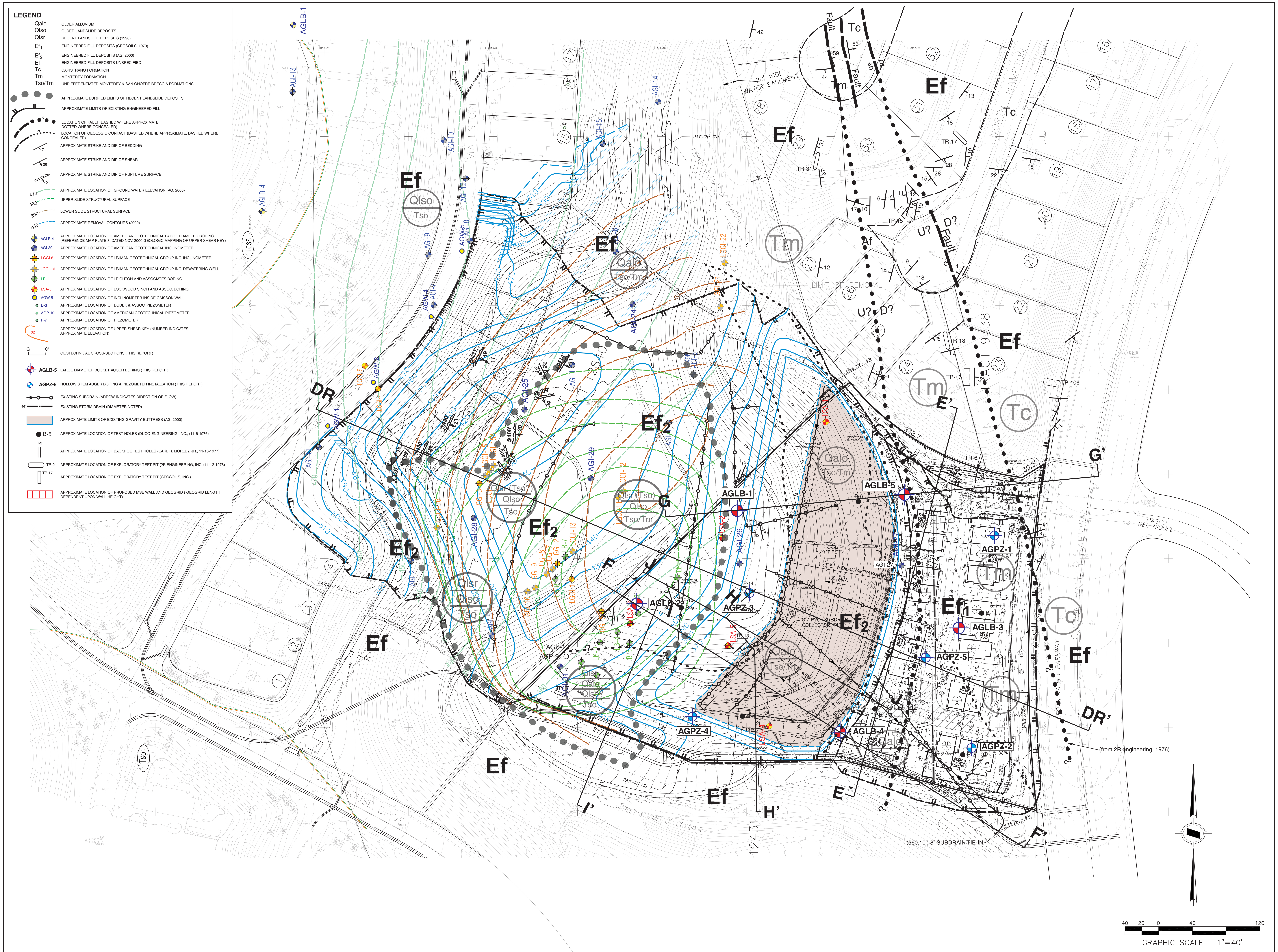
The contractor is the party responsible for providing a safe site. American Geotechnical will not direct the contractor's operations and cannot be responsible for the safety of personnel other than our own representatives on-site. The contractor should notify the owner if he is aware of and/or anticipates unsafe conditions. At the time of construction, if the geotechnical consultant considers conditions to be unsafe, the contractor, as well as the owner's representative, will be notified.

9.0 REMARKS

Conclusions, recommendations, and/or other information contained in this report are based upon the subsurface investigation conducted at the site and the assumption that subsurface conditions do not vary appreciably between observation points. Although no significant variation is anticipated, it must be recognized that variations can occur.

This report has been prepared for the sole use and benefit of our client. The intent of the report is to advise our client on geotechnical matters involving the proposed development. It should be understood that the geotechnical consulting provided and the contents of this report are not perfect. Any errors or omissions noted by any party reviewing this report, and/or any other geotechnical aspects of the project, should be reported to this office in a timely fashion. The client is the only party intended by this office to directly receive this advice. The client can only authorize subsequent use of this report.

Conclusions and recommendations presented herein are based upon the evaluation of technical information gathered, experience, and professional judgment. Other consultants could arrive at different conclusions and recommendations. Final decisions on matters presented are the responsibility of the client and/or the governing agencies. No warranties in any respect are made as to the performance of the project. As the project develops further, the conclusions and recommendations herein should be considered subject to review and updating.



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PRELIMINARY GEOTECHNICAL/GEOLOGIC MAP

THE COVE AT EL NIGUEL TENTATIVE TRACT NO. 17721

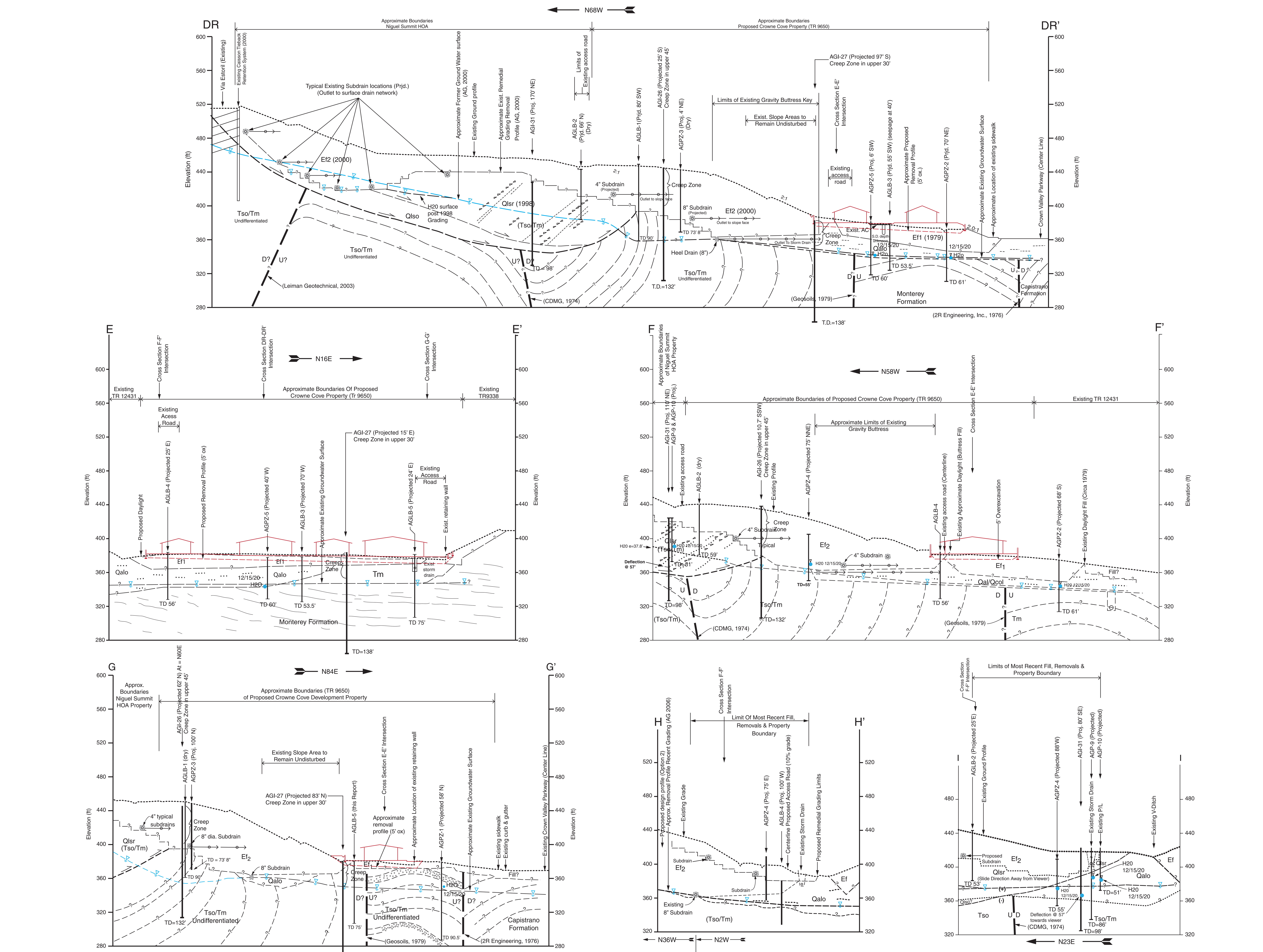
SITE DEVELOPMENT PLAN

LAGUNA NIGUEL, CALIFORNIA

File No. 33348.08

Date : JAN 2021

PLATE 1



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GEOTECHNICAL CROSS SECTIONS DR-DR', E-E', F-F', G-G', H-H' & I-I'

THE COVE AT EL NIGUEL TENTATIVE TRACT NO. 17721 SITE DEVELOPMENT PLAN

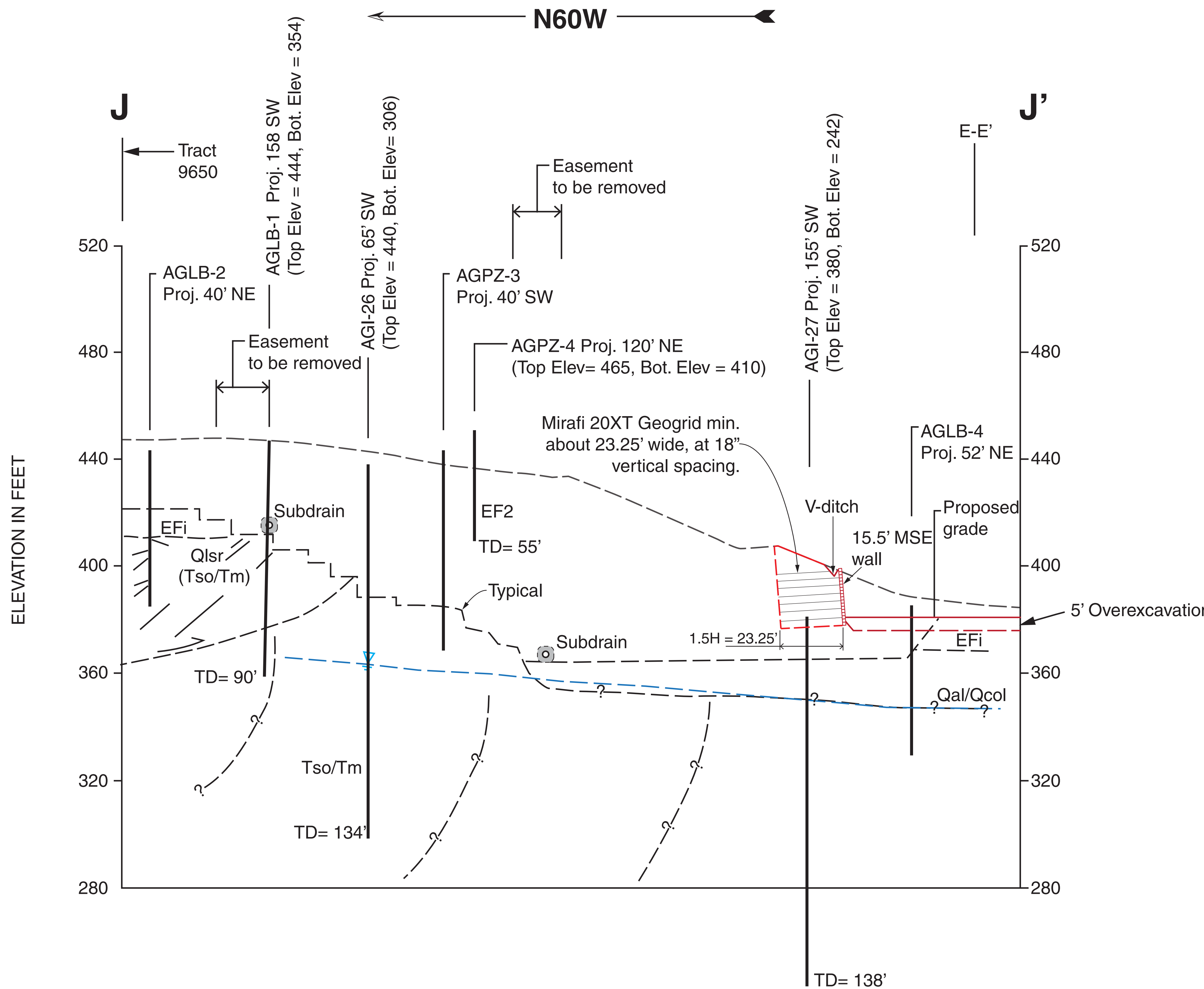
30667 CROWN VALLEY PARKWAY

LAGUNA NIGUEL, CALIFORNIA

File No. 33348.08

Date : JAN 2021

PLATE 2



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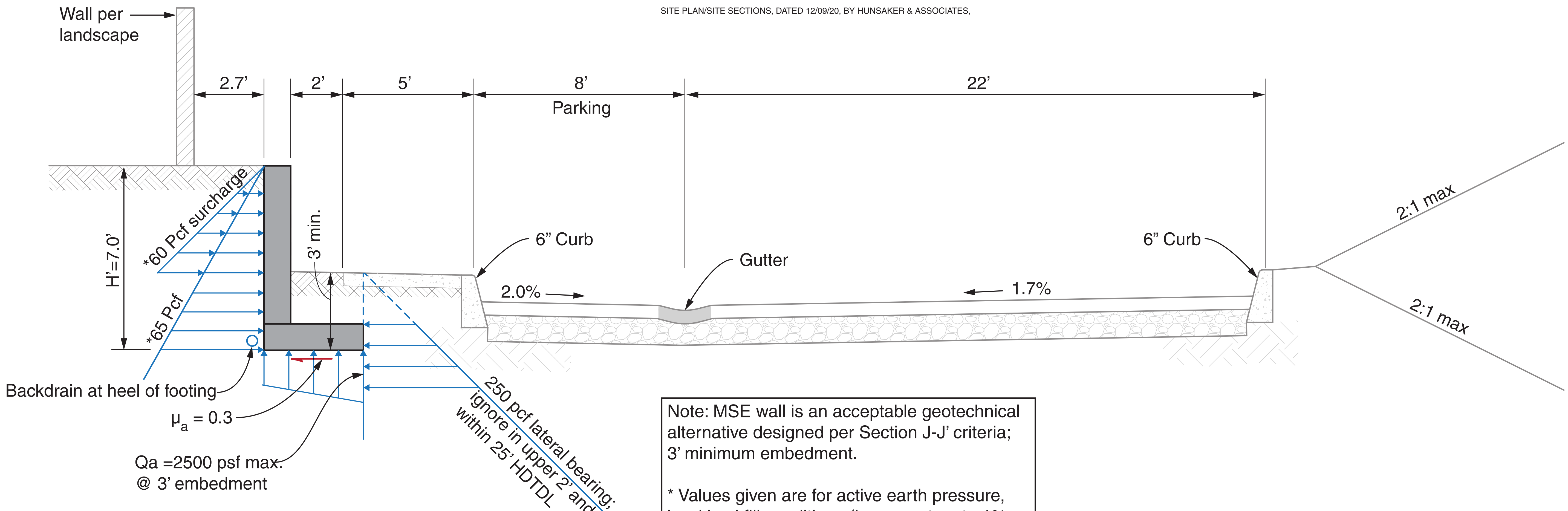
GEOTECHNICAL CROSS SECTION J-J'
THE COVE AT EL NIGUEL TENTATIVE TRACT NO. 17721
SITE DEVELOPMENT PLAN, LAGUNA NIGUEL, CA

SCALE:	DATE:	FILE NO.:
1" = 40'	JAN 2021	33348.08

PLATE
3

SECTION B-B'

REFERENCE: SHEET SP-2, THE COVE AT EL NIGUEL, TENTATIVE TRACT NO. 17721
SITE PLAN/SITE SECTIONS, DATED 12/09/20, BY HUNSAKER & ASSOCIATES,



Note: MSE wall is an acceptable geotechnical alternative designed per Section J-J' criteria; 3' minimum embedment.

* Values given for active earth pressure, level backfill conditions (i.e. expect up to 1% wall rotation). For "At-Rest" soil conditions, use 125 pcf total equivalent fluid pressure over effective wall entire height.

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TITLE: **RETAINING WALL DESIGN CRITERIA**
THE COVE AT EL NIGUEL

SCALE:	DATE:	FILE NO.:
1" = 4'	JAN 2021	33348-08

PLATE **4**

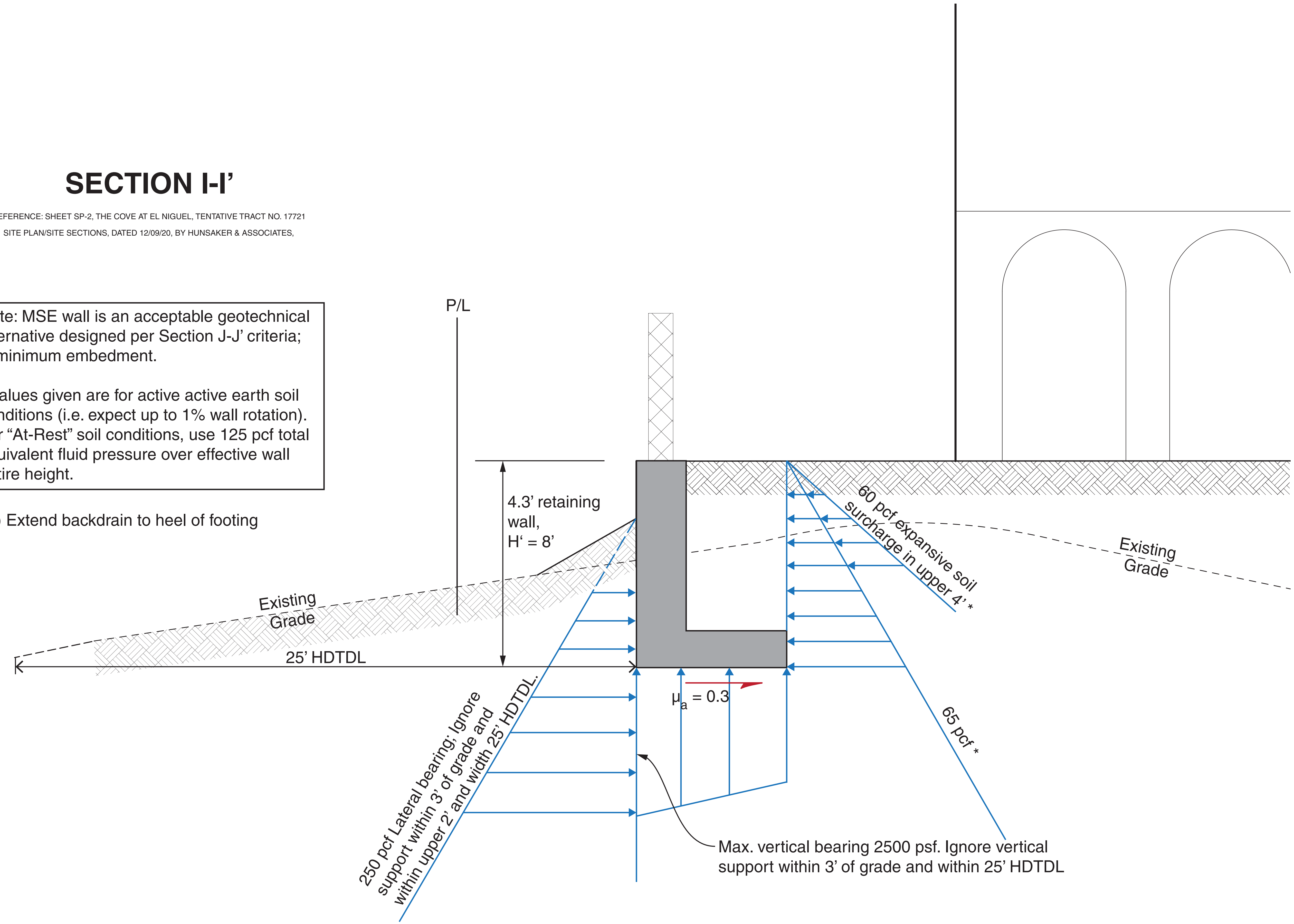
SECTION I-I’

REFERENCE: SHEET SP-2, THE COVE AT EL NIGUEL, TENTATIVE TRACT NO. 17721
SITE PLAN/SITE SECTIONS, DATED 12/09/20, BY HUNSAKER & ASSOCIATES,

Note: MSE wall is an acceptable geotechnical alternative designed per Section J-J’ criteria; 3’ minimum embedment.

* Values given are for active active earth soil conditions (i.e. expect up to 1% wall rotation). For “At-Rest” soil conditions, use 125 pcf total equivalent fluid pressure over effective wall entire height.

(1) Extend backdrain to heel of footing



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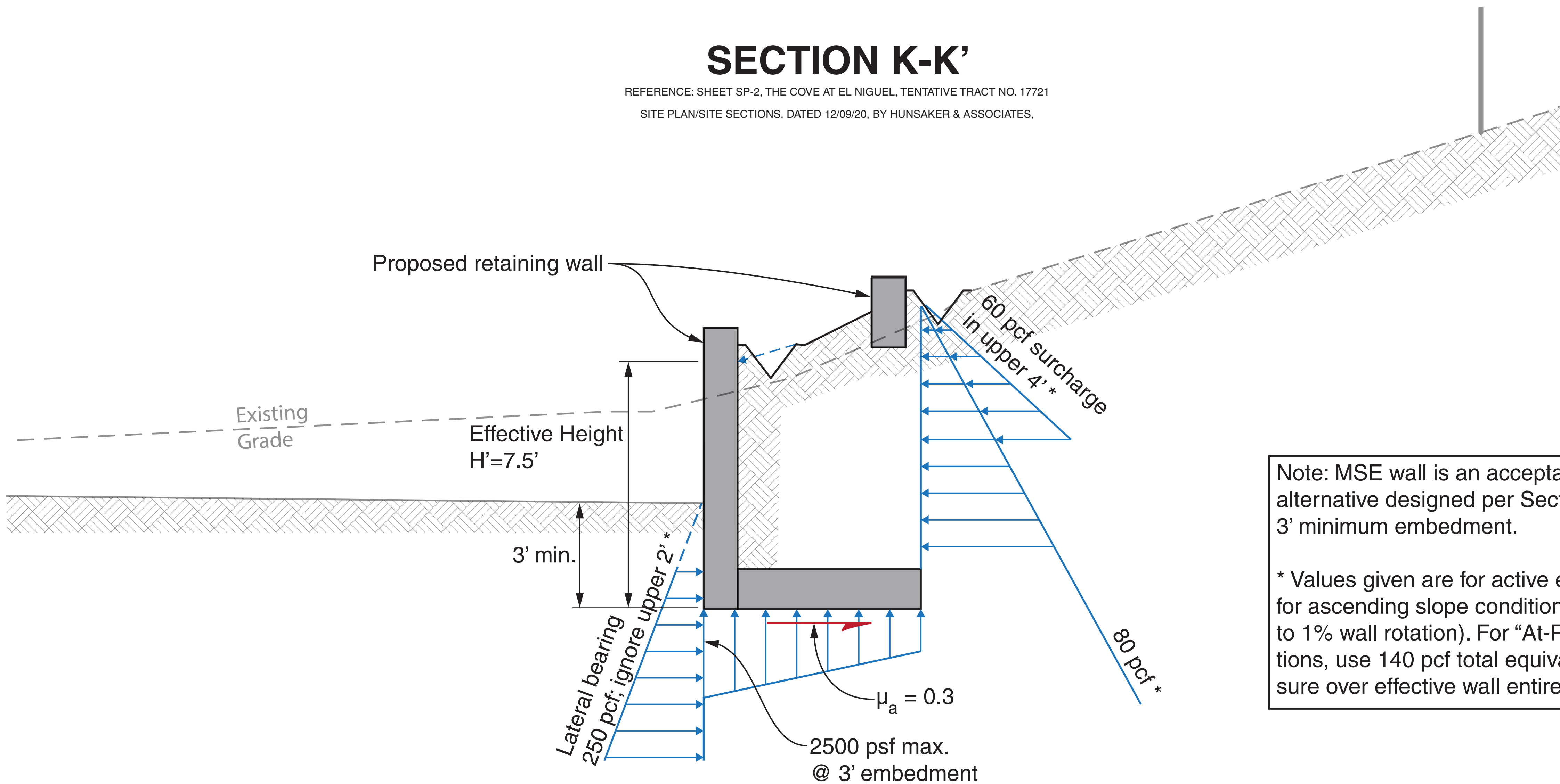
TITLE: **RETAINING WALL DESIGN CRITERIA**
THE COVE AT EL NIGUEL

SCALE:	DATE:	FILE NO.:
1” = 4’	JAN 2021	33348-08

PLATE
5

SECTION K-K'

REFERENCE: SHEET SP-2, THE COVE AT EL NIGUEL, TENTATIVE TRACT NO. 17721
SITE PLAN/SITE SECTIONS, DATED 12/09/20, BY HUNSAKER & ASSOCIATES,



Note: MSE wall is an acceptable geotechnical alternative designed per Section J-J' criteria; 3' minimum embedment.

* Values given are for active earth pressure for ascending slope conditions (i.e. expect up to 1% wall rotation). For "At-Rest" soil conditions, use 140 pcf total equivalent fluid pressure over effective wall entire height.

* Also ignore lateral support within 25' horizontal distance to daylight (HDTDL).

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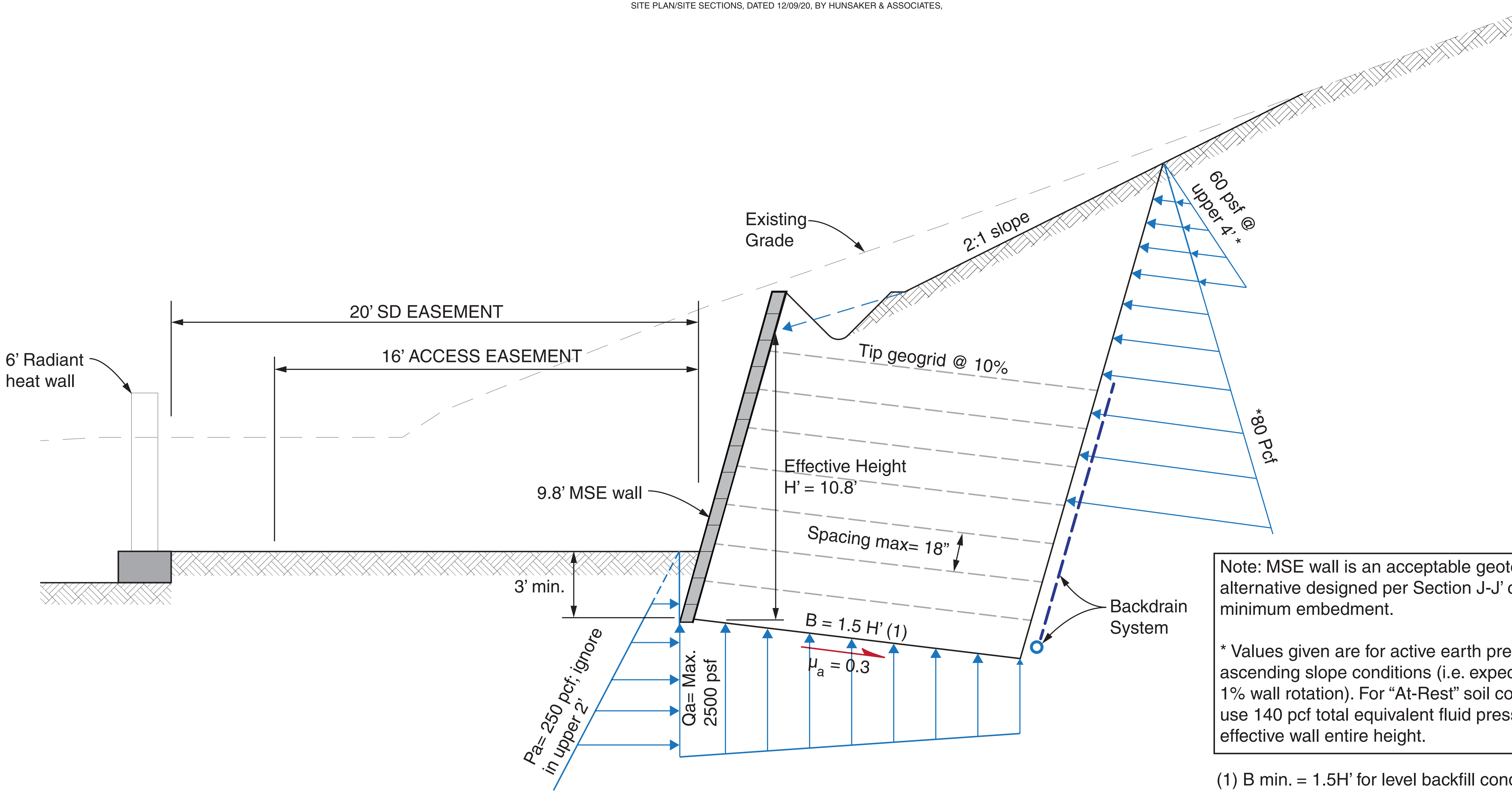
TITLE: RETAINING WALL DESIGN CRITERIA THE COVE AT EL NIGUEL			PLATE 6
SCALE: 1" = 4'	DATE: JAN 2021	FILE NO.: 33348-08	

SECTION H-H'

REFERENCE: SHEET SP-2, THE COVE AT EL NIGUEL, TENTATIVE TRACT NO. 17721
SITE PLAN/SITE SECTIONS, DATED 12/09/20, BY HUNSAKER & ASSOCIATES,



Note: MSE wall is an acceptable geotechnical alternative designed per Section J-J' criteria; 3' minimum embedment.

* Values given are for active earth pressure for ascending slope conditions (i.e. expect up to 1% wall rotation). For "At-Rest" soil conditions, use 140 pcf total equivalent fluid pressure over effective wall entire height.

(1) B min. = 1.5H' for level backfill conditions,
B min. = 1.5H' for ascending slope

AMERICAN GEOTECHNICAL, INC.
22725 OLD CANAL ROAD, YORBA LINDA, CA 92887
(714) 685-3900 (714) 685-3909
www.amgt.com

TITLE: RETAINING WALL DESIGN CRITERIA
THE COVE AT EL NIGUEL

SCALE:	DATE:	FILE NO.:
1" = 4'	JAN 2021	33348-08

PLATE 7

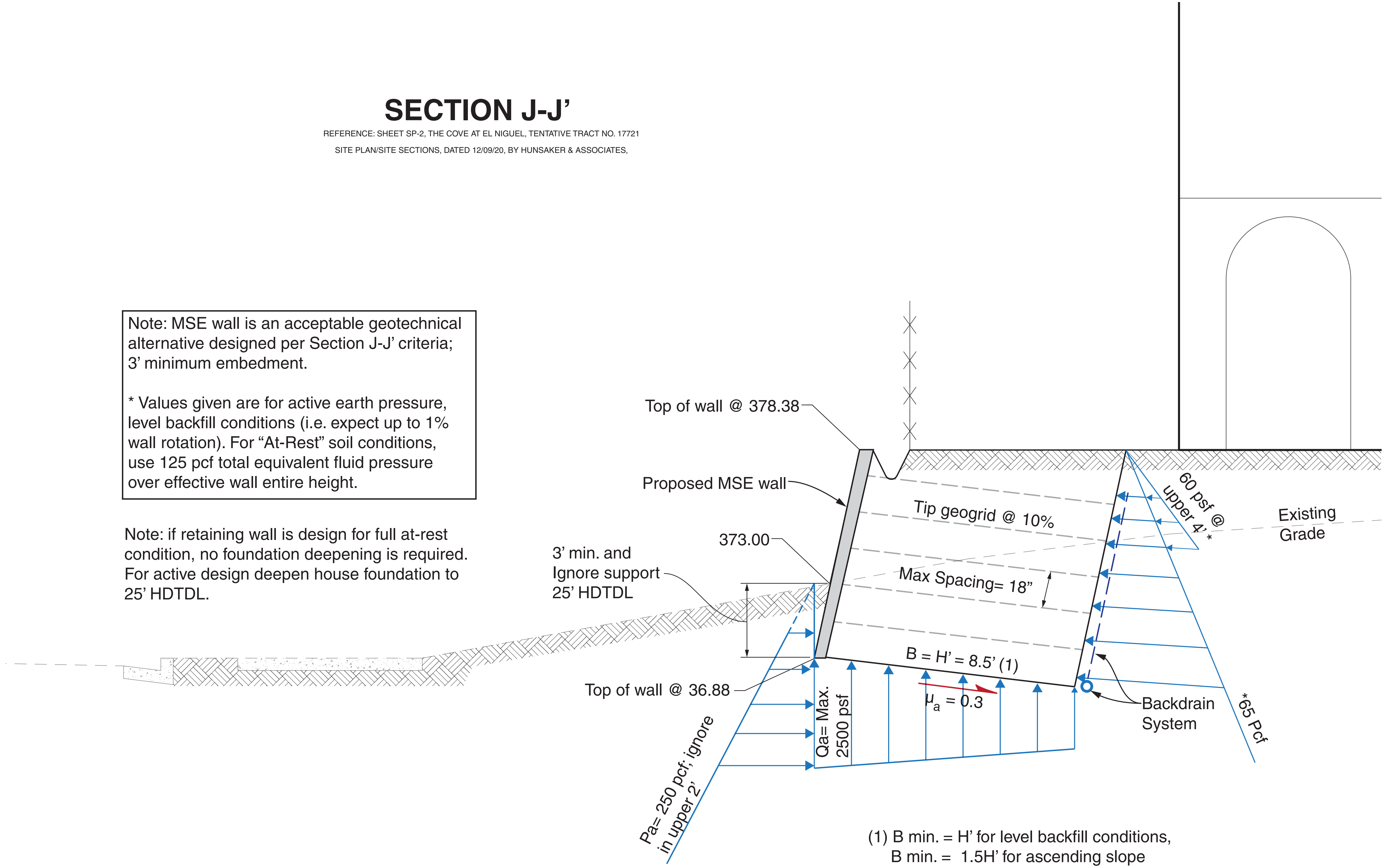
SECTION J-J'

REFERENCE: SHEET SP-2, THE COVE AT EL NIGUEL, TENTATIVE TRACT NO. 17721
SITE PLAN/SITE SECTIONS, DATED 12/09/20, BY HUNSAKER & ASSOCIATES,

Note: MSE wall is an acceptable geotechnical alternative designed per Section J-J' criteria; 3' minimum embedment.

* Values given are for active earth pressure, level backfill conditions (i.e. expect up to 1% wall rotation). For "At-Rest" soil conditions, use 125 pcf total equivalent fluid pressure over effective wall entire height.

Note: if retaining wall is design for full at-rest condition, no foundation deepening is required. For active design deepen house foundation to 25' HDTDL.



(1) B min. = H' for level backfill conditions,
B min. = 1.5H' for ascending slope

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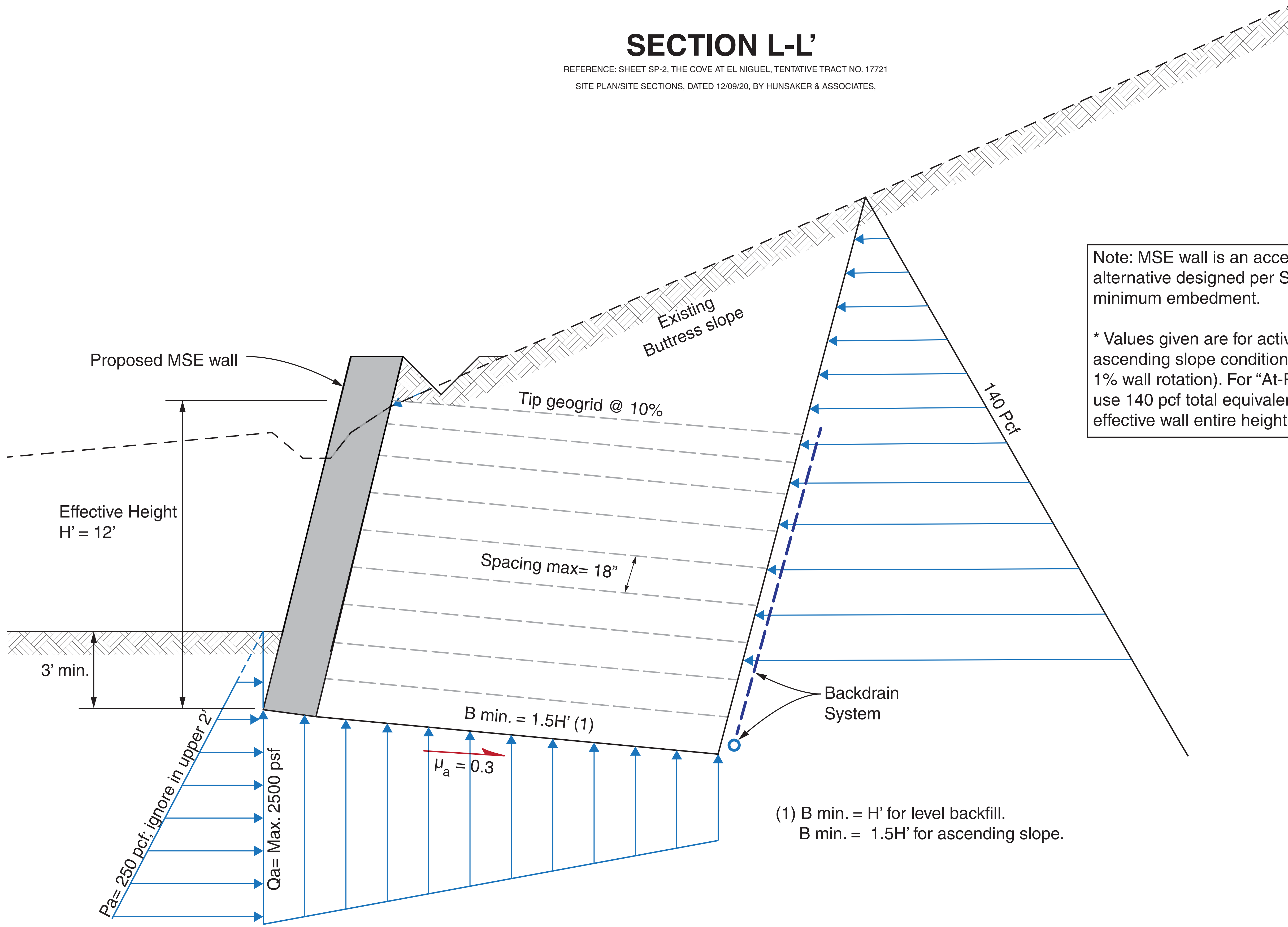
TITLE: RETAINING WALL DESIGN CRITERIA
THE COVE AT EL NIGUEL

SCALE:	DATE:	FILE NO.:
1" = 4'	JAN 2021	33348-08

PLATE 8

SECTION L-L'

REFERENCE: SHEET SP-2, THE COVE AT EL NIGUEL, TENTATIVE TRACT NO. 17721
SITE PLAN/SITE SECTIONS, DATED 12/09/20, BY HUNSAKER & ASSOCIATES,



Note: MSE wall is an acceptable geotechnical alternative designed per Section J-J' criteria; 3' minimum embedment.

* Values given are for active earth pressure for ascending slope conditions (i.e. expect up to 1% wall rotation). For "At-Rest" soil conditions, use 140 pcf total equivalent fluid pressure over effective wall entire height.

(1) B min. = H' for level backfill.
B min. = 1.5H' for ascending slope.

AMERICAN GEOTECHNICAL, INC.
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TITLE: RETAINING WALL DESIGN CRITERIA THE COVE AT EL NIGUEL			PLATE 9
SCALE: 1" = 4'	DATE: JAN 2021	FILE NO.: 33348-08	

APPENDIX A

REFERENCES

REFERENCES

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28. American Geotechnical, Inc., 2012, “Geotechnical Feasibility Study, Proposed Crowne Cove 40-Unit Redevelopment, 30667 Crown Valley Parkway, Laguna Niguel, California,” dated April 18, 2012, prepared for Laguna Niguel Properties, Inc.; File No. 33348.01.

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APPENDIX B

CITY OF LAGUNA NIGUEL

GEOTECHNICAL REVIEW SHEET & NOTICE OF INCOMPLETENESS

CITY OF LAGUNA NIGUEL

30111 Crown Valley Parkway
Laguna Niguel, California 92677
Phone (949) 362-4300 | Fax (949) 362-4340

GEOTECHNICAL REVIEW SHEET

Reviewed by:

GMU GEOTECHNICAL, INC.
23241 Arroyo Vista
Rancho Santa Margarita, CA 92688
(949) 888-6513 · Fax: (949) 888-1380

REFERENCE NO.: SP13-06 DATE: November 6, 2013

PROJECT DESCRIPTION: Proposed Crowne Cove 40-Unit Redevelopment

LOCATION: 30667 Crown Valley Parkway

DEVELOPER/OWNER: Laguna Niguel Properties

GEOTECHNICAL FIRM: American Geotechnical, Inc.

THEIR JOB NO.: File No. 33348-01

GEOTECHNICAL ENGINEER: Gregory W. Axten, G.E. 103, Jerry Huang. G.E. 2601

ENGINEERING GEOLOGIST: Jeff Hull, CEG 2056

DOCUMENT(S) REVIEWED: "Comment on Inclinometer AGI-31, Proposed 38-Unit Condominium Complex and Optional 16-Unit Single Family Residence Development, Laguna Niguel, California", dated October 23, 2013.

REFERENCE DOCUMENT(S): (1) "Preliminary Geotechnical Investigation, Proposed 38-Unit Condominium Complex and Optional 16-Unit Single Family Residence Development, 30667 Crown Valley Parkway, Laguna Niguel, California", dated July 17, 2013; (2) "Geotechnical Feasibility Study, Proposed Crowne Cove Trumark 41-unit Redevelopment, 30667 Crown Valley parkway, Laguna Niguel, California", by American Geotechnical, Inc., dated February 16, 2012; (3) "Geotechnical Feasibility Study, Proposed Crowne Cove 40-unit Redevelopment, 30667 Crown Valley Parkway, Laguna Niguel, California" by American Geotechnical, Inc. dated April 18th, 2012.

ACTION:

____ RECOMMENDED APPROVAL OF DOCUMENT SUBMITTED
_X_ CONDITIONAL APPROVAL OF DOCUMENT SUBMITTED – ***See Comments***
____ REQUEST ADDITIONAL DATA FOR REVIEW

CITY OF LAGUNA NIGUEL
30111 Crown Valley Parkway
Laguna Niguel, California 92677
Phone (949) 362-4300 | Fax (949) 362-4340

GEOTECHNICAL REVIEW SHEET

Reviewed by:

GMU GEOTECHNICAL, INC.
23241 Arroyo Vista
Rancho Santa Margarita, CA 92688
(949) 888-6513 · Fax: (949) 888-1380

REFERENCE NO.: SP13-06 DATE: November 6, 2013

COMMENTS

Project is approved from a planning perspective. The following issues/items will need to be addressed and/or completed prior to the issuance of a grading permit.

1. A grading plan review report will be required.
2. The report should contain all necessary recommendations for proposed grading, wall and foundation systems as well as all necessary technical calculations and justification.
3. All available slope inclinometers will need to be re-read and evaluated. The evaluation shall be contained in the grading plan review report. Inclinometer AGI-31 should be specifically addressed.
4. The existing fill at the back of the development should be evaluated relative to the proposed grading and improvements.
5. The report should contain specific recommendations for pre-construction surveys and monitoring during construction.
6. Maintenance recommendations (i.e., for surface drainage, subsurface drains, streets utilities, etc.) should be provided within the evaluation report and under separate cover for later distribution to homeowners and/or HOA.
7. Full time observation and testing will be required during rough grading operations.

REVIEWED BY:

Gregory Silver, GE 2336
Principal Geotechnical Engineer

Lisa Bates, CEG 2293
Senior Engineering Geologist

DISTRIBUTION:
Mr. Jeff Gibson, City of Laguna Niguel
Mr. Gregory W. Axten, American Geotechnical, Inc.
Laguna Niguel Properties

CITY of LAGUNA NIGUEL
Community Development Department
30111 Crown Valley Parkway • Laguna Niguel, California 92677
Phone/949 • 362 • 4300 Fax/949 • 362 • 4369

October 26, 2016

C. Blair Pruett
Northstar Pacific Partners, Inc.
761 Second Street
Encinitas, CA 92024

Subject: Notice of Incompleteness for Tentative Map TT 17721, Site Development Permit SP 16-04 (Crowne Cove)

Dear Mr. Pruett:

Thank you for your application submittal, dated September 26, 2016, for the proposed development of a 23-unit residential condominium project at 30667 Crown Valley Parkway (Crowne Cove). After review of your initial submittal, in conformance with Government Code Section 65943, this is notification that your application is considered incomplete at this time. In order to fully facilitate the analysis of the proposal by the Community Development Department, the following additional items and/or revisions are necessary:

Issues of Concern:

1. Staff recommends the reduction retaining wall height along the Crown Valley Parkway frontage to the extent possible through the use of multiple, smaller walls, with separation between walls equal to or exceeding the maximum exposed wall height.

2. Retaining wall sections exceeding 12 feet in height are proposed in the interior portion of the project site. Explore options to reduce retaining wall height to a maximum of 12 feet. Where additional height is necessary, please provide justification.

3. Staff recommends inclusion of a sidewalk on at least one side of the internal street for pedestrian circulation and varying unit setbacks to create interest and avoid an alley effect.

4. All homeowners must receive three waste containers to accommodate trash, recycling and green waste. Placement of green waste in trash containers is not acceptable and is in violation of AB 939. In discussion with staff, the applicant suggested that homeowners place waste receptacles right outside their garage doors to be picked up. This will be problematic for garage ingress and egress. Please revisit the proposed design to address the issues noted above and provide discussion and/or an exhibit showing the proposed locations of waste containers for pickup.

5. The project will be required to have a right turn-in and right turn-out, with no left turn out of the project. Due to the high speed on Crown Valley, with multiple conflicts with left turns at that intersection, the project will not be allowed to have a left turn out. Please revise the plans accordingly.

Incomplete Items:

1. The geotechnical consultant should review the current plans and provide an update letter. The letter should include, at a minimum:

    a. Discussion of current site conditions;
    b. Current monitoring, including slope inclinometers, piezometers, subdrain video discussion, and results of recent visual observation of surface drainage facilities (i.e., terrace and down drains). These results should include offsite monitoring devices/drains that may impact the proposed development;
    c. Detailed discussion of slope stability, including the existing gravity buttress. Both temporary and gross stability should be discussed, and calculations provided as necessary.
    d. Discussion of the proposed development's geotechnical impacts on the site and adjacent properties.

2. The Preliminary Hydrology Report and Conceptual WQMP have been reviewed by the City's consultant, J.T. Yean. Please see the attached comment letter for necessary revisions/comments.

3. Please submit project plans directly to OCFA for review. Contact OCFA Fire Prevention Analyst Ruben Colmenares with any process related questions at (714) 573-6126.

4. In follow up discussions with staff, the applicant proposed inclusion of a back-of-curb sidewalk along Crown Valley Parkway, similar to development north and south along Crown Valley Parkway. The request is acceptable to the Public Works department. The sidewalk should meet City standards. Please revise the plans accordingly.

5. It is recommended that the applicant provides a trench drain across the driveway entrance to intercept flows leaving the site, and direct them to a treatment system, such as Filterra prior to discharge into the City's storm drain system.

6. It is recommended that 1-2 feet of flat surface be included on the upper side of v-gutters to intercept sediments or that fiber rolls or plants be placed to intercept sediments from entering the v-gutter.

7. Review and ensure that tree selection and location flanking the driveway entrance will not interfere with vehicular ingress and egress.

8. Provide a turning radius template showing adequate width with the proposed hammerhead.

9. On the tentative map, please include flood zone information.

10. On the site plan, please identify proposed above-ground utility structures (if any) and include any screening thereof.

11. On the project plans, please include common lighting details and locations. All exterior light fixtures are to be decorative and architecturally compatible with the project. This is a standard submittal requirement for a project of this type and may not be deferred as a condition of a subsequent approval.

12. As part of the project landscape plans, please include a fuel modification plan prepared in accordance with the standards of the Orange County Fire Authority (OCFA).

13. On the project landscape plan, please demonstrate compliance with the City's Water Efficient Landscape Ordinance.

14. On the building elevations, please include rain gutters.

15. On the side building elevations for each unit type, please delineate the building height envelope as shown in the "Maximum Building Height" exhibit included in Section 9-1-33.4.

16. Please provide before and after visual simulations that illustrate how the overall project would relate to the surrounding roadways, hillside and existing development.

17. Please submit a detailed sign plan including all proposed signage (e.g., location, type, fabrication, materials, height, dimensions, copy, color, etc.). This is a standard submittal requirement for a project of this type and may not be deferred as a condition of a subsequent approval.

18. Please provide a utility plan and letters from the utility providers (gas, water and sewer agencies) indicating that they will serve the proposed project.

19. Please provide a material and color board or samples for all exterior surfaces shown on the project elevations and landscape plans.

20. Please provide any information that has resulted from applicant public outreach efforts to surrounding residents.

As a part of your resubmittal, please provide the following: 1) 10 complete sets of full size revised plans, 2) three reduced size plans and one digital copy (on disc), 3) items requested above, and 4) a written response following the same numeric format of this notice stating how the issues have or will be addressed. After receiving your resubmittal, the application will be reevaluated for completeness and you will be notified of your application's status.

For reference, once the project has been deemed complete, the City will retain the services of an environmental consulting firm to assist in preparing an assessment of the potential environmental impacts resulting from implementation of the proposed project pursuant to the California Environmental Quality Act (CEQA) and its Guidelines. This will also include preparation of a series of technical studies. The applicant shall be responsible for any and all costs incurred as part of the project environmental review.

Should you have any questions, please contact me by phone at (949) 362-4357 or by email at jorduna@cityoflagunaniguel.org.

Sincerely,

Jonathan Orduna
Senior Planner
Direct Phone: 949-362-4357
E-mail: jorduna@cityoflagunaniguel.org

Jonathan Orduna
Senior Planner
City of Laguna Niguel
30111 Crown Valley Parkway
Laguna Niguel, CA 92677

J.T. Yean, Ph.D., P.E., D.WRE
Hydrologist
22341 Blueberry Lane
Lake Forest, CA 92630
(949) 331-5889

Preliminary Hydrology Study
REVIEW SHEET

Permits Numbers: SP16-04

Report Date	***Date Received***	***Date Reviewed***	***Plan Check Status***
09/23/2016	10/05/2016	10/10/2016	1st Plan Check

By the Consulting Firm: Michael Baker Intl. (949) 855-5711
Project Title: **Tentative Tract 17721 (Crowne Cove Development)**
Project Address: TT 17721 Corner of Crown Valley Pkwy and Playa Blanca Dr., Laguna Niguel, CA
APN: 656-231-02
Engineers: Fabio Escobar Jr. & Rick L. Howe

Developer/Owner: Laguna Niguel Properties, Inc.
Address: 27422 Portola Parkway, Suite 300, Foothill Ranch, CA 92610
Telephone: (714) 272-9278

Prior to approval of the preliminary hydrology report attend to the items below:

1. Cover Page: Add "Site Development Permit: SP16-04", project site location, and Assessor Parcel Numbers on the cover page with the engineer's stamp and signature.

2. Table of Contents: All maps should be in 24x36 sheet size.

3. Existing Hydrology: Provide exhibits and narrative describing the drainage conditions specifically:
   a. The off-site runoff to be collected into the existing 42" and 36" RCP.
   b. The existing drainage areas upstream 42" storm drain as shown in the Site Plan.
   c. The outlet of the 48" storm drain at El Niguel Country Club and Salt Creek.
   d. Reference of the existing storm drain hydrology/hydraulic/as-built documentation.

4. Proposed Hydrology: Provide exhibits and narrative describing the drainage conditions specifically:
   a. Proposed inlet structures to receive off-site runoff to enter the development site.
   b. Desilting basins to convey the off-site clear flow through the development site.
   c. A summary table listing the proposed drainage facilities.
   d. Potential impacts on the existing drainage facilities,
   e. The scour and erosion impacts at the outlet in El Niguel Country Club and Salt Creek.

5. Add an Existing Hydraulic Section:
   a. Provide a drainage exhibit to show the layout of the existing backbone drainage facilities in the vicinity of the project site including pipe lines, catch basins, inlets, outlets and energy dissipators.
   b. Provide a summary table listing all existing backbone facilities including pipelines, catch basins, detention basins, and other drainage structures, including hydraulic data (S%, Q, V, D, etc.)
   c. Discuss the hydraulic capacity and ownership of the existing 42", 36" and 48" RCP.

6. Add a Proposed Hydraulic Section:
   a. Provide a drainage exhibit to show the layout of backbone drainage facilities within the project site, such as main lines, catch basins, split flow structures, detention basins, inlets, outlets and energy dissipators.
   b. Provide a summary table listing all proposed backbone facilities including pipelines, catch basins, detention basins, and other drainage structures, including hydraulic data (S%, Q, V, D, etc.) for the proposed drainage facilities.
   c. Provide hydraulic analysis at the upstream inlets (water ponding elevations) and the downstream outlets (potential scour velocities).

7. Conclusion:
   a. Provide a summary table showing the comparison between the existing and proposed peak discharges, volumes, and velocities at the point of discharge (POD) before discharging out from the development site.
   b. Identify all potential hydrologic impacts (runoff, scour, erosion, flood, debris, etc.) on the downstream drainage facilities and natural watercourse.
   c. Provide mitigation measures.

8. Hydrology Maps (Existing and Proposed Map in 24x36 Sheet Size):
   a. Show on-site and off-site runoff flow directions (with arrow heads).
   b. Identify the sizes of drainage facilities including main lines, catch basins, split flow structures, detention basins, inlets, outlets and energy dissipators.
   c. Provide a summary table listing all proposed backbone facilities including pipelines, catch basins, detention basins, and other drainage structures, including hydraulic data (S%, Q, V, D, etc.) for the proposed drainage facilities.
   d. Provide a hydrology map to show the entire drainage areas related with the upstream/downstream drainage facilities and Salt Creek.

Provide a response letter to the above issues with re-submittal.

Report Reviewed by: JT Yean ______________________ **Date: 10-10-2016**
J.T. Yean, Civil Engineer

Jonathan Orduna
Senior Planner
City of Laguna Niguel
30111 Crown Valley Parkway
Laguna Niguel, CA 92677

J.T. Yean, Ph.D., P.E., D.WRE
Hydrologist
22341 Blueberry Lane
Lake Forest, CA 92630
(949) 331-5889

Conceptual WQMP
REVIEW SHEET

Permits Numbers: SP16-04

Report Date	***Date Received***	***Date Reviewed***	***Plan Check Status***
09/22/2016	10/05/2016	10/14/2016	1st Plan Check

By the Consulting Firm: Michael Baker Intl. (949) 855-5711
Project Title: **Tentative Tract 17721 (Crowne Cove Development)**
Project Address: TT 17721 Corner of Crown Valley Pkwy and Playa Blanca Dr., Laguna Niguel, CA
APN: 656-231-02
Engineers: Fabio Escobar Jr. & Rick L. Howe

Developer/Owner: Laguna Niguel Properties, Inc.
Address: 27422 Portola Parkway, Suite 300, Foothill Ranch, CA 92610
Telephone: (714) 272-9278

Prior to approval of the Conceptual WQMP attend to the items below:

1. Cover Page: Add "Site Development Permit: SP 16-04" and engineer's stamp and signature.

2. Owner's Certification: Provide complete information with the owner's and engineer's signatures.

3. Table of Contents: Remove Appendices C, D and E. They are not required in C-WQMP.

4. Section I: Fill in SP permit number.

5. Section III – Off-Site Drainage: "Off-Site" is considered as the drainage areas outside the development boundary (or disturbed areas), which includes open space, slopes, un-disturbed areas or adjacent properties that drains runoff through the development site. Describe how the proposed facilities are to mitigate the drainage and water quality impacts due to the development.

6. Section III – Off-Site Drainage: If "Off-Site" runoff mixed with "On-Site" runoff, BMPs shall be sized to the treat the mixed flow.

7. Section IV.1 – LID: Flow-based BMP (catch basin insert) cannot be granted for LID/DCV as an alternative compliance in SOC. Use Bio-retention or Bio-filtration for DCV following TGD Appendix XI for design criteria.

8. Section IV.3.2 – Infiltration BMPs: To claim the infeasibility of infiltration, provide Infiltration BMP Feasibility Worksheets (Table 2.7 - TGD) with related soil data (excerpt from the soil report. Do not include the entire soil report).

9. Section IV.3.4 – Biofiltration BMPs: Provide site specific engineering sketches of volume based Modular Wetland System (MWS) including inlet, outlet, overflow and the connection to POD (Point of Discharge). Show sizing and dimensions of BMPs.

10. Section IV.3.5 – Hydromodification BMPs: Provide site specific engineering sketches of the underground detention basin (Contech CMP System) including sections, inlets, outlets, overflow, and connections to POD (Point of Discharge). Show sizing and dimensions of BMPs.

11. WQMP Exhibits: Should be shown in 24"x36" or larger sheet size, and:
    a. Provide location map, vicinity map, site address, and permit number.
    b. Show flow directions, drainage facilities and BMP locations.
    c. Show delineation of drainage management area (DMA) to each BMP with proper ID's.
    d. Provide the engineering sketches and section details for proposed BMPs.
    e. Provide a summary table listing DMA, LID/DCV, and BMP sizing details (i.e. footprint, depth, and capacity)

12. Attachment A: Remove Education Materials (Not required in C-WQMP).

13. Attachment A: BMP Factsheets: Select the proposed model with sizing details in BMP factsheets. Provide sizing calculation sheets. All sizing data shall be able to be verified in the WQMP Exhibits.

14. Attachment B: SOHM report: Provide engineering sketches for all input data including inlet, outlet, overflow, and Stage - Storage - Discharge (SSD) curves.

15. Do not include any irrelevant materials and educational materials in the report. The C-WQMP shall be brief, concise and to-the- point.

16. Both WQMP Report and Exhibit need engineer's stamp and signature

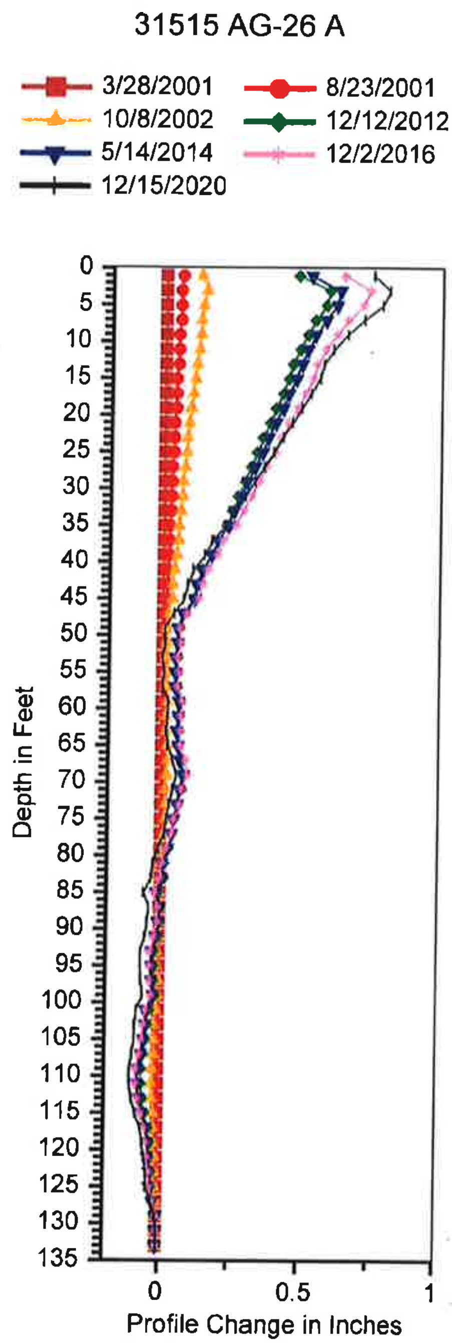
Provide a response letter to the above issues with re-submittal.

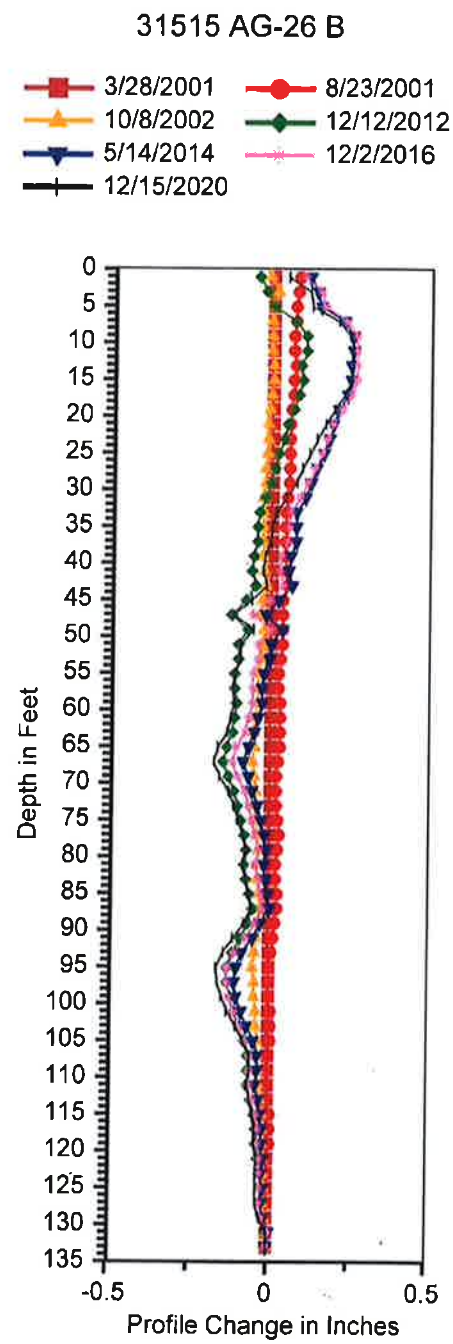
Report Reviewed by: JT Yean **Date: 10-14-2016**
J.T. Yean, Civil Engineer

File No. 33348-08
January 5, 2021

APPENDIX C

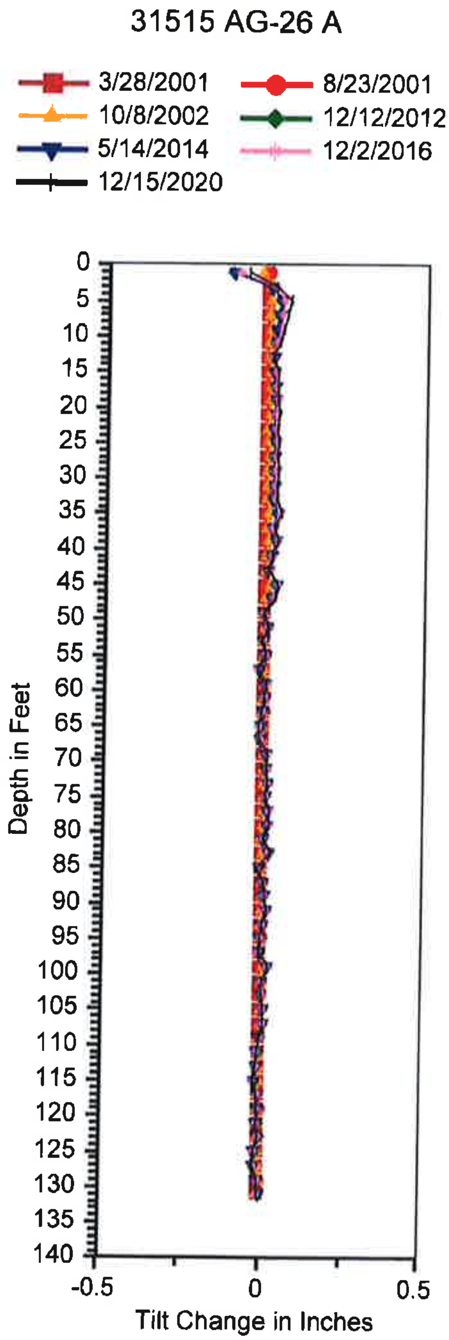
SLOPE INCLINOMETER PLOTS

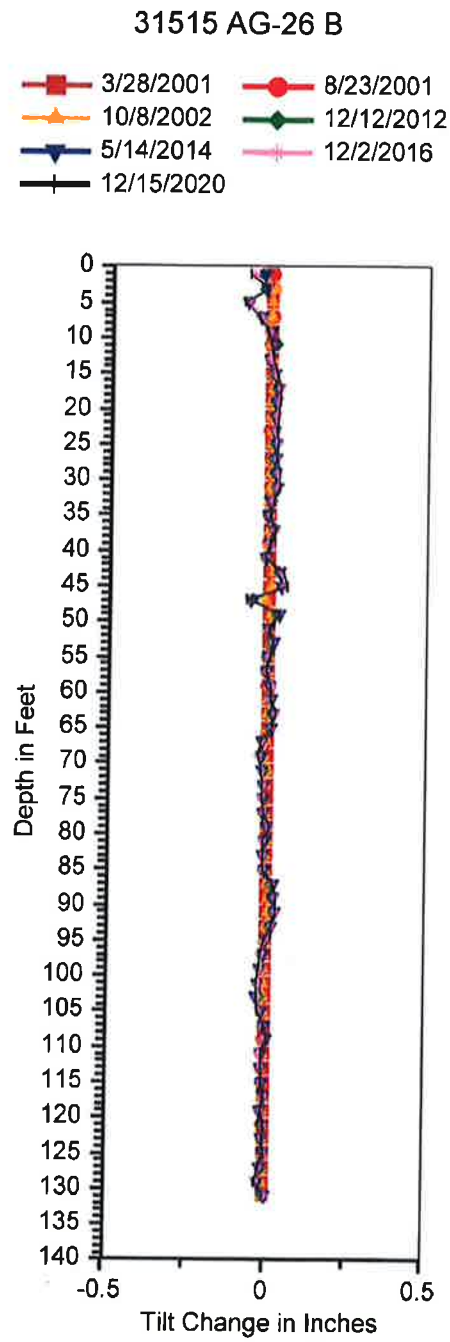




Cumulative Displacement (in)
Niguel Summit FN: 31515
American Geotechnical
Inclinometer elev. 440'
First reading depth: 134'

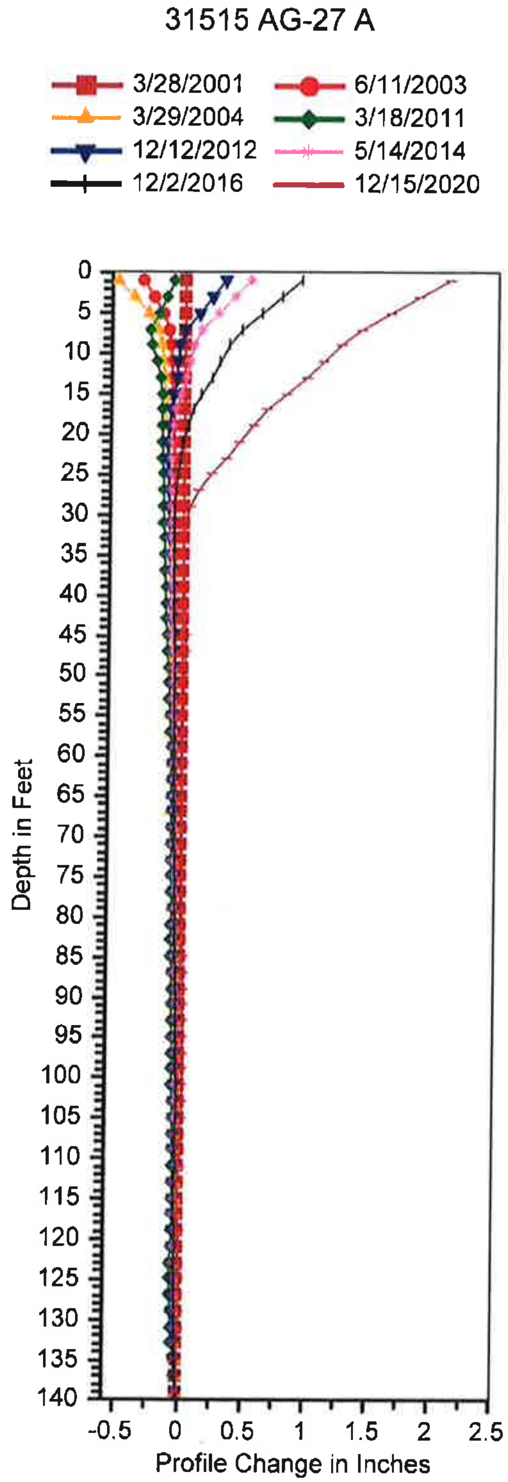
Approx bedrock elev. 375'
(both slip surfaces removed during repair)
Re-zeroed probe on 3/28/01
Note: bottom 4' lost to sediment filling prior to 3/28/01

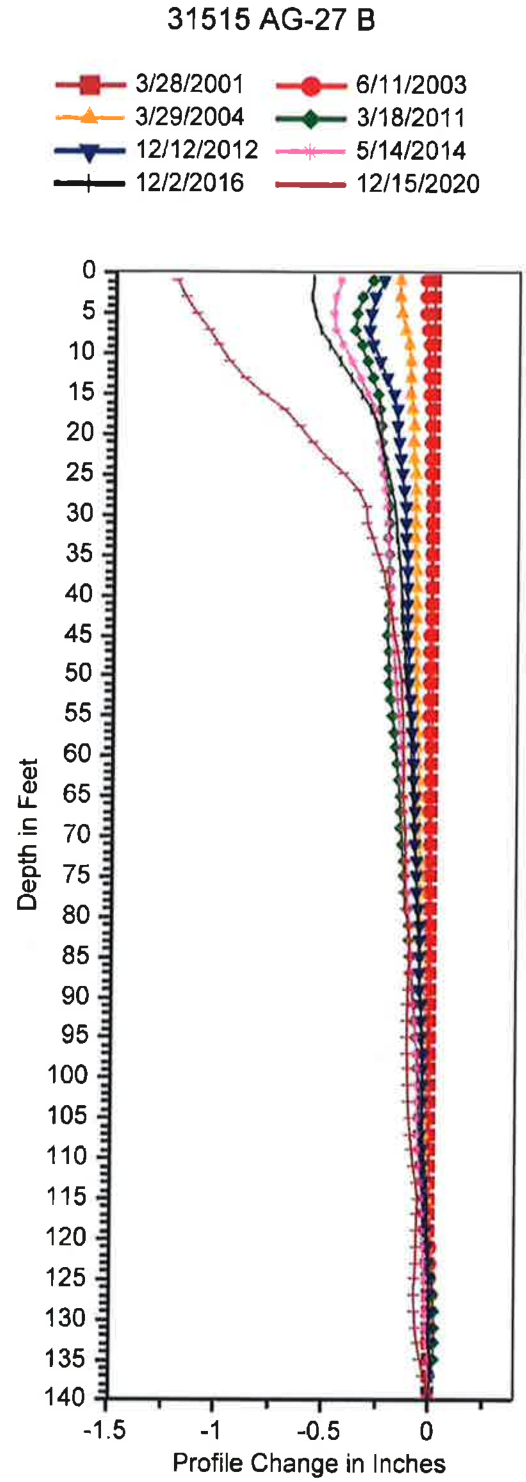




Incremental Displacement (in)
Niguel Summit FN: 31515
American Geotechnical
Inclinometer elev. 440'
First reading depth: 134'

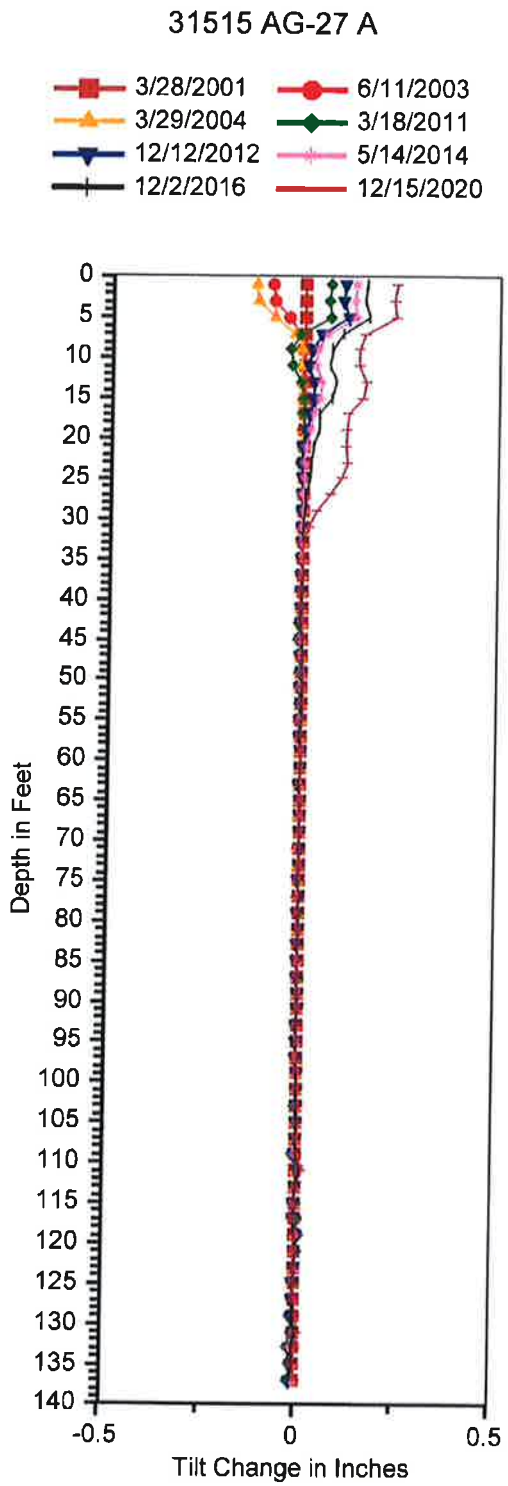
Approx. bedrock elev. 375'
(both slip surfaces removed during repair)
Re-zeroed probe on 3/28/01
Note: bottom 4' lost to sediment filling prior to 3/28/01

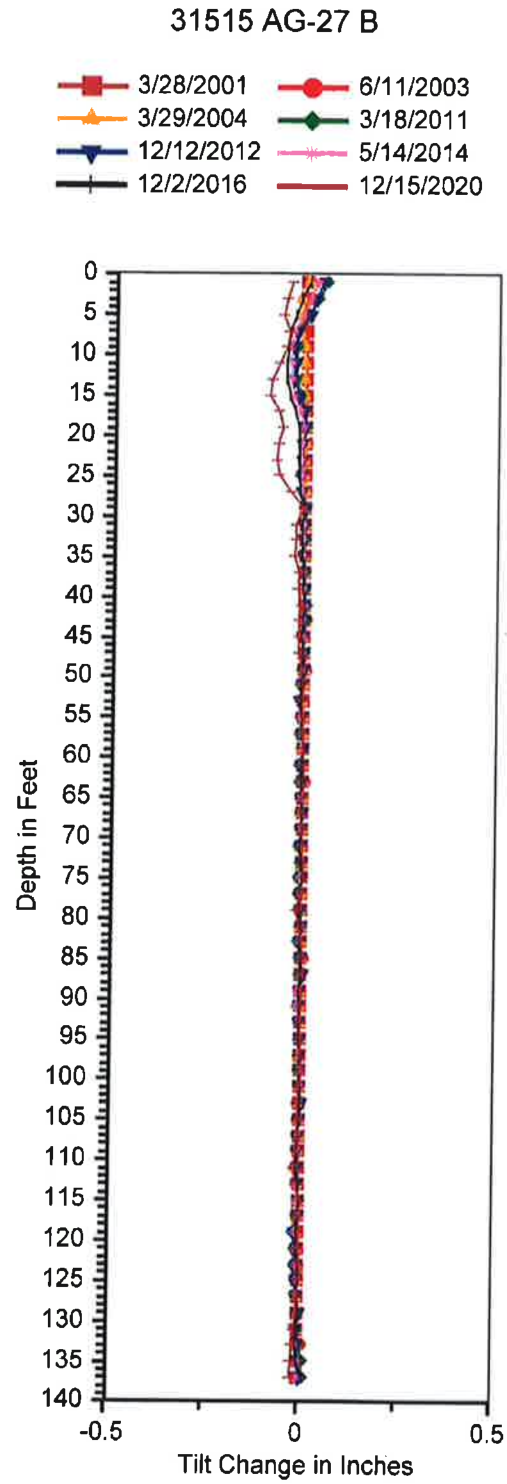




Cumulative Displacement (in)
Niguel Summit FN: 31515
American Geotechnical
Inclinometer elev. 380'
First reading depth: 138'

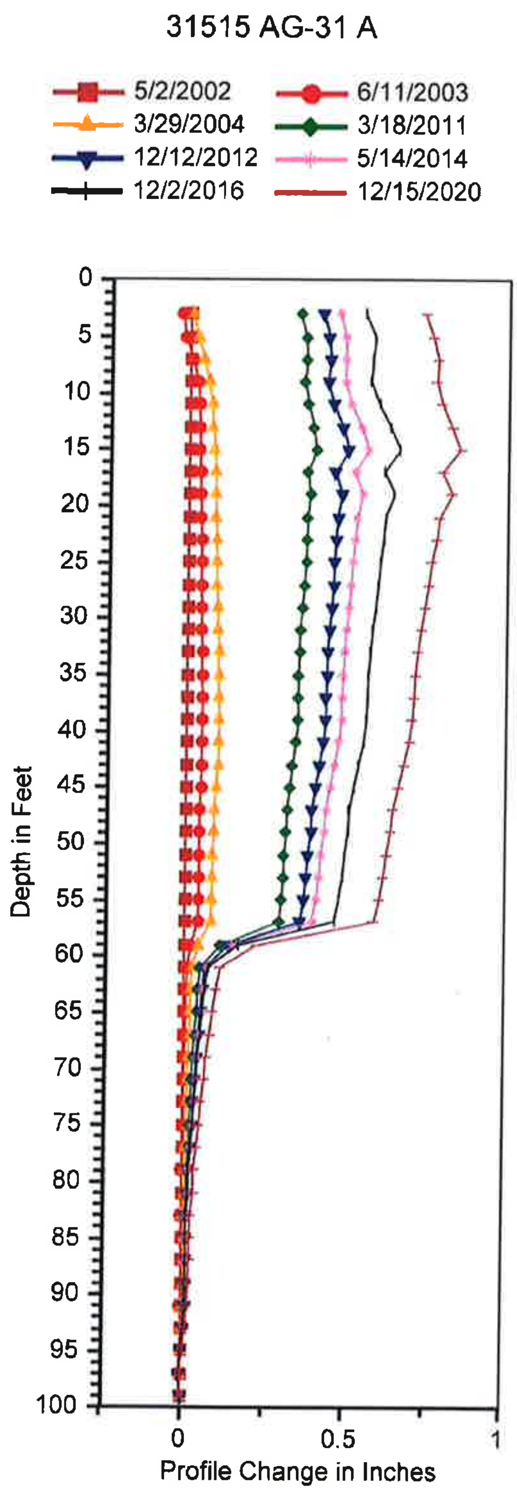
Re-zeroed probe on 3/28/01

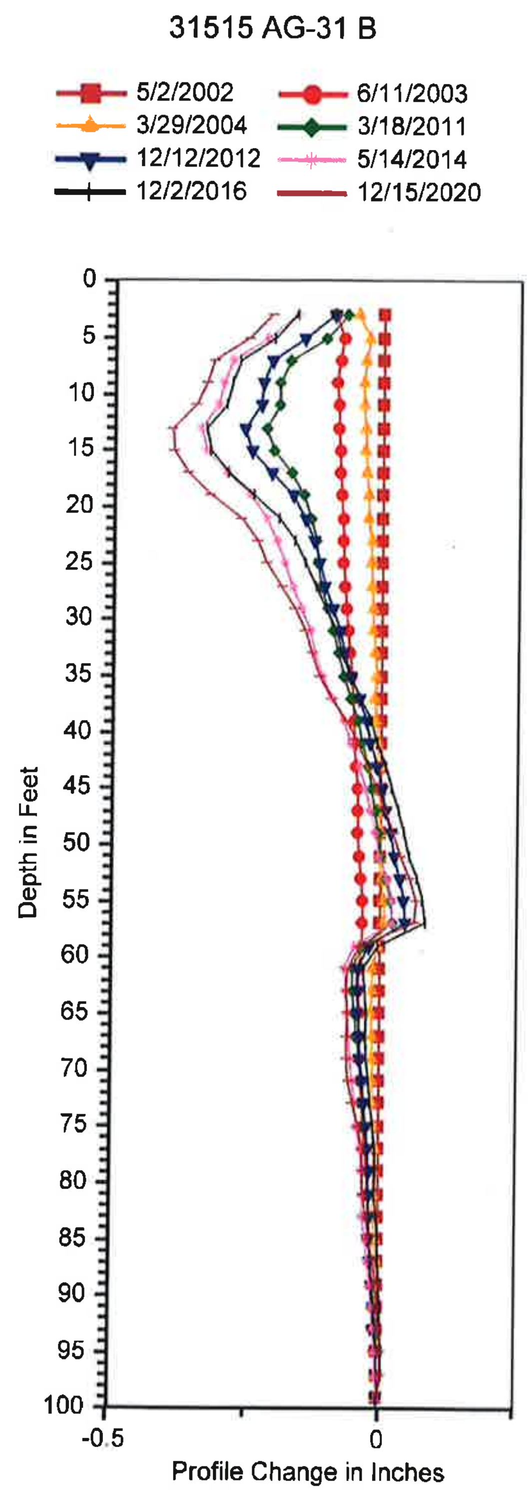




Incremental Displacement (in)
Niguel Summit FN: 31515
American Geotechnical
Inclinometer elev. 380'
First reading depth: 138'

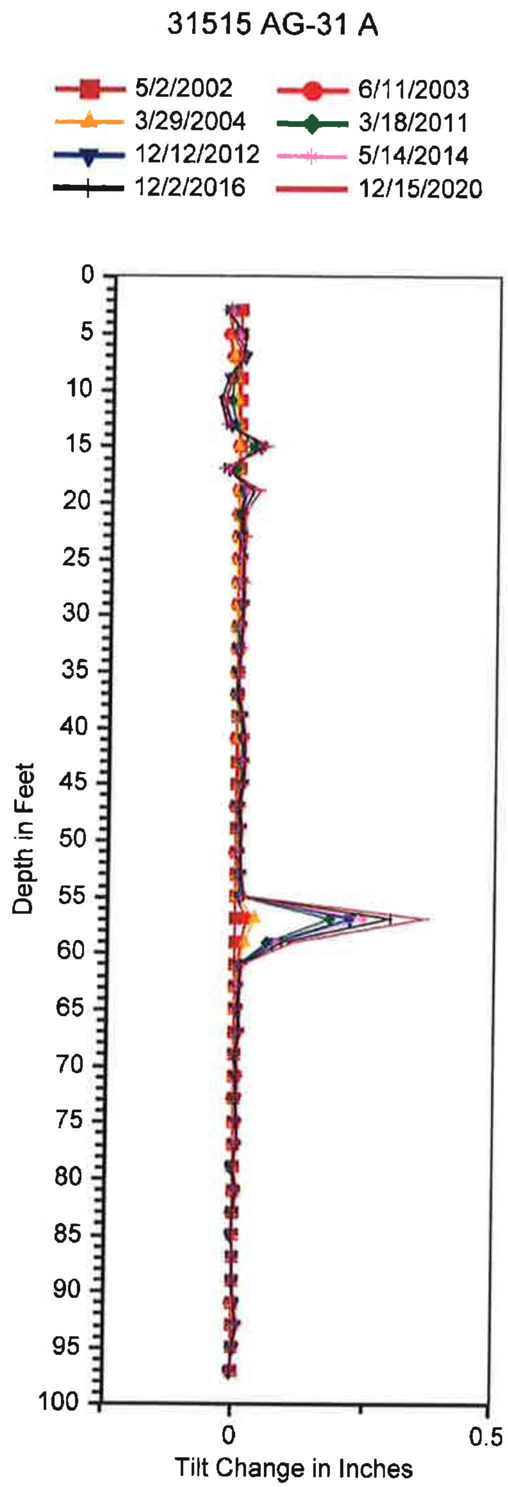
Re-zeroed probe on 3/28/01

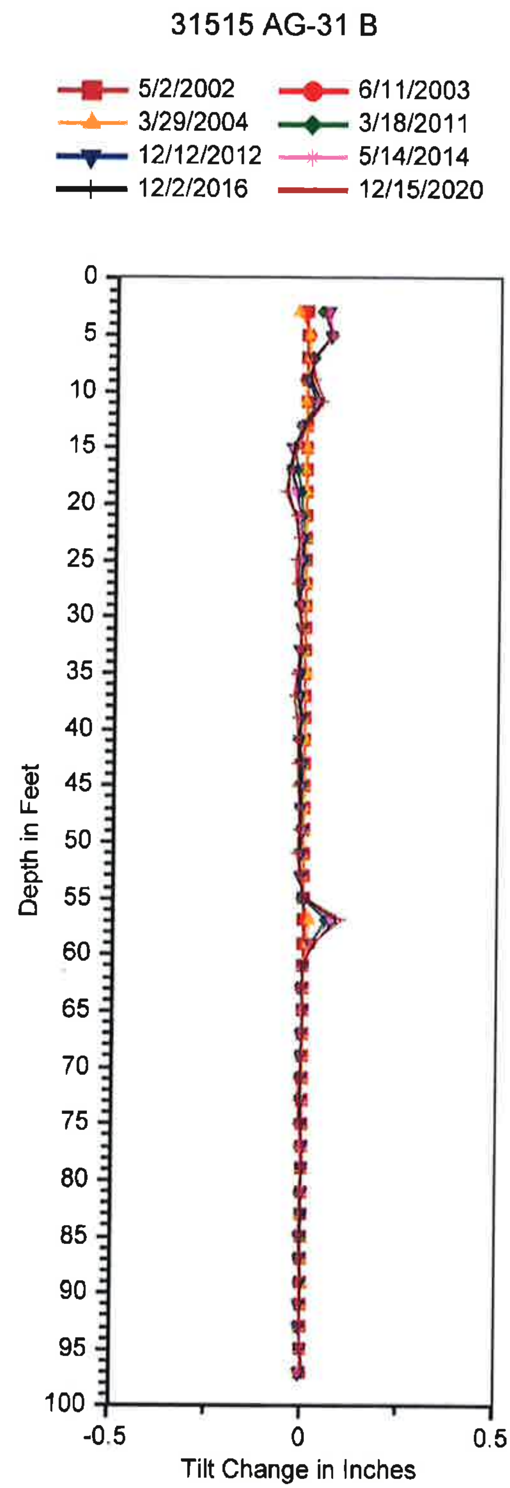




Cumulative Displacement (in)
Niquel Summit FN: 31515
American Geotechnical

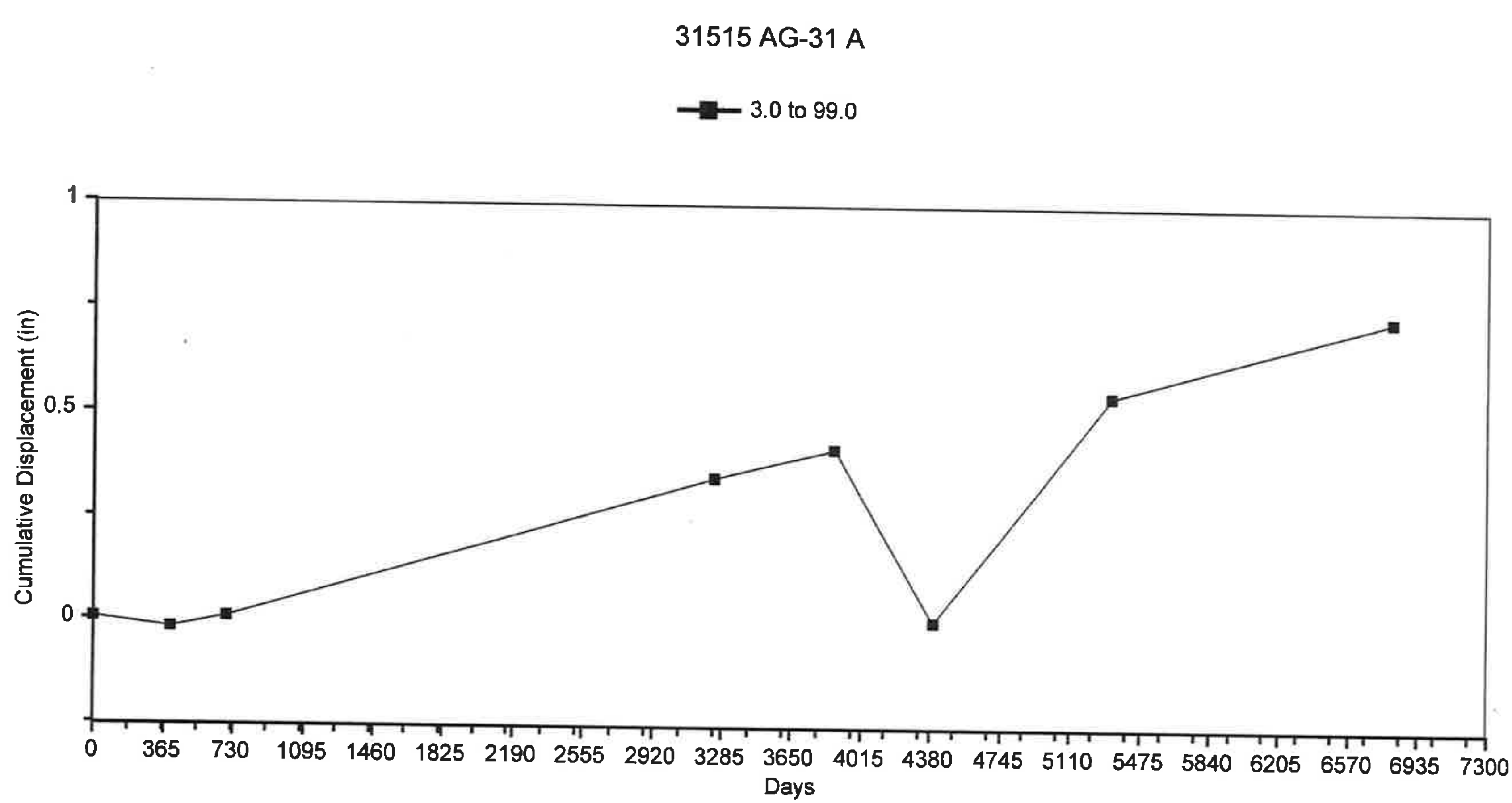
End of fire road





Incremental Displacement (in)
Niguel Summit FN: 31515
American Geotechnical

End of fire road



Time Plot of Cumulative Displacement (in) since 5/2/02
Niguel Summit
AGI-31
American Geotechnical

APPENDIX D

PIEZOMETER READINGS

33348-08 The Cove at El Niguel
Summary of Piezometer Readings

Piezo. No.	Casing/ Grade Elev. (ft. amsl)	Piezo. Reading Date	Water Level Elev. (ft. amsl)	Remarks
AGP-9	420.0	06/06/02	385.2	
AGP-9	420.0	07/02/02	387.0	
AGP-9	420.0	08/23/02	387.3	
AGP-9	420.0	10/09/02	387.3	
AGP-9	420.0	11/22/02	387.4	
AGP-9	420.0	12/23/02	387.5	
AGP-9	420.0	06/11/03	388.2	
AGP-9	420.0	04/02/04	388.3	
AGP-9	420.0	12/23/04	387.6	
AGP-9	420.0	01/22/13	385.6	
AGP-9	420.0	12/02/16	382.8	
AGP-9	420.0	12/15/20	385.4	
AGP-10	420.5	06/06/02	386.5	
AGP-10	420.5	07/02/02	386.4	
AGP-10	420.5	08/23/02	386.6	
AGP-10	420.5	10/09/02	386.6	
AGP-10	420.5	11/22/02	386.5	
AGP-10	420.5	12/23/02	386.6	
AGP-10	420.5	06/11/03	386.8	
AGP-10	420.5	04/02/04	386.2	
AGP-10	420.5	12/23/04	385.0	
AGP-10	420.5	01/22/13	377.6	
AGP-10	420.5	12/02/16	371.4	
AGP-10	420.5	12/15/20	382.7	
AGPZ-1	376.4	05/16/13	345.6	
AGPZ-1	376.4	05/17/13	346.4	
AGPZ-1	376.4	05/22/13	344.4	
AGPZ-1	376.4	12/05/16	342.0	
AGPZ-1	376.4	12/16/20	348.2	
AGPZ-2	374.9	05/17/13	339.6	
AGPZ-2	374.9	05/22/13	339.5	
AGPZ-2	374.9	12/05/16	337.4	
AGPZ-2	374.9	12/15/20	342.6	
AGPZ-3	442.5	05/17/13	Dry	
AGPZ-3	442.5	05/22/13	Dry	
AGPZ-3	442.5	12/05/16	N/A	* Inaccessable
AGPZ-3	442.5	12/15/20	N/A	* Damaged and clogged
AGPZ-4	408.0	05/17/13	374.2	
AGPZ-4	408.0	05/22/13	374.6	
AGPZ-4	408.0	12/05/16	373.0	
AGPZ-4	408.0	12/15/20	373.9	
AGPZ-5	377.0	05/17/13	339.3	
AGPZ-5	377.0	05/22/13	339.1	
AGPZ-5	377.0	12/05/16	341.0	
AGPZ-5	377.0	12/15/20	341.7	

APPENDIX E

EXISTING STORM DRAIN INSPECTION REPORT

HOUSTON HARRIS PCS, Inc.

Houston & Harris PCS, Inc.
21831 Barton Rd.
Grand Terrace, CA 92313
Tel: 909-422-8990
Fax: 909-422-0841
E-mail: info@houstonandharris.com

Inspection Report

Date	P/O. No.	Weather	Surveyor's Name	Pipe ID:	Section No.
12/28/2020		**Light Rain**	**TREVOR**	**LINE A (UPSTREAM FRO**	**1**
Certificate No.	Survey Customer	System Owner	Date Cleaned	Pre-Cleaning	Sewer Category
U06180703002245				**No Pre-Cleaning**	

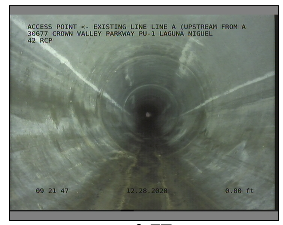
Street	**30677 CROWN VALLEY PARKWAY**	Use of Sewer	**Stormwater**	Upstream MH	**EXISTING LINE**
City	**LAGUNA NIGUEL**	Drainage Area		Dowstream MH	**ACCESS POINT**
Loc. details		Flow Control		Dir. of Survey	**Upstream**
Location Code	**PU-1**	Length surveyed	**181.87 ft**	Section Length	**181.87 ft**

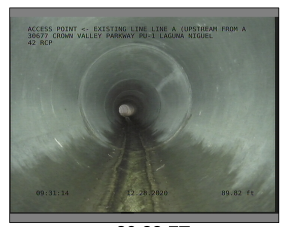
Purpose of Survey	**Capital Improvement Program Assessment**	Joint Length	
Year Laid		Dia./Height	**42 inch**
Year Rehabilitated		Material	**RCP**
Tape / Media No.		Lining Method	

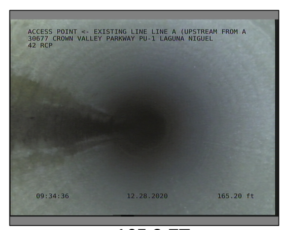
Add. Information :

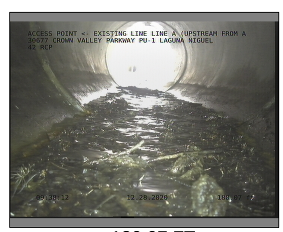
ACCESS POINT

1:462 Position	Code	Observation	Grade
0.00	AMH	Downstream Manhole, Survey Begins / ACCESS POINT 1	
0.00	MWL	Water Level, 5 %of cross sectional area	
89.33	MGP	General Photograph	
165.20	TF	Tap Factory Made, at 10 o'clock, -, within 8 inches of joint: YES, 18"	
180.07	DSZ	Deposits Settled Other, 10 %of cross sectional area, from 05 to 07 o'clock, , within 8 inches of joint: YES / MISC DEBRI	M 2
181.87	MSA	Survey Abandoned / DEBRI BLOCKING CAMERA	


0 FT


89.33 FT


165.2 FT


180.07 FT

QSR	QMR	SPR	MPR	OPR	SPRI	MPRI	OPRI
0000	**2100**	**0**	**2**	**2**	**0**	**2**	**2**

Houston & Harris PCS, Inc.
21831 Barton Rd.
Grand Terrace, CA 92313
Tel: 909-422-8990
Fax: 909-422-0841
E-mail: info@houstonandharris.com

Inspection photos

City :	Street :	Date :	Pipe ID :	Section No :
LAGUNA NIGUEL	**30677 CROWN VALLEY PARKWAY**	**12/28/2020**	**LINE A (UPSTREAM FROM**	**1**

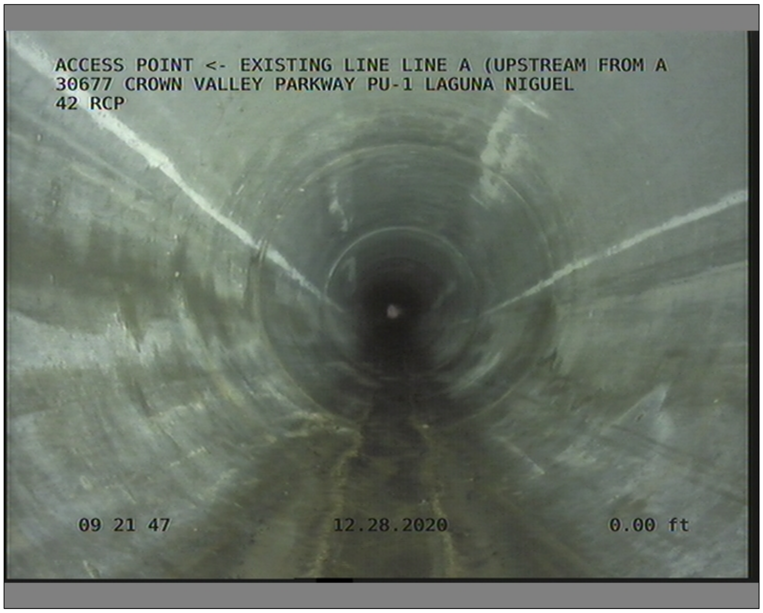


Photo: EXISTING LINE-ACCESS POINT-U-092039.JPG
0FT, Water Level, 5 %of cross sectional area

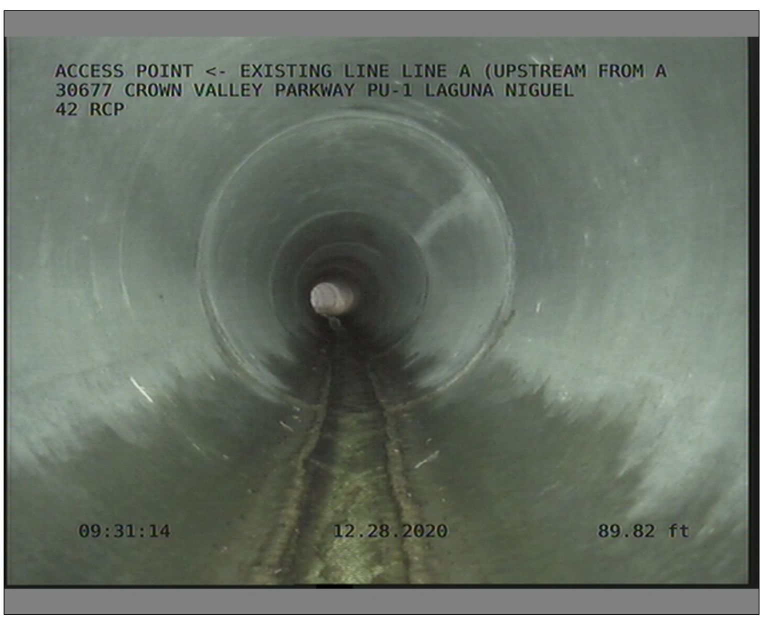


Photo: EXISTING LINE-ACCESS POINT-U-093006.JPG
89.33FT, General Photograph

Houston & Harris PCS, Inc.
21831 Barton Rd.
Grand Terrace, CA 92313
Tel: 909-422-8990
Fax: 909-422-0841
E-mail: info@houstonandharris.com

Inspection photos

City :	Street :	Date :	Pipe ID :	Section No :
LAGUNA NIGUEL	**30677 CROWN VALLEY PARKWAY**	**12/28/2020**	**LINE A (UPSTREAM FROM**	**1**

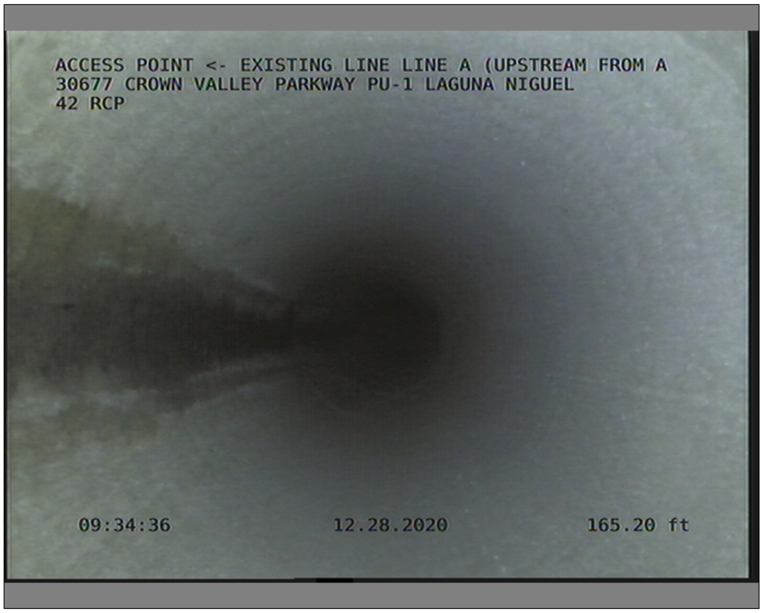


Photo: EXISTING LINE-ACCESS POINT-U-093328.JPG
165.2FT, Tap Factory Made, at 10 o'clock, -, within 8 inches of joint: YES, 18"

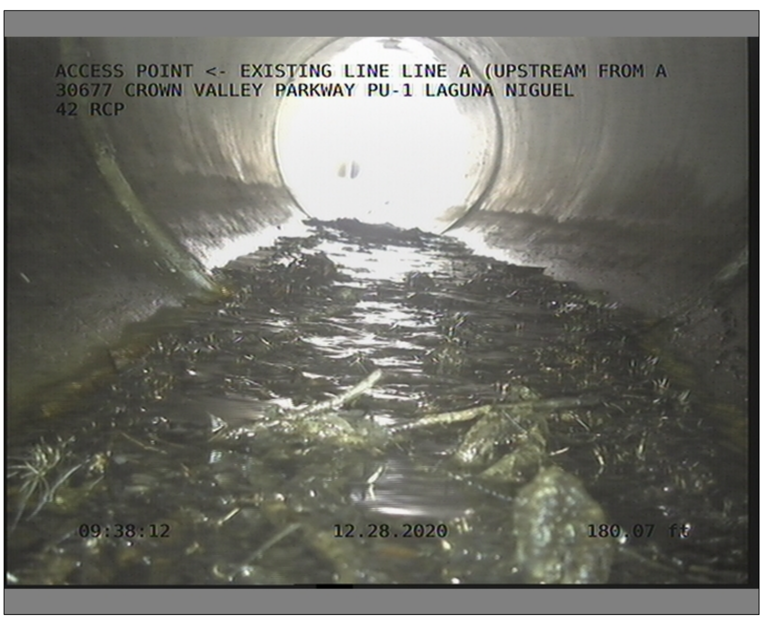


Photo: EXISTING LINE-ACCESS POINT-U-093657.JPG
180.07FT, Deposits Settled Other, 10 %of cross sectional area, from 05 to 07 o'clock,
, within 8 inches of joint: YES

City : LAGUNA NIGUEL

Houston & Harris PCS, Inc.
21831 Barton Rd.
Grand Terrace, CA 92313
Tel: 909-422-8990
Fax: 909-422-0841
E-mail: info@houstonandharris.com

Inspection Report

Date	P/O. No.	Weather	Surveyor's Name	Pipe ID:	Section No.
12/28/2020		**Light Rain**	**TREVOR**	**LINE A (DOWNSTREAM F**	**2**
Certificate No.	Survey Customer	System Owner	Date Cleaned	Pre-Cleaning	Sewer Category
U06180703002245				**No Pre-Cleaning**	

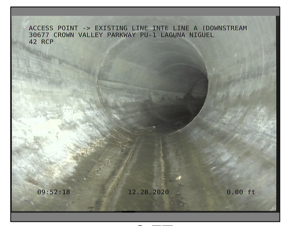
Street	**30677 CROWN VALLEY PARKWAY**	Use of Sewer	**Stormwater**	Upstream MH	**ACCESS POINT**
City	**LAGUNA NIGUEL**	Drainage Area		Dowstream MH	**EXISTING LINE INTERSECT**
Loc. details		Flow Control		Dir. of Survey	**Downstream**
Location Code	**PU-1**	Length surveyed	**34.61 ft**	Section Length	**34.61 ft**

Purpose of Survey	**Capital Improvement Program Assessment**	Joint Length	
Year Laid		Dia./Height	**42 inch**
Year Rehabilitated		Material	**RCP**
Tape / Media No.		Lining Method	

Add. Information :

ACCESS POINT

1:98	Position		Code	Observation	Grade
	0.00		AMH	Upstream Manhole, Survey Begins / ACCESS POINT 1	
	0.00		MWL	Water Level, 5 %of cross sectional area	
	10.08	S1	LR	Alignment Right, 5 %, Start	M 1
	19.74		MWL	Water Level, 15 %of cross sectional area	
	30.46		DSZ	Deposits Settled Other, 15 %of cross sectional area, from 05 to 07 o'clock, , within 8 inches of joint: YES / DEBRI	M 3
	34.28	F1	LR	Alignment Right, 5 %, Finish	M 1
	34.61		MSA	Survey Abandoned / DSZ BLOCKING CAMERA	



0 FT

30.46 FT

QSR	QMR	SPR	MPR	OPR	SPRI	MPRI	OPRI
0000	3115	0	8	8	0	1.33	1.33

Houston & Harris PCS, Inc.
21831 Barton Rd.
Grand Terrace, CA 92313
Tel: 909-422-8990
Fax: 909-422-0841
E-mail: info@houstonandharris.com

Inspection photos

City :	Street :	Date :	Pipe ID :	Section No :
LAGUNA NIGUEL	**30677 CROWN VALLEY PARKWAY**	**12/28/2020**	**LINE A (DOWNSTREAM FR**	**2**

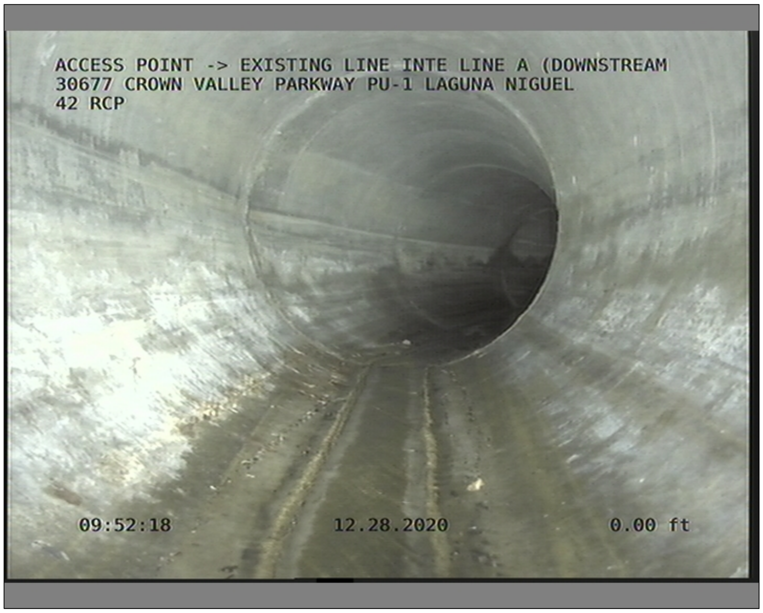


Photo: ACCESS POINT-EXISTING LINE INTERSECT-D-095110.JPG
0FT, Water Level, 5 %of cross sectional area

Photo: ACCESS POINT-EXISTING LINE INTERSECT-D-095336.JPG
30.46FT, Deposits Settled Other, 15 %of cross sectional area, from 05 to 07 o'clock,
, within 8 inches of joint: YES

APPENDIX F

PROBABILISTIC SEISMIC HAZARD ANALYSIS

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u...

Latitude

Decimal degrees

33.517695

Longitude

Decimal degrees, negative values for western longitudes

-117.720455

Site Class

360 m/s (C/D boundary)

Spectral Period

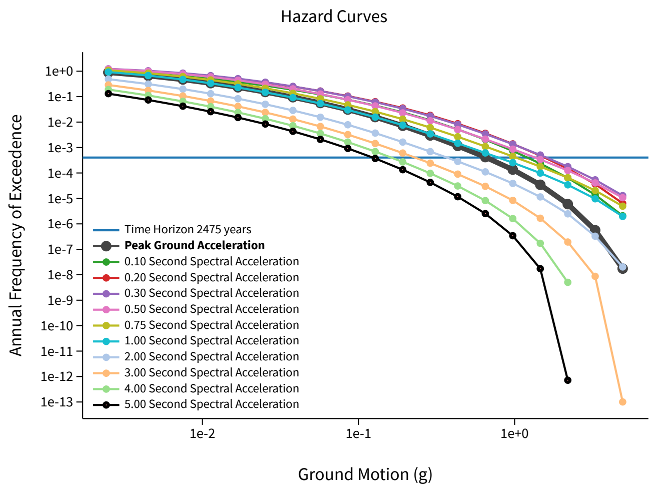
Peak Ground Acceleration

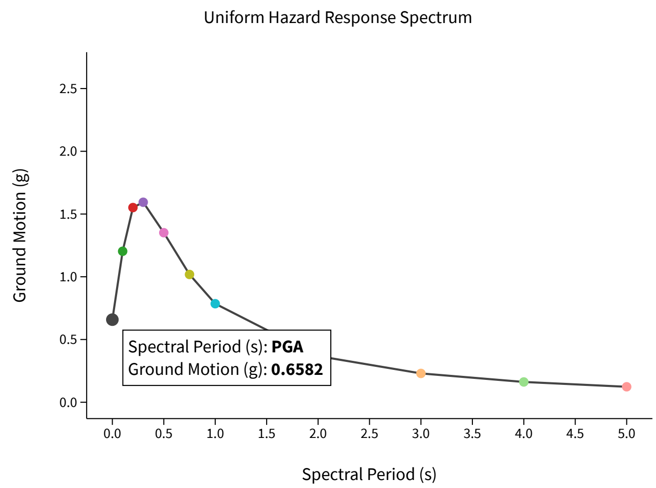
Time Horizon

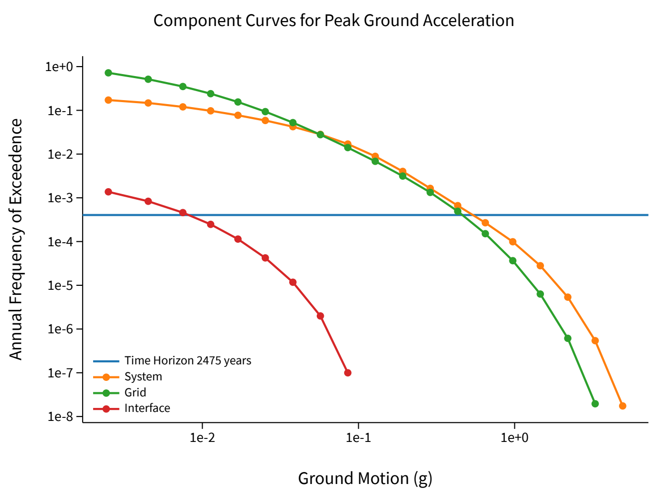
Return period in years

2475

Hazard Curve





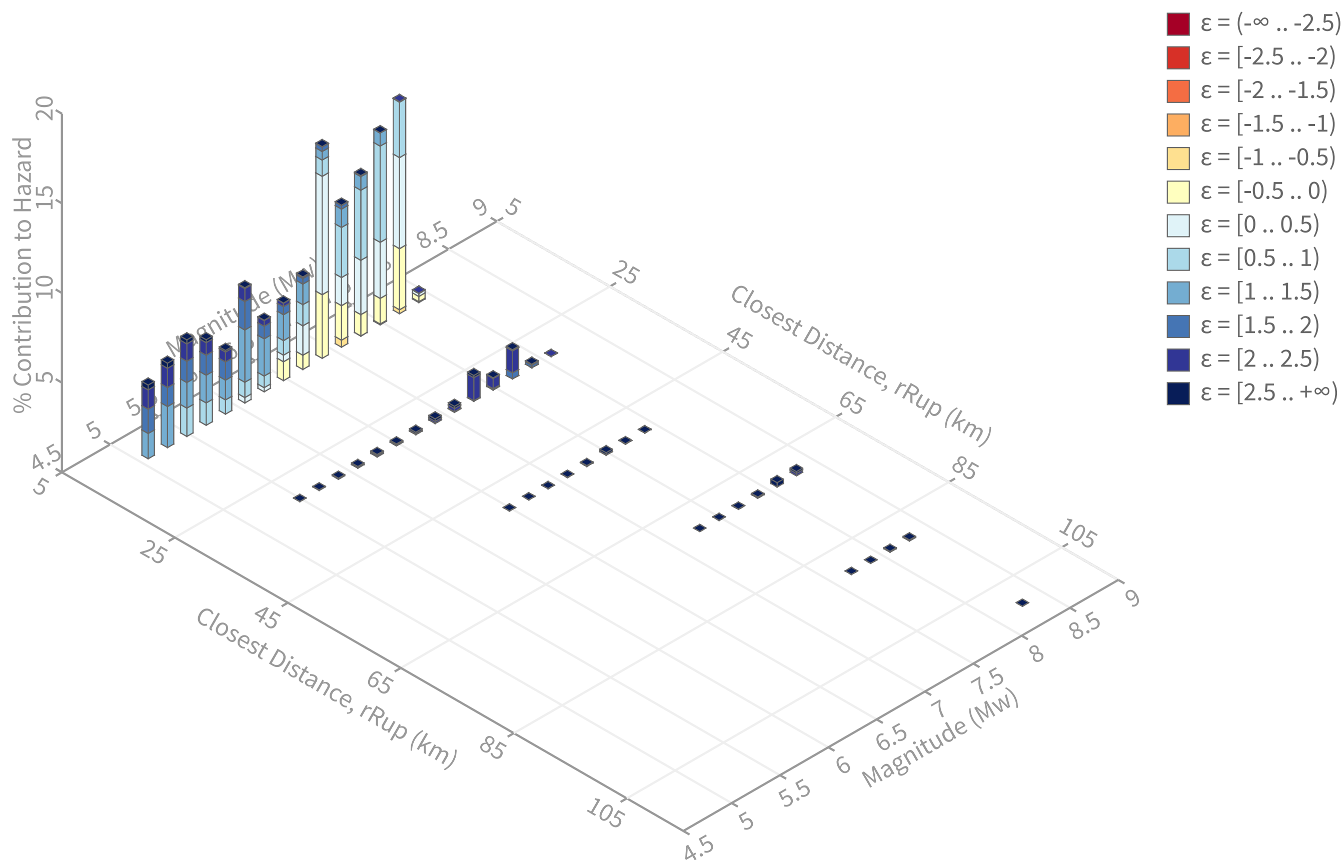


View Raw Data

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.65820948 g

Recovered targets

Return period: 2754.9755 yrs
Exceedance rate: 0.00036297963 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.07 %

Mean (over all sources)

m: 6.73
r: 8.61 km
ε_0: 0.87 σ

Mode (largest m-r bin)

m: 7.69
r: 5.29 km
ε_0: 0.19 σ
Contribution: 11.82 %

Mode (largest m-r-ε_0 bin)

m: 6.89
r: 5.66 km
ε_0: 0.17 σ
Contribution: 6.6 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

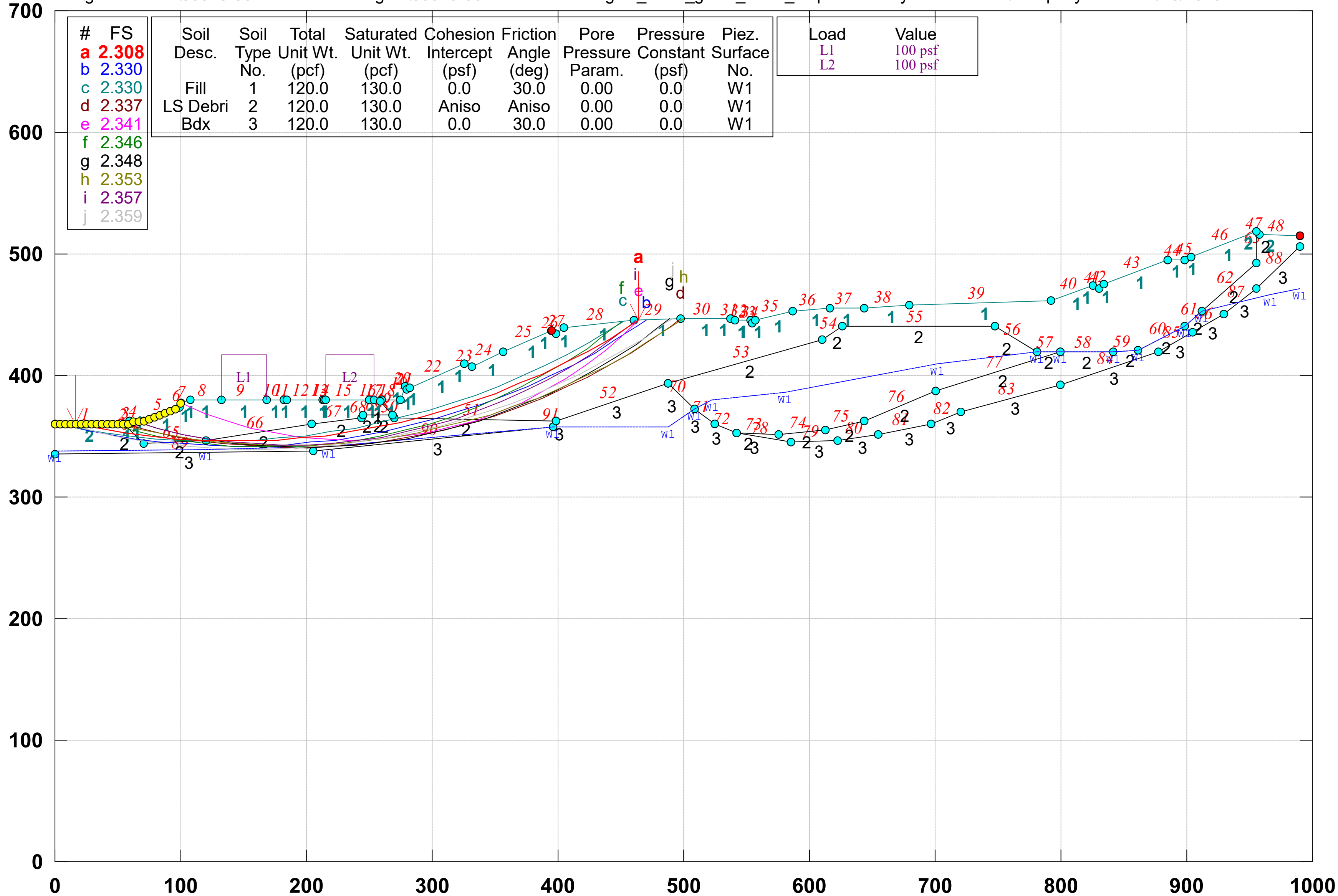
Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM32	System							33.58
San Joaquin Hills [2]		5.69	6.95	0.09	117.685°W	33.577°N	26.56	13.10
Newport-Inglewood (Offshore) [3]		6.26	7.25	0.78	117.771°W	33.482°N	229.60	10.19
Oceanside alt2 [12]		4.68	7.63	-0.05	117.802°W	33.515°N	267.65	5.09
UC33brAvg_FM31	System							32.20
Oceanside alt1 [6]		4.65	7.31	0.01	117.802°W	33.514°N	267.22	14.74
Newport-Inglewood (Offshore) [3]		6.26	7.24	0.78	117.771°W	33.482°N	229.60	10.60
San Joaquin Hills [2]		5.69	7.24	0.11	117.685°W	33.577°N	26.56	1.35
UC33brAvg_FM31 (opt)	Grid							17.12
PointSourceFinite: -117.720, 33.540		5.75	5.56	1.16	117.720°W	33.540°N	0.00	3.66
PointSourceFinite: -117.720, 33.540		5.75	5.56	1.16	117.720°W	33.540°N	0.00	3.66
PointSourceFinite: -117.720, 33.585		8.74	5.72	1.59	117.720°W	33.585°N	0.00	1.63
PointSourceFinite: -117.720, 33.585		8.74	5.72	1.59	117.720°W	33.585°N	0.00	1.63
PointSourceFinite: -117.720, 33.621		11.22	5.96	1.79	117.720°W	33.621°N	0.00	1.01
PointSourceFinite: -117.720, 33.621		11.22	5.96	1.79	117.720°W	33.621°N	0.00	1.01
UC33brAvg_FM32 (opt)	Grid							17.10
PointSourceFinite: -117.720, 33.540		5.75	5.56	1.16	117.720°W	33.540°N	0.00	3.69
PointSourceFinite: -117.720, 33.540		5.75	5.56	1.16	117.720°W	33.540°N	0.00	3.69
PointSourceFinite: -117.720, 33.585		8.74	5.72	1.59	117.720°W	33.585°N	0.00	1.62
PointSourceFinite: -117.720, 33.585		8.74	5.72	1.59	117.720°W	33.585°N	0.00	1.62

APPENDIX G

RESULTS OF SLOPE STABILITY ANALYSES

33348-08 The Cove at El Niguel Section DR-DR'_Gross_Static

u:\gstabl7 data\33348-08 the cove at el niguel\33348-08 the cove at el niguel_dr-dr'_gross_static_v1.pl2 Run By: Insert Name/company Here 12/23/2020 11:24AM



#	FS
a	2.308
b	2.330
c	2.330
d	2.337
e	2.341
f	2.346
g	2.348
h	2.353
i	2.357
j	2.359

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	120.0	130.0	0.0	30.0	0.00	0.0	W1
LS Debri	2	120.0	130.0	Aniso	Aniso	0.00	0.0	W1
Bdx	3	120.0	130.0	0.0	30.0	0.00	0.0	W1

Load	Value
L1	100 psf
L2	100 psf

GSTABL7 v.2 FSmin=2.308

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

*** 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 **

****
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: 12/23/2020
Time of Run: 11:24AM
Run By: Insert Name/company Here
Input Data Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_gross_static_v1.in
Output Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_gross_static_v1.OUT
Unit System: English

Plotted Output Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_gross_static_v1.PLT

PROBLEM DESCRIPTION: 33348-08 The Cove at El Niguel
Section DR-DR'_Gross_Static

BOUNDARY COORDINATES

48 Top Boundaries
91 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	360.00	55.00	360.00	2
2	55.00	360.00	59.00	360.00	1
3	59.00	360.00	59.01	362.00	1
4	59.01	362.00	70.00	362.00	1
5	70.00	362.00	99.00	373.50	1
6	99.00	373.50	100.00	377.00	1
7	100.00	377.00	108.00	379.95	1
8	108.00	379.95	132.00	379.95	1
9	132.00	379.95	168.00	379.95	1
10	168.00	379.95	182.00	379.95	1
11	182.00	379.95	184.00	379.50	1
12	184.00	379.50	213.00	379.50	1
13	213.00	379.50	214.00	380.19	1
14	214.00	380.19	215.00	380.19	1
15	215.00	380.19	250.00	380.19	1
16	250.00	380.19	254.00	380.19	1
17	254.00	380.19	260.00	380.19	1
18	260.00	380.19	275.00	380.19	1
19	275.00	380.19	278.00	390.44	1
20	278.00	390.44	280.00	389.00	1
21	280.00	389.00	282.00	390.00	1

22	282.00	390.00	326.00	410.00	1
23	326.00	410.00	332.00	407.00	1
24	332.00	407.00	357.00	420.00	1
25	357.00	420.00	395.00	437.00	1
26	395.00	437.00	399.00	434.00	1
27	399.00	434.00	405.00	439.00	1
28	405.00	439.00	460.00	445.00	1
29	460.00	445.00	498.00	447.00	1
30	498.00	447.00	537.00	447.00	1
31	537.00	447.00	541.00	445.00	1
32	541.00	445.00	553.00	446.00	1
33	553.00	446.00	555.00	443.00	1
34	555.00	443.00	557.00	446.00	1
35	557.00	446.00	586.00	453.00	1
36	586.00	453.00	616.00	456.00	1
37	616.00	456.00	644.00	456.00	1
38	644.00	456.00	680.00	458.00	1
39	680.00	458.00	792.00	462.00	1
40	792.00	462.00	825.00	474.00	1
41	825.00	474.00	830.00	471.00	1
42	830.00	471.00	834.00	475.00	1
43	834.00	475.00	885.00	495.00	1
44	885.00	495.00	899.00	495.00	1
45	899.00	495.00	903.00	498.00	1
46	903.00	498.00	955.00	519.00	1
47	955.00	519.00	958.00	516.00	2
48	958.00	516.00	990.00	515.00	2
49	259.00	379.00	268.00	368.00	1
50	268.00	368.00	270.00	365.00	2
51	270.00	365.00	398.00	363.00	2
52	398.00	363.00	488.00	394.00	3
53	488.00	394.00	610.00	430.00	2
54	610.00	430.00	626.00	440.00	2
55	626.00	440.00	748.00	440.00	2
56	748.00	440.00	781.00	420.00	2
57	781.00	420.00	800.00	420.00	2
58	800.00	420.00	841.00	419.00	2
59	841.00	419.00	861.00	421.00	2
60	861.00	421.00	898.00	440.00	2
61	898.00	440.00	912.00	453.00	2
62	912.00	453.00	954.99	493.00	2
63	954.99	493.00	955.00	519.00	2
64	55.00	360.00	70.00	344.00	2
65	70.00	344.00	120.00	346.00	2
66	120.00	346.00	204.00	360.00	2
67	204.00	360.00	244.00	365.00	2
68	244.00	365.00	245.00	368.00	2
69	245.00	368.00	268.00	368.00	2
70	488.00	394.00	508.00	372.00	3
71	508.00	372.00	525.00	360.00	3
72	525.00	360.00	542.00	353.00	3
73	542.00	353.00	576.00	352.00	2
74	576.00	352.00	613.00	355.00	2
75	613.00	355.00	644.00	362.00	2
76	644.00	362.00	700.00	387.00	2
77	700.00	387.00	800.00	420.00	2
78	542.00	353.00	585.00	345.00	3
79	585.00	345.00	622.00	347.00	3
80	622.00	347.00	655.00	351.00	3
81	655.00	351.00	697.00	360.00	3
82	697.00	360.00	720.00	370.00	3
83	720.00	370.00	800.00	392.00	3
84	800.00	392.00	877.00	420.00	3
85	877.00	420.00	905.00	435.00	3
86	905.00	435.00	929.00	450.00	3
87	929.00	450.00	955.00	471.00	3
88	955.00	471.00	990.00	506.00	3
89	0.00	335.00	205.00	338.00	3
90	205.00	338.00	396.00	358.00	3
91	396.00	358.00	398.00	363.00	3

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	130.0	0.0	30.0	0.00	0.0	1
2	120.0	130.0	0.0	13.0	0.00	0.0	1
3	120.0	130.0	0.0	30.0	0.00	0.0	1

ANISOTROPIC STRENGTH PARAMETERS
1 soil type(s)

Soil Type 2 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	0.0	0.00	30.00
2	30.0	0.00	13.00
3	90.0	0.00	30.00

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

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 17 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	338.00
2	120.00	339.00
3	218.00	342.00
4	396.00	358.00

5	488.00	358.00
6	508.00	372.00
7	522.00	380.00
8	580.00	386.00
9	702.00	409.00
10	781.00	420.00
11	800.00	420.00
12	841.00	419.00
13	861.00	421.00
14	898.00	440.00
15	912.00	453.00
16	966.00	466.00
17	990.00	471.00

1

BOUNDARY LOAD(S)

2 Load(s) Specified

Load No.	X-Left (ft)	X-Right (ft)	Intensity (psf)	Deflection (deg)
1	132.00	168.00	100.0	0.0
2	215.00	254.00	100.0	0.0

NOTE - Intensity Is Specified As A Uniformly Distributed Force Acting On A Horizontally Projected Surface.

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

2000 Trial Surfaces Have Been Generated.

80 Surface(s) Initiate(s) From Each Of 25 Points Equally Spaced Along The Ground Surface Between X = 0.00(ft) and X = 100.00(ft)

Each Surface Terminates Between X = 395.00(ft) and X = 990.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = 0.00(ft)

20.00(ft) Line Segments Define Each Trial Failure Surface.

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 (0-1) * *

Selected ki function = Constant (1.0)

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 = 2000

WARNING! The Factor of Safety Calculation for one or More Trial Surfaces Did Not Converge in 20 Iterations.

Number of Trial Surfaces with Non-Converged FS = 275

Number of Trial Surfaces with Misleading FS = 105

Number of Trial Surfaces With Valid FS = 1620

Percentage of Trial Surfaces With Non-Valid FS Solutions of the Total Attempted = 19.0 %

Statistical Data On All Valid FS Values:
FS Max = 6.521 FS Min = 2.308 FS Ave = 3.459
Standard Deviation = 0.418 Coefficient of Variation = 12.10 %

((Modified Bishop FS for Critical Surface = 2.275))

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	16.667	360.000
2	36.250	355.941
3	55.965	352.571
4	75.784	349.894
5	95.686	347.912
6	115.645	346.630
7	135.636	346.047
8	155.636	346.165
9	175.619	346.984
10	195.562	348.502
11	215.438	350.718
12	235.225	353.629
13	254.898	357.232
14	274.433	361.521
15	293.805	366.493
16	312.991	372.140
17	331.968	378.456
18	350.711	385.433
19	369.199	393.062
20	387.408	401.335
21	405.316	410.240
22	422.901	419.767
23	440.142	429.905
24	457.016	440.640
25	463.646	445.192

Circle Center At X = 142.268 ; Y = 916.756 ; and Radius = 570.748

*** FOS = 2.308 Theta (ki=1.0) = 10.05 ***
Lambda = 0.177

Individual data on the 61 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	19.6	4769.2	0.0	0.0	0.	0.	0.0	0.0	0.0
2	18.7	12737.8	0.0	0.0	0.	0.	0.0	0.0	0.0
3	1.0	850.3	0.0	0.0	0.	0.	0.0	0.0	0.0
4	3.0	2780.7	0.0	0.0	0.	0.	0.0	0.0	0.0
5	0.0	10.6	0.0	0.0	0.	0.	0.0	0.0	0.0
6	3.8	4634.9	0.0	0.0	0.	0.	0.0	0.0	0.0
7	7.2	9321.5	0.0	0.0	0.	0.	0.0	0.0	0.0
8	5.8	8928.4	0.0	0.0	0.	0.	0.0	0.0	0.0
9	19.9	46180.1	0.0	0.0	0.	0.	0.0	0.0	0.0
10	3.3	9956.4	0.0	0.0	0.	0.	0.0	0.0	0.0
11	1.0	3309.9	0.0	0.0	0.	0.	0.0	0.0	0.0
12	8.0	29853.1	0.0	0.0	0.	0.	0.0	0.0	0.0
13	7.6	30342.4	0.0	0.0	0.	0.	0.0	0.0	0.0
14	6.9	27760.8	0.0	0.0	0.	0.	0.0	0.0	0.0
15	9.4	38102.0	0.0	0.0	0.	0.	0.0	0.0	0.0
16	3.6	14771.2	0.0	0.0	0.	0.	0.0	0.0	363.6
17	20.0	81224.5	0.0	0.0	0.	0.	0.0	0.0	2000.0
18	12.4	49750.2	0.0	0.0	0.	0.	0.0	0.0	1236.4
19	7.6	30284.4	0.0	0.0	0.	0.	0.0	0.0	0.0
20	6.4	25055.8	0.0	0.0	0.	0.	0.0	0.0	0.0
21	2.0	7723.1	0.0	0.0	0.	0.	0.0	0.0	0.0
22	11.6	43617.1	0.0	0.0	0.	0.	0.0	0.0	0.0
23	8.4	30912.6	0.0	0.0	0.	0.	0.0	0.0	0.0
24	9.0	31920.0	0.0	0.0	0.	0.	0.0	0.0	0.0
25	1.0	3521.2	0.0	0.0	0.	0.	0.0	0.0	0.0
26	1.0	3549.2	0.0	0.0	0.	0.	0.0	0.0	0.0
27	0.4	1551.9	0.0	0.0	0.	0.	0.0	0.0	43.8
28	19.8	66523.2	0.0	0.0	0.	0.	0.0	0.0	1978.7
29	8.8	27121.2	0.0	0.0	0.	0.	0.0	0.0	877.5
30	1.0	2983.5	0.0	0.0	0.	0.	0.0	0.0	100.0
31	5.0	14587.8	0.0	0.0	0.	0.	0.0	0.0	500.0
32	4.0	11274.7	0.0	0.0	0.	0.	0.0	0.0	400.0
33	0.9	2483.6	0.0	0.0	0.	0.	0.0	0.0	0.0
34	4.1	11078.5	0.0	0.0	0.	0.	0.0	0.0	0.0
35	1.0	2633.7	0.0	0.0	0.	0.	0.0	0.0	0.0
36	8.0	20121.1	0.0	0.0	0.	0.	0.0	0.0	0.0
37	2.0	4766.8	0.0	0.0	0.	0.	0.0	0.0	0.0
38	4.4	10189.4	0.0	0.0	0.	0.	0.0	0.0	0.0
39	0.6	1265.6	0.0	0.0	0.	0.	0.0	0.0	0.0
40	3.0	8374.7	0.0	0.0	0.	0.	0.0	0.0	0.0
41	2.0	6486.4	0.0	0.0	0.	0.	0.0	0.0	0.0
42	2.0	6310.4	0.0	0.0	0.	0.	0.0	0.0	0.0
43	5.0	16070.9	0.0	0.0	0.	0.	0.0	0.0	0.0
44	6.8	23176.0	0.0	0.0	0.	0.	0.0	0.0	0.0
45	19.2	70014.0	0.0	0.0	0.	0.	0.0	0.0	0.0
46	13.0	51106.4	0.0	0.0	0.	0.	0.0	0.0	0.0
47	6.0	22232.7	0.0	0.0	0.	0.	0.0	0.0	0.0
48	0.0	110.2	0.0	0.0	0.	0.	0.0	0.0	0.0
49	18.7	67169.4	0.0	0.0	0.	0.	0.0	0.0	0.0
50	6.3	23872.2	0.0	0.0	0.	0.	0.0	0.0	0.0
51	12.2	47113.5	0.0	0.0	0.	0.	0.0	0.0	0.0
52	18.2	70648.5	0.0	0.0	0.	0.	0.0	0.0	0.0
53	7.6	29225.3	0.0	0.0	0.	0.	0.0	0.0	0.0
54	4.0	14109.9	0.0	0.0	0.	0.	0.0	0.0	0.0
55	6.0	20094.6	0.0	0.0	0.	0.	0.0	0.0	0.0
56	0.3	1094.7	0.0	0.0	0.	0.	0.0	0.0	0.0
57	17.6	52734.4	0.0	0.0	0.	0.	0.0	0.0	0.0
58	17.2	35289.1	0.0	0.0	0.	0.	0.0	0.0	0.0
59	16.9	17175.4	0.0	0.0	0.	0.	0.0	0.0	0.0
60	3.0	1136.2	0.0	0.0	0.	0.	0.0	0.0	0.0
61	3.6	505.7	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 26 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4.167	360.000
2	23.604	355.289
3	43.197	351.276
4	62.921	347.964
5	82.751	345.359
6	102.661	343.463
7	122.626	342.280
8	142.620	341.810
9	162.619	342.054
10	182.596	343.012
11	202.526	344.683
12	222.383	347.064
13	242.144	350.152
14	261.781	353.944
15	281.270	358.435
16	300.587	363.619
17	319.706	369.489
18	338.603	376.037
19	357.255	383.257
20	375.637	391.137
21	393.726	399.669
22	411.498	408.842
23	428.932	418.643
24	446.005	429.060
25	462.695	440.080
26	470.363	445.545

Circle Center At X = 145.764 ; Y = 901.425 ; and Radius = 559.634

*** FOS = 2.330 Theta (ki=1.0) = 9.63 ***
Lambda = 0.170

1

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	0.000	360.000
2	19.601	356.027
3	39.326	352.722
4	59.152	350.089
5	79.056	348.132
6	99.015	346.853
7	119.006	346.253
8	139.006	346.332
9	158.992	347.092
10	178.940	348.530
11	198.828	350.645
12	218.632	353.435
13	238.330	356.897
14	257.900	361.026
15	277.317	365.818
16	296.560	371.267
17	315.607	377.368
18	334.436	384.112
19	353.025	391.492
20	371.352	399.499
21	389.396	408.125

22	407.136	417.359
23	424.553	427.191
24	441.625	437.610
25	451.439	444.066

Circle Center At X = 126.665 ; Y = 934.534 ; and Radius = 588.331

*** FOS = 2.330 Theta (ki=1.0) = 10.04 ***
Lambda = 0.177

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	54.167	360.000
2	73.503	354.889
3	93.028	350.560
4	112.712	347.018
5	132.523	344.271
6	152.428	342.321
7	172.395	341.173
8	192.392	340.828
9	212.386	341.287
10	232.346	342.549
11	252.240	344.613
12	272.034	347.473
13	291.698	351.127
14	311.198	355.568
15	330.505	360.789
16	349.586	366.781
17	368.411	373.536
18	386.949	381.041
19	405.171	389.285
20	423.047	398.255
21	440.548	407.935
22	457.646	418.311
23	474.313	429.366
24	490.522	441.082
25	498.054	447.000

Circle Center At X = 190.971 ; Y = 838.467 ; and Radius = 497.641

*** FOS = 2.337 Theta (ki=1.0) = 9.19 ***
Lambda = 0.162

1

Failure Surface Specified By 21 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.000	377.000
2	118.376	369.105
3	137.212	362.381
4	156.433	356.856
5	175.964	352.551
6	195.728	349.483
7	215.645	347.664
8	235.637	347.102
9	255.625	347.799

10	275.529	349.752
11	295.271	352.953
12	314.773	357.390
13	333.957	363.044
14	352.747	369.895
15	371.069	377.914
16	388.850	387.070
17	406.020	397.326
18	422.511	408.642
19	438.258	420.973
20	453.197	434.270
21	464.033	445.212

Circle Center At X = 234.566 ; Y = 664.861 ; and Radius = 317.761

*** FOS = 2.341 Theta (ki=1.0) = 8.82 ***
Lambda = 0.155

Failure Surface Specified By 23 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	41.667	360.000
2	60.921	354.589
3	80.403	350.068
4	100.072	346.447
5	119.888	343.733
6	139.806	341.932
7	159.787	341.049
8	179.787	341.084
9	199.764	342.038
10	219.676	343.909
11	239.482	346.693
12	259.138	350.384
13	278.604	354.973
14	297.839	360.453
15	316.802	366.810
16	335.453	374.031
17	353.752	382.102
18	371.661	391.005
19	389.142	400.721
20	406.159	411.231
21	422.674	422.510
22	438.654	434.537
23	449.981	443.907

Circle Center At X = 169.018 ; Y = 776.177 ; and Radius = 435.226

*** FOS = 2.346 Theta (ki=1.0) = 9.96 ***
Lambda = 0.176

1

Failure Surface Specified By 24 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	62.500	362.000
2	81.687	356.355
3	101.106	351.569

4	120.718	347.652
5	140.486	344.611
6	160.369	342.454
7	180.329	341.183
8	200.325	340.802
9	220.319	341.310
10	240.270	342.709
11	260.139	344.993
12	279.887	348.160
13	299.474	352.202
14	318.862	357.112
15	338.012	362.880
16	356.887	369.494
17	375.448	376.941
18	393.660	385.208
19	411.486	394.276
20	428.891	404.129
21	445.840	414.747
22	462.299	426.108
23	478.237	438.190
24	488.223	446.485

Circle Center At X = 198.865 ; Y = 789.711 ; and Radius = 448.923

*** FOS = 2.348 Theta (ki=1.0) = 9.19 ***
Lambda = 0.162

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	50.000	360.000
2	69.436	355.284
3	89.036	351.306
4	108.773	348.072
5	128.618	345.588
6	148.543	343.856
7	168.519	342.880
8	188.518	342.659
9	208.511	343.196
10	228.469	344.488
11	248.364	346.535
12	268.168	349.333
13	287.851	352.878
14	307.386	357.165
15	326.745	362.189
16	345.900	367.941
17	364.824	374.413
18	383.489	381.597
19	401.869	389.482
20	419.938	398.057
21	437.669	407.309
22	455.037	417.225
23	472.018	427.791
24	488.587	438.992
25	499.517	447.000

Circle Center At X = 184.339 ; Y = 871.190 ; and Radius = 528.547

*** FOS = 2.353 Theta (ki=1.0) = 9.31 ***
Lambda = 0.164

1

Failure Surface Specified By 23 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	62.500	362.000
2	81.732	356.510
3	101.204	351.944
4	120.871	348.311
5	140.689	345.620
6	160.613	343.877
7	180.597	343.086
8	200.597	343.248
9	220.565	344.364
10	240.458	346.431
11	260.230	349.443
12	279.836	353.395
13	299.231	358.278
14	318.371	364.079
15	337.212	370.787
16	355.713	378.385
17	373.830	386.856
18	391.523	396.182
19	408.751	406.341
20	425.475	417.310
21	441.657	429.063
22	457.260	441.575
23	461.209	445.064

Circle Center At X = 187.190 ; Y = 762.392 ; and Radius = 419.358

*** FOS = 2.357 Theta (ki=1.0) = 9.69 ***
Lambda = 0.171

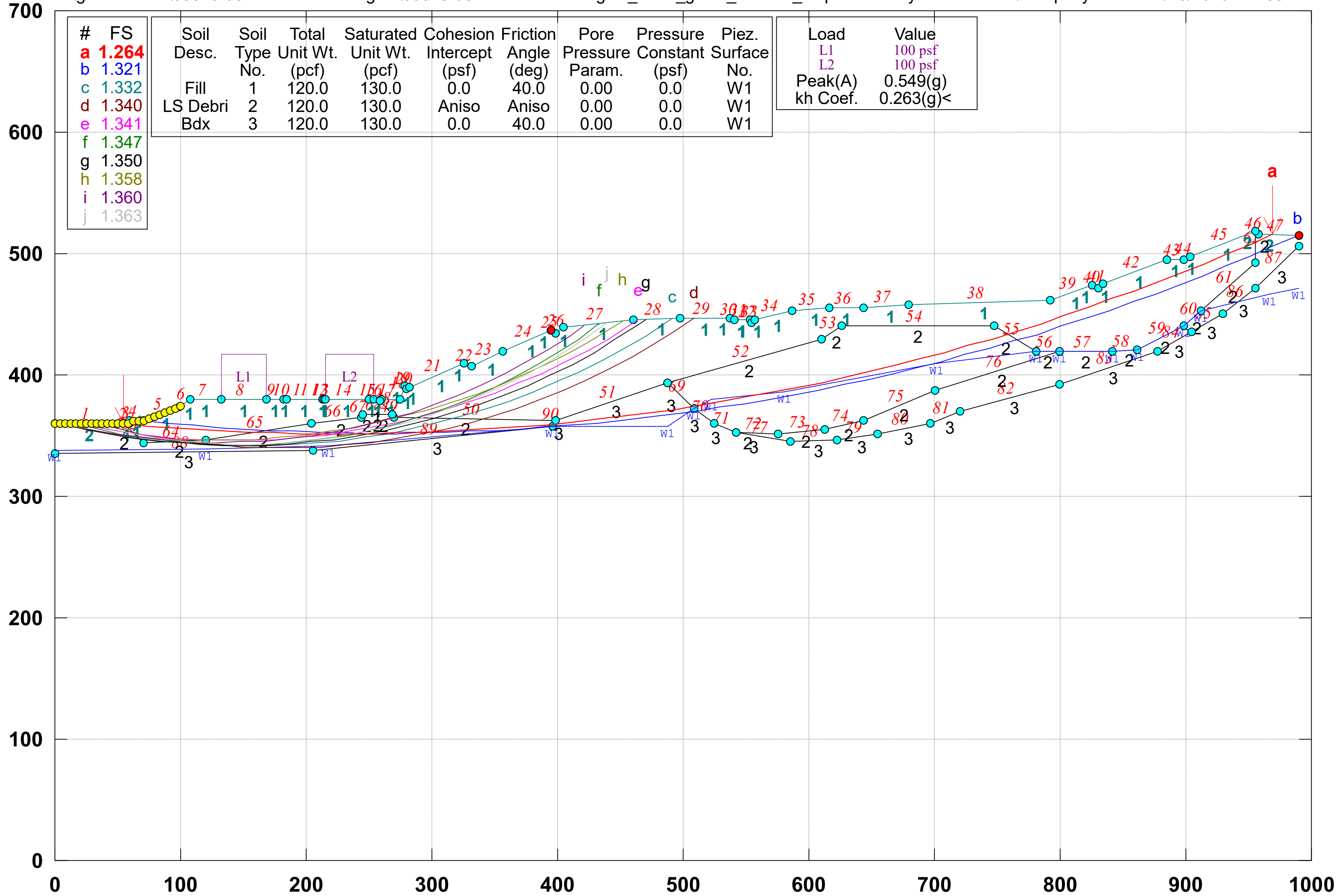
Failure Surface Specified By 27 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	8.333	360.000
2	27.690	354.970
3	47.214	350.629
4	66.878	346.983
5	86.660	344.036
6	106.534	341.792
7	126.475	340.254
8	146.457	339.423
9	166.457	339.302
10	186.448	339.889
11	206.406	341.185
12	226.306	343.187
13	246.122	345.893
14	265.830	349.300
15	285.404	353.403
16	304.821	358.198
17	324.056	363.677
18	343.084	369.835
19	361.882	376.664
20	380.427	384.155
21	398.694	392.298
22	416.661	401.083
23	434.305	410.500
24	451.605	420.536

```
   25        468.537      431.180
   26        485.082      442.416
   27        490.827      446.622

Circle Center At X =   159.886 ; Y =   903.440 ; and Radius =   564.177


***  FOS =     2.359   Theta (ki=1.0) =     9.36  ***
                    Lambda =   0.165



          **** END OF GSTABL7 OUTPUT ****
```

33348-08 The Cove at El Niguel Section DR-DR'_Gross_Seismic

u:\gstabl7 data\33348-08 the cove at el niguel\33348-08 the cove at el niguel_dr-dr'_gross_seismic_v2.pl2 Run By: Insert Name/company Here 12/23/2020 11:53AM



GSTABL7 v.2 FSmin=1.264

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

*** 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 **

****

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: 12/23/2020
Time of Run: 11:53AM
Run By: Insert Name/company Here
Input Data Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_gross_seismic_v2.in
Output Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_gross_seismic_v2.OUT
Unit System: English

Plotted Output Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_gross_seismic_v2.PLT

PROBLEM DESCRIPTION: 33348-08 The Cove at El Niguel
Section DR-DR'_Gross_Seismic

BOUNDARY COORDINATES

47 Top Boundaries
90 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	360.00	55.00	360.00	2
2	55.00	360.00	59.00	360.00	1
3	59.00	360.00	59.01	362.00	1
4	59.01	362.00	70.00	362.00	1
5	70.00	362.00	99.00	373.50	1
6	99.00	373.50	108.00	379.95	1
7	108.00	379.95	132.00	379.95	1
8	132.00	379.95	168.00	379.95	1
9	168.00	379.95	182.00	379.95	1
10	182.00	379.95	184.00	379.50	1
11	184.00	379.50	213.00	379.50	1
12	213.00	379.50	214.00	380.19	1
13	214.00	380.19	215.00	380.19	1
14	215.00	380.19	250.00	380.19	1
15	250.00	380.19	254.00	380.19	1
16	254.00	380.19	260.00	380.19	1
17	260.00	380.19	275.00	380.19	1
18	275.00	380.19	278.00	390.44	1
19	278.00	390.44	280.00	389.00	1
20	280.00	389.00	282.00	390.00	1
21	282.00	390.00	326.00	410.00	1

22	326.00	410.00	332.00	407.00	1
23	332.00	407.00	357.00	420.00	1
24	357.00	420.00	395.00	437.00	1
25	395.00	437.00	399.00	434.00	1
26	399.00	434.00	405.00	439.00	1
27	405.00	439.00	460.00	445.00	1
28	460.00	445.00	498.00	447.00	1
29	498.00	447.00	537.00	447.00	1
30	537.00	447.00	541.00	445.00	1
31	541.00	445.00	553.00	446.00	1
32	553.00	446.00	555.00	443.00	1
33	555.00	443.00	557.00	446.00	1
34	557.00	446.00	586.00	453.00	1
35	586.00	453.00	616.00	456.00	1
36	616.00	456.00	644.00	456.00	1
37	644.00	456.00	680.00	458.00	1
38	680.00	458.00	792.00	462.00	1
39	792.00	462.00	825.00	474.00	1
40	825.00	474.00	830.00	471.00	1
41	830.00	471.00	834.00	475.00	1
42	834.00	475.00	885.00	495.00	1
43	885.00	495.00	899.00	495.00	1
44	899.00	495.00	903.00	498.00	1
45	903.00	498.00	955.00	519.00	1
46	955.00	519.00	958.00	516.00	2
47	958.00	516.00	990.00	515.00	2
48	259.00	379.00	268.00	368.00	1
49	268.00	368.00	270.00	365.00	2
50	270.00	365.00	398.00	363.00	2
51	398.00	363.00	488.00	394.00	3
52	488.00	394.00	610.00	430.00	2
53	610.00	430.00	626.00	440.00	2
54	626.00	440.00	748.00	440.00	2
55	748.00	440.00	781.00	420.00	2
56	781.00	420.00	800.00	420.00	2
57	800.00	420.00	841.00	419.00	2
58	841.00	419.00	861.00	421.00	2
59	861.00	421.00	898.00	440.00	2
60	898.00	440.00	912.00	453.00	2
61	912.00	453.00	954.99	493.00	2
62	954.99	493.00	955.00	519.00	2
63	55.00	360.00	70.00	344.00	2
64	70.00	344.00	120.00	346.00	2
65	120.00	346.00	204.00	360.00	2
66	204.00	360.00	244.00	365.00	2
67	244.00	365.00	245.00	368.00	2
68	245.00	368.00	268.00	368.00	2
69	488.00	394.00	508.00	372.00	3
70	508.00	372.00	525.00	360.00	3
71	525.00	360.00	542.00	353.00	3
72	542.00	353.00	576.00	352.00	2
73	576.00	352.00	613.00	355.00	2
74	613.00	355.00	644.00	362.00	2
75	644.00	362.00	700.00	387.00	2
76	700.00	387.00	800.00	420.00	2
77	542.00	353.00	585.00	345.00	3
78	585.00	345.00	622.00	347.00	3
79	622.00	347.00	655.00	351.00	3
80	655.00	351.00	697.00	360.00	3
81	697.00	360.00	720.00	370.00	3
82	720.00	370.00	800.00	392.00	3
83	800.00	392.00	877.00	420.00	3
84	877.00	420.00	905.00	435.00	3
85	905.00	435.00	929.00	450.00	3
86	929.00	450.00	955.00	471.00	3
87	955.00	471.00	990.00	506.00	3
88	0.00	335.00	205.00	338.00	3
89	205.00	338.00	396.00	358.00	3
90	396.00	358.00	398.00	363.00	3

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	130.0	0.0	40.0	0.00	0.0	1
2	120.0	130.0	0.0	17.3	0.00	0.0	1
3	120.0	130.0	0.0	40.0	0.00	0.0	1

ANISOTROPIC STRENGTH PARAMETERS
1 soil type(s)

Soil Type 2 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	0.0	0.00	40.00
2	30.0	0.00	17.33
3	90.0	0.00	40.00

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

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 17 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	338.00
2	120.00	339.00
3	218.00	342.00
4	396.00	358.00
5	488.00	358.00

6	508.00	372.00
7	522.00	380.00
8	580.00	386.00
9	702.00	409.00
10	781.00	420.00
11	800.00	420.00
12	841.00	419.00
13	861.00	421.00
14	898.00	440.00
15	912.00	453.00
16	966.00	466.00
17	990.00	471.00

1

BOUNDARY LOAD(S)

2 Load(s) Specified

Load No.	X-Left (ft)	X-Right (ft)	Intensity (psf)	Deflection (deg)
1	132.00	168.00	100.0	0.0
2	215.00	254.00	100.0	0.0

NOTE - Intensity Is Specified As A Uniformly Distributed
Force Acting On A Horizontally Projected Surface.

Specified Peak Ground Acceleration Coefficient (A) = 0.549(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.263(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)

Specified Seismic Pore-Pressure Factor = 0.000

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

2000 Trial Surfaces Have Been Generated.

80 Surface(s) Initiate(s) From Each Of 25 Points Equally Spaced Along The Ground Surface Between X = 0.00(ft)
and X = 100.00(ft)

Each Surface Terminates Between X = 395.00(ft)
and X = 990.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = 0.00(ft)

20.00(ft) Line Segments Define Each Trial Failure Surface.

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 (0-1) * *

Selected ki function = Constant (1.0)

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 = 2000

WARNING! The Factor of Safety Calculation for one or More Trial Surfaces Did Not Converge in 20 Iterations.

Number of Trial Surfaces with Non-Converged FS = 746

Number of Trial Surfaces with Misleading FS = 434

Number of Trial Surfaces With Valid FS = 820

Percentage of Trial Surfaces With Non-Valid FS Solutions of the Total Attempted = 59.0 %

Statistical Data On All Valid FS Values:
FS Max = 21.090 FS Min = 1.264 FS Ave = 1.831
Standard Deviation = 0.712 Coefficient of Variation = 38.88 %

((Modified Bishop FS for Critical Surface = 1.213))

Failure Surface Specified By 48 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	54.167	360.000
2	74.081	358.154
3	94.016	356.534
4	113.967	355.140
5	133.933	353.974
6	153.911	353.035
7	173.898	352.323
8	193.892	351.838
9	213.891	351.580
10	233.891	351.550
11	253.890	351.747
12	273.885	352.172
13	293.875	352.823
14	313.855	353.702
15	333.825	354.809
16	353.780	356.142
17	373.719	357.701
18	393.639	359.488
19	413.538	361.500
20	433.412	363.739
21	453.260	366.204
22	473.078	368.894
23	492.864	371.809
24	512.616	374.949
25	532.331	378.313
26	552.007	381.901
27	571.640	385.713
28	591.229	389.747

29	610.771	394.004
30	630.263	398.483
31	649.703	403.183
32	669.088	408.104
33	688.416	413.245
34	707.684	418.605
35	726.890	424.184
36	746.032	429.981
37	765.106	435.995
38	784.111	442.226
39	803.044	448.672
40	821.902	455.333
41	840.683	462.208
42	859.385	469.296
43	878.005	476.597
44	896.541	484.108
45	914.990	491.830
46	933.350	499.760
47	951.619	507.900
48	968.542	515.671

Circle Center At X = 226.549 ; Y = 2110.720 ; and Radius = 1759.186

*** FOS = 1.264 Theta (ki=1.0) = 7.04 ***
Lambda = 0.123

Individual data on the 113 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	0.8	3.9	0.0	0.0	0.	0.	1.0	0.0	0.0
2	0.1	0.8	0.0	0.0	0.	0.	0.2	0.0	0.0
3	3.9	125.3	0.0	0.0	0.	0.	33.0	0.0	0.0
4	0.0	1.7	0.0	0.0	0.	0.	0.5	0.0	0.0
5	11.0	3901.7	0.0	0.0	0.	0.	1026.2	0.0	0.0
6	4.1	2187.5	0.0	0.0	0.	0.	575.3	0.0	0.0
7	19.9	24465.1	0.0	0.0	0.	0.	6434.3	0.0	0.0
8	5.0	9661.2	0.0	0.0	0.	0.	2540.9	0.0	0.0
9	9.0	22522.0	0.0	0.0	0.	0.	5923.3	0.0	0.0
10	6.0	17615.4	0.0	0.0	0.	0.	4632.9	0.0	0.0
11	18.0	54827.0	0.0	0.0	0.	0.	14419.5	0.0	0.0
12	1.9	6012.1	0.0	0.0	0.	0.	1581.2	0.0	193.3
13	20.0	63399.5	0.0	0.0	0.	0.	16674.1	0.0	1997.8
14	6.8	22178.5	0.0	0.0	0.	0.	5832.9	0.0	683.6
15	7.3	23751.5	0.0	0.0	0.	0.	6246.6	0.0	725.3
16	5.9	19479.8	0.0	0.0	0.	0.	5123.2	0.0	0.0
17	8.1	26955.4	0.0	0.0	0.	0.	7089.3	0.0	0.0
18	2.0	6629.6	0.0	0.0	0.	0.	1743.6	0.0	0.0
19	9.9	32694.9	0.0	0.0	0.	0.	8598.8	0.0	0.0
20	10.1	33631.2	0.0	0.0	0.	0.	8845.0	0.0	0.0
21	9.0	30078.5	0.0	0.0	0.	0.	7910.6	0.0	0.0
22	0.9	3016.2	0.0	0.0	0.	0.	793.3	0.0	0.0
23	0.1	375.0	0.0	0.0	0.	0.	98.6	0.0	0.0
24	1.0	3433.3	0.0	0.0	0.	0.	903.0	0.0	0.0
25	18.9	64891.1	0.0	0.0	0.	0.	17066.4	0.0	1889.1
26	10.1	34683.5	0.0	0.0	0.	0.	9121.8	0.0	1010.9
27	1.0	3424.3	0.0	0.0	0.	0.	900.6	0.0	100.0
28	5.0	17103.5	0.0	0.0	0.	0.	4498.2	0.0	500.0
29	3.9	13284.9	0.0	0.0	0.	0.	3493.9	0.0	389.0
30	0.1	376.6	0.0	0.0	0.	0.	99.1	0.0	11.0
31	5.0	17032.5	0.0	0.0	0.	0.	4479.5	0.0	0.0
32	1.0	3398.9	0.0	0.0	0.	0.	893.9	0.0	0.0
33	8.0	27099.1	0.0	0.0	0.	0.	7127.1	0.0	0.0

34	2.0	6749.3	0.0	0.0	0.	0.	1775.1	0.0	0.0
35	3.9	13081.8	0.0	0.0	0.	0.	3440.5	0.0	0.0
36	1.1	3746.0	0.0	0.0	0.	0.	985.2	0.0	0.0
37	3.0	11900.9	0.0	0.0	0.	0.	3129.9	0.0	0.0
38	2.0	8971.6	0.0	0.0	0.	0.	2359.5	0.0	0.0
39	2.0	8903.1	0.0	0.0	0.	0.	2341.5	0.0	0.0
40	11.9	57095.8	0.0	0.0	0.	0.	15016.2	0.0	0.0
41	20.0	111913.2	0.0	0.0	0.	0.	29433.2	0.0	0.0
42	12.1	77534.0	0.0	0.0	0.	0.	20391.4	0.0	0.0
43	6.0	38850.3	0.0	0.0	0.	0.	10217.6	0.0	0.0
44	1.8	11542.3	0.0	0.0	0.	0.	3035.6	0.0	0.0
45	20.0	138081.5	0.0	0.0	0.	0.	36315.4	0.0	0.0
46	3.2	24301.8	0.0	0.0	0.	0.	6391.4	0.0	0.0
47	16.7	133805.4	0.0	0.0	0.	0.	35190.8	0.0	0.0
48	19.9	175314.8	0.0	0.0	0.	0.	46107.8	0.0	0.0
49	1.4	12595.7	0.0	0.0	0.	0.	3312.7	0.0	0.0
50	1.7	15816.6	0.0	0.0	0.	0.	4159.8	0.0	0.0
51	1.3	11578.7	0.0	0.0	0.	0.	3045.2	0.0	0.0
52	1.0	8927.5	0.0	0.0	0.	0.	2347.9	0.0	0.0
53	6.0	54839.9	0.0	0.0	0.	0.	14422.9	0.0	0.0
54	8.5	80320.1	0.0	0.0	0.	0.	21124.2	0.0	0.0
55	19.9	186967.0	0.0	0.0	0.	0.	49172.3	0.0	0.0
56	19.8	186274.9	0.0	0.0	0.	0.	48990.3	0.0	0.0
57	6.7	63066.7	0.0	0.0	0.	0.	16586.5	0.0	0.0
58	13.1	121369.9	0.0	0.0	0.	0.	31920.3	0.0	0.0
59	14.9	136247.2	0.0	0.0	0.	0.	35833.0	0.0	0.0
60	4.9	43866.8	0.0	0.0	0.	0.	11537.0	0.0	0.0
61	5.1	46004.3	0.0	0.0	0.	0.	12099.1	0.0	0.0
62	8.2	72899.3	0.0	0.0	0.	0.	19172.5	0.0	0.0
63	6.4	55512.6	0.0	0.0	0.	0.	14599.8	0.0	0.0
64	0.8	6702.7	0.0	0.0	0.	0.	1762.8	0.0	0.0
65	8.6	73658.9	0.0	824.2	0.	0.	19372.3	0.0	0.0
66	10.3	86566.7	0.0	2018.4	0.	0.	22767.0	0.0	0.0
67	4.7	38362.4	0.0	757.5	0.	0.	10089.3	0.0	0.0
68	4.0	31994.7	0.0	562.6	0.	0.	8414.6	0.0	0.0
69	11.0	85452.3	0.0	1137.0	0.	0.	22474.0	0.0	0.0
70	1.0	7634.8	0.0	72.7	0.	0.	2007.9	0.0	0.0
71	2.0	14951.2	0.0	129.3	0.	0.	3932.2	0.0	0.0
72	2.0	14854.4	0.0	106.3	0.	0.	3906.7	0.0	0.0
73	8.3	62882.9	0.0	196.1	0.	0.	16538.2	0.0	0.0
74	6.4	48662.8	0.0	0.0	0.	0.	12798.3	0.0	0.0
75	14.4	110413.1	0.0	0.0	0.	0.	29038.6	0.0	0.0
76	5.2	40192.2	0.0	0.0	0.	0.	10570.6	0.0	0.0
77	18.8	141164.0	0.0	0.0	0.	0.	37126.1	0.0	0.0
78	0.8	5689.8	0.0	0.0	0.	0.	1496.4	0.0	0.0
79	5.2	38361.8	0.0	0.0	0.	0.	10089.2	0.0	0.0
80	10.0	71574.3	0.0	0.0	0.	0.	18824.0	0.0	0.0
81	4.3	29672.6	0.0	0.0	0.	0.	7803.9	0.0	0.0
82	13.7	92076.8	0.0	0.0	0.	0.	24216.2	0.0	0.0
83	5.7	36723.9	0.0	0.0	0.	0.	9658.4	0.0	0.0
84	19.4	119129.7	0.0	0.0	0.	0.	31331.1	0.0	0.0
85	10.9	63039.2	0.0	0.0	0.	0.	16579.3	0.0	0.0
86	8.4	46480.5	0.0	0.0	0.	0.	12224.4	0.0	0.0
87	19.3	98776.2	0.0	0.0	0.	0.	25978.1	0.0	0.0
88	19.2	87434.8	0.0	0.0	0.	0.	22995.3	0.0	0.0
89	19.1	75648.4	0.0	0.0	0.	0.	19895.5	0.0	0.0
90	2.0	7109.6	0.0	0.0	0.	0.	1869.8	0.0	0.0
91	10.2	34764.5	0.0	0.0	0.	0.	9143.1	0.0	0.0
92	6.9	21553.6	0.0	0.0	0.	0.	5668.6	0.0	0.0
93	19.0	50784.3	0.0	0.0	0.	0.	13356.3	0.0	0.0
94	7.9	17315.3	0.0	0.0	0.	0.	4553.9	0.0	0.0
95	11.0	22814.8	0.0	0.0	0.	0.	6000.3	0.0	0.0
96	18.9	39470.7	0.0	0.0	0.	0.	10380.8	0.0	0.0
97	3.1	6520.0	0.0	0.0	0.	0.	1714.8	0.0	0.0
98	5.0	9070.5	0.0	0.0	0.	0.	2385.5	0.0	0.0
99	4.0	6705.7	0.0	0.0	0.	0.	1763.6	0.0	0.0
100	6.7	12290.0	0.0	0.0	0.	0.	3232.3	0.0	0.0
101	18.7	34865.1	0.0	0.0	0.	0.	9169.5	0.0	0.0
102	18.6	34989.4	0.0	0.0	0.	0.	9202.2	0.0	0.0
103	7.0	13107.3	0.0	0.0	0.	0.	3447.2	0.0	0.0
104	11.5	18322.5	0.0	0.0	0.	0.	4818.8	0.0	0.0

105	2.5	3062.6	0.0	0.0	0.	0.	805.5	0.0	0.0
106	4.0	5052.3	0.0	0.0	0.	0.	1328.8	0.0	0.0
107	12.0	15971.4	0.0	0.0	0.	0.	4200.5	0.0	0.0
108	18.4	23694.5	0.0	0.0	0.	0.	6231.7	0.0	0.0
109	18.3	22176.5	0.0	0.0	0.	0.	5832.4	0.0	0.0
110	3.4	3907.3	0.0	0.0	0.	0.	1027.6	0.0	0.0
111	0.0	4.2	0.0	0.0	0.	0.	1.1	0.0	0.0
112	3.0	2649.3	0.0	0.0	0.	0.	696.8	0.0	0.0
113	10.5	3270.1	0.0	0.0	0.	0.	860.0	0.0	0.0

Failure Surface Specified By 49 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	70.833	362.330
2	90.725	360.252
3	110.640	358.406
4	130.574	356.793
5	150.527	355.412
6	170.494	354.264
7	190.473	353.349
8	210.461	352.667
9	230.456	352.219
10	250.455	352.003
11	270.455	352.021
12	290.453	352.273
13	310.448	352.758
14	330.435	353.476
15	350.412	354.427
16	370.377	355.611
17	390.327	357.028
18	410.259	358.677
19	430.170	360.559
20	450.058	362.673
21	469.920	365.020
22	489.753	367.597
23	509.555	370.406
24	529.322	373.446
25	549.053	376.716
26	568.744	380.216
27	588.394	383.946
28	607.998	387.904
29	627.555	392.091
30	647.061	396.506
31	666.515	401.148
32	685.914	406.017
33	705.254	411.112
34	724.533	416.432
35	743.749	421.977
36	762.899	427.745
37	781.981	433.737
38	800.991	439.951
39	819.927	446.386
40	838.787	453.041
41	857.569	459.916
42	876.268	467.010
43	894.884	474.322
44	913.413	481.850
45	931.853	489.594
46	950.201	497.552
47	968.456	505.724
48	986.613	514.109
49	988.579	515.044

Circle Center At X =   258.904 ; Y = 2066.219 ; and Radius = 1714.237

*** FOS =   1.321   Theta (ki=1.0) =   7.31 ***
Lambda =   0.128

1

Failure Surface Specified By 27 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	8.333	360.000
2	27.789	355.366
3	47.390	351.392
4	67.115	348.083
5	86.939	345.442
6	106.842	343.473
7	126.800	342.177
8	146.791	341.557
9	166.791	341.613
10	186.777	342.344
11	206.728	343.751
12	226.619	345.831
13	246.429	348.582
14	266.135	352.001
15	285.714	356.084
16	305.143	360.826
17	324.401	366.222
18	343.466	372.266
19	362.316	378.951
20	380.929	386.269
21	399.284	394.212
22	417.360	402.771
23	435.137	411.936
24	452.594	421.696
25	469.711	432.040
26	486.468	442.958
27	491.769	446.672

Circle Center At X = 155.144 ; Y = 933.210 ; and Radius = 591.712

*** FOS = 1.332 Theta (ki=1.0) = 7.07 ***
Lambda = 0.124

Failure Surface Specified By 27 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	33.333	360.000
2	52.710	355.045
3	72.252	350.789
4	91.934	347.237
5	111.731	344.395
6	131.617	342.266
7	151.567	340.852
8	171.555	340.156
9	191.555	340.179
10	211.541	340.919
11	231.488	342.378
12	251.370	344.551
13	271.160	347.438
14	290.834	351.033
15	310.367	355.333
16	329.732	360.331
17	348.905	366.022

18	367.862	372.398
19	386.577	379.451
20	405.027	387.171
21	423.187	395.549
22	441.035	404.574
23	458.548	414.234
24	475.702	424.518
25	492.475	435.410
26	508.847	446.898
27	508.981	447.000

Circle Center At X = 180.933 ; Y = 896.796 ; and Radius = 556.719

*** FOS = 1.340 Theta (ki=1.0) = 6.82 ***
Lambda = 0.120

1

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	16.667	360.000
2	36.250	355.941
3	55.965	352.571
4	75.784	349.894
5	95.686	347.912
6	115.645	346.630
7	135.636	346.047
8	155.636	346.165
9	175.619	346.984
10	195.562	348.502
11	215.438	350.718
12	235.225	353.629
13	254.898	357.232
14	274.433	361.521
15	293.805	366.493
16	312.991	372.140
17	331.968	378.456
18	350.711	385.433
19	369.199	393.062
20	387.408	401.335
21	405.316	410.240
22	422.901	419.767
23	440.142	429.905
24	457.016	440.640
25	463.646	445.192

Circle Center At X = 142.268 ; Y = 916.756 ; and Radius = 570.748

*** FOS = 1.341 Theta (ki=1.0) = 7.87 ***
Lambda = 0.138

Failure Surface Specified By 24 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	16.667	360.000
2	36.006	354.901
3	55.546	350.639

4	75.252	347.222
5	95.087	344.657
6	115.014	342.948
7	134.996	342.098
8	154.996	342.109
9	174.977	342.982
10	194.902	344.713
11	214.734	347.301
12	234.436	350.740
13	253.972	355.024
14	273.305	360.145
15	292.400	366.094
16	311.221	372.859
17	329.733	380.428
18	347.903	388.787
19	365.695	397.920
20	383.079	407.811
21	400.020	418.441
22	416.488	429.790
23	432.452	441.838
24	432.671	442.019

Circle Center At X = 144.732 ; Y = 806.455 ; and Radius = 464.460

*** FOS = 1.347 Theta (ki=1.0) = 7.59 ***
Lambda = 0.133

1

Failure Surface Specified By 26 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	4.167	360.000
2	23.604	355.289
3	43.197	351.276
4	62.921	347.964
5	82.751	345.359
6	102.661	343.463
7	122.626	342.280
8	142.620	341.810
9	162.619	342.054
10	182.596	343.012
11	202.526	344.683
12	222.383	347.064
13	242.144	350.152
14	261.781	353.944
15	281.270	358.435
16	300.587	363.619
17	319.706	369.489
18	338.603	376.037
19	357.255	383.257
20	375.637	391.137
21	393.726	399.669
22	411.498	408.842
23	428.932	418.643
24	446.005	429.060
25	462.695	440.080
26	470.363	445.545

Circle Center At X = 145.764 ; Y = 901.425 ; and Radius = 559.634

*** FOS = 1.350 Theta (ki=1.0) = 7.46 ***
Lambda = 0.131

Failure Surface Specified By 25 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	0.000	360.000
2	19.601	356.027
3	39.326	352.722
4	59.152	350.089
5	79.056	348.132
6	99.015	346.853
7	119.006	346.253
8	139.006	346.332
9	158.992	347.092
10	178.940	348.530
11	198.828	350.645
12	218.632	353.435
13	238.330	356.897
14	257.900	361.026
15	277.317	365.818
16	296.560	371.267
17	315.607	377.368
18	334.436	384.112
19	353.025	391.492
20	371.352	399.499
21	389.396	408.125
22	407.136	417.359
23	424.553	427.191
24	441.625	437.610
25	451.439	444.066

Circle Center At X = 126.665 ; Y = 934.534 ; and Radius = 588.331

*** FOS = 1.358 Theta (ki=1.0) = 15.45 ***
Lambda = 0.276

1

Failure Surface Specified By 23 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	8.333	360.000
2	27.792	355.378
3	47.424	351.559
4	67.196	348.549
5	87.075	346.355
6	107.028	344.979
7	127.020	344.424
8	147.019	344.691
9	166.989	345.779
10	186.898	347.686
11	206.711	350.410
12	226.396	353.945
13	245.920	358.287
14	265.248	363.426
15	284.349	369.355
16	303.190	376.064
17	321.740	383.542
18	339.967	391.775
19	357.840	400.749
20	375.329	410.451

21	392.405	420.863
22	409.039	431.967
23	421.100	440.756

Circle Center At X = 130.531 ; Y = 831.145 ; and Radius = 486.734

*** FOS = 1.360 Theta (ki=1.0) = 7.51 ***
Lambda = 0.132

Failure Surface Specified By 23 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	25.000	360.000
2	44.379	355.056
3	63.952	350.945
4	83.683	347.673
5	103.535	345.248
6	123.473	343.672
7	143.460	342.950
8	163.460	343.082
9	183.435	344.068
10	203.351	345.907
11	223.169	348.595
12	242.855	352.126
13	262.372	356.496
14	281.684	361.695
15	300.757	367.714
16	319.555	374.543
17	338.044	382.168
18	356.191	390.576
19	373.962	399.752
20	391.325	409.678
21	408.248	420.337
22	424.700	431.709
23	439.285	442.740

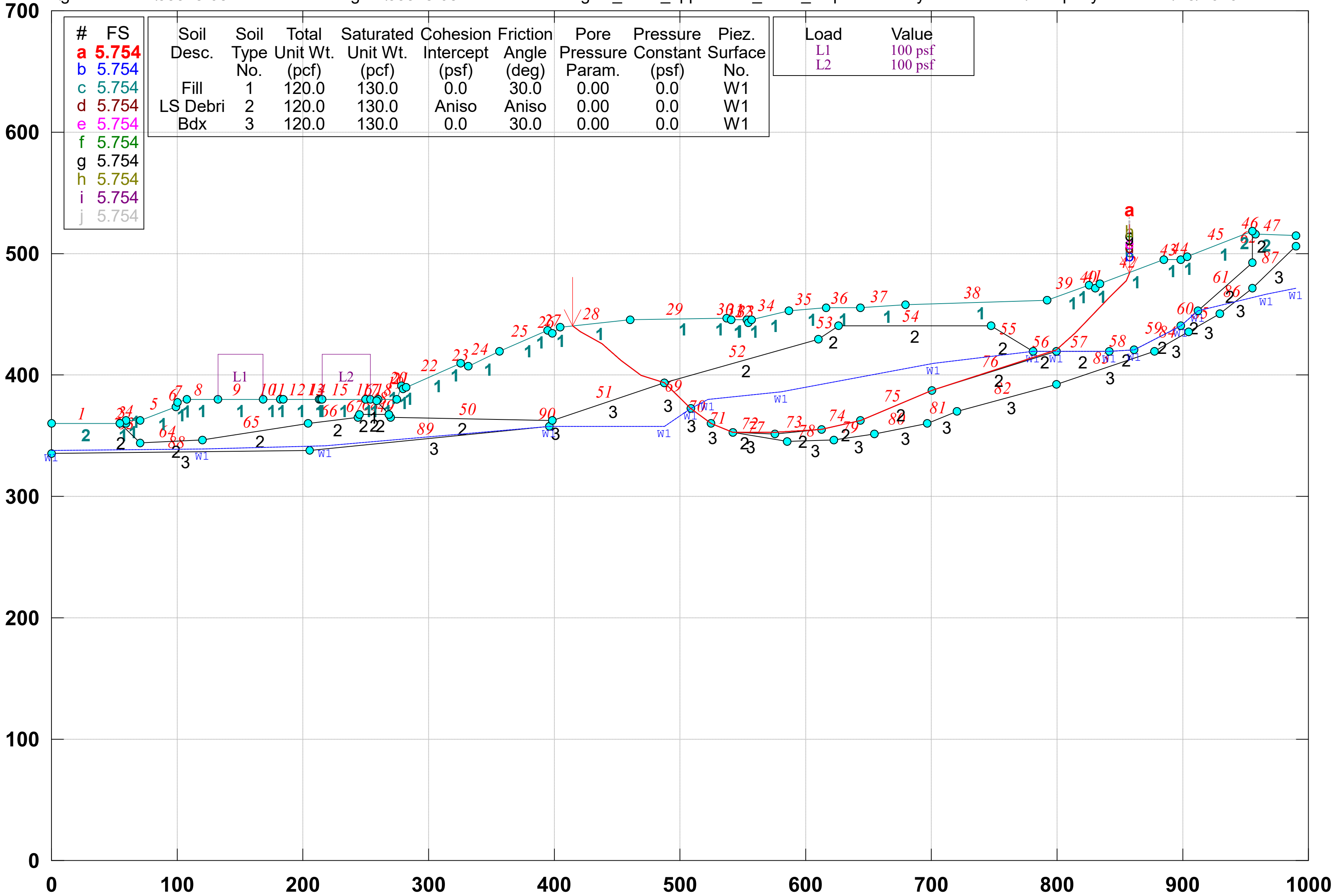
Circle Center At X = 150.370 ; Y = 811.000 ; and Radius = 468.101

*** FOS = 1.363 Theta (ki=1.0) = 7.89 ***
Lambda = 0.139

**** END OF GSTABL7 OUTPUT ****

33348-08 The Cove at El Niguel Section DR-DR'_Upper Slide_Static

u:\gstabl7 data\33348-08 the cove at el niguel\33348-08 the cove at el niguel_dr-dr'_upper slide_static_v1.pl2 Run By: Insert Name/company Here 12/23/2020 11:27AM



GSTABL7 v.2 FSmin=5.754

Safety Factors Are Calculated By The Simplified Janbu Method

*** 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 **

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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: 12/23/2020
Time of Run: 11:27AM
Run By: Insert Name/company Here
Input Data Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_upper slide_static_v1.in
Output Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_upper slide_static_v1.OUT
Unit System: English

Plotted Output Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_upper slide_static_v1.PLT

PROBLEM DESCRIPTION: 33348-08 The Cove at El Niguel
Section DR-DR'_Upper Slide_Static

BOUNDARY COORDINATES

47 Top Boundaries
90 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	360.00	55.00	360.00	2
2	55.00	360.00	59.00	360.00	1
3	59.00	360.00	59.01	362.00	1
4	59.01	362.00	70.00	362.00	1
5	70.00	362.00	99.00	373.50	1
6	99.00	373.50	100.00	377.00	1
7	100.00	377.00	108.00	379.95	1
8	108.00	379.95	132.00	379.95	1
9	132.00	379.95	168.00	379.95	1
10	168.00	379.95	182.00	379.95	1
11	182.00	379.95	184.00	379.50	1
12	184.00	379.50	213.00	379.50	1
13	213.00	379.50	214.00	380.19	1
14	214.00	380.19	215.00	380.19	1
15	215.00	380.19	250.00	380.19	1
16	250.00	380.19	254.00	380.19	1
17	254.00	380.19	260.00	380.19	1
18	260.00	380.19	275.00	380.19	1
19	275.00	380.19	278.00	390.44	1
20	278.00	390.44	280.00	389.00	1
21	280.00	389.00	282.00	390.00	1

22	282.00	390.00	326.00	410.00	1
23	326.00	410.00	332.00	407.00	1
24	332.00	407.00	357.00	420.00	1
25	357.00	420.00	395.00	437.00	1
26	395.00	437.00	399.00	434.00	1
27	399.00	434.00	405.00	439.00	1
28	405.00	439.00	460.00	445.00	1
29	460.00	445.00	537.00	447.00	1
30	537.00	447.00	541.00	445.00	1
31	541.00	445.00	553.00	446.00	1
32	553.00	446.00	555.00	443.00	1
33	555.00	443.00	557.00	446.00	1
34	557.00	446.00	586.00	453.00	1
35	586.00	453.00	616.00	456.00	1
36	616.00	456.00	644.00	456.00	1
37	644.00	456.00	680.00	458.00	1
38	680.00	458.00	792.00	462.00	1
39	792.00	462.00	825.00	474.00	1
40	825.00	474.00	830.00	471.00	1
41	830.00	471.00	834.00	475.00	1
42	834.00	475.00	885.00	495.00	1
43	885.00	495.00	899.00	495.00	1
44	899.00	495.00	903.00	498.00	1
45	903.00	498.00	955.00	519.00	1
46	955.00	519.00	958.00	516.00	2
47	958.00	516.00	990.00	515.00	2
48	259.00	379.00	268.00	368.00	1
49	268.00	368.00	270.00	365.00	2
50	270.00	365.00	398.00	363.00	2
51	398.00	363.00	488.00	394.00	3
52	488.00	394.00	610.00	430.00	2
53	610.00	430.00	626.00	440.00	2
54	626.00	440.00	748.00	440.00	2
55	748.00	440.00	781.00	420.00	2
56	781.00	420.00	800.00	420.00	2
57	800.00	420.00	841.00	419.00	2
58	841.00	419.00	861.00	421.00	2
59	861.00	421.00	898.00	440.00	2
60	898.00	440.00	912.00	453.00	2
61	912.00	453.00	954.99	493.00	2
62	954.99	493.00	955.00	519.00	2
63	55.00	360.00	70.00	344.00	2
64	70.00	344.00	120.00	346.00	2
65	120.00	346.00	204.00	360.00	2
66	204.00	360.00	244.00	365.00	2
67	244.00	365.00	245.00	368.00	2
68	245.00	368.00	268.00	368.00	2
69	488.00	394.00	508.00	372.00	3
70	508.00	372.00	525.00	360.00	3
71	525.00	360.00	542.00	353.00	3
72	542.00	353.00	576.00	352.00	2
73	576.00	352.00	613.00	355.00	2
74	613.00	355.00	644.00	362.00	2
75	644.00	362.00	700.00	387.00	2
76	700.00	387.00	800.00	420.00	2
77	542.00	353.00	585.00	345.00	3
78	585.00	345.00	622.00	347.00	3
79	622.00	347.00	655.00	351.00	3
80	655.00	351.00	697.00	360.00	3
81	697.00	360.00	720.00	370.00	3
82	720.00	370.00	800.00	392.00	3
83	800.00	392.00	877.00	420.00	3
84	877.00	420.00	905.00	435.00	3
85	905.00	435.00	929.00	450.00	3
86	929.00	450.00	955.00	471.00	3
87	955.00	471.00	990.00	506.00	3
88	0.00	335.00	205.00	338.00	3
89	205.00	338.00	396.00	358.00	3
90	396.00	358.00	398.00	363.00	3

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	130.0	0.0	30.0	0.00	0.0	1
2	120.0	130.0	0.0	13.0	0.00	0.0	1
3	120.0	130.0	0.0	30.0	0.00	0.0	1

ANISOTROPIC STRENGTH PARAMETERS
1 soil type(s)

Soil Type 2 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	0.0	0.00	30.00
2	30.0	0.00	13.00
3	90.0	0.00	30.00

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

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 17 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	338.00
2	120.00	339.00
3	218.00	342.00
4	396.00	358.00
5	488.00	358.00

6	508.00	372.00
7	522.00	380.00
8	580.00	386.00
9	702.00	409.00
10	781.00	420.00
11	800.00	420.00
12	841.00	419.00
13	861.00	421.00
14	898.00	440.00
15	912.00	453.00
16	966.00	466.00
17	990.00	471.00

1

BOUNDARY LOAD(S)

2 Load(s) Specified

Load No.	X-Left (ft)	X-Right (ft)	Intensity (psf)	Deflection (deg)
1	132.00	168.00	100.0	0.0
2	215.00	254.00	100.0	0.0

NOTE - Intensity Is Specified As A Uniformly Distributed
Force Acting On A Horizontally Projected Surface.

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

2000 Trial Surfaces Have Been Generated.

9 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 20.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	488.00	394.00	488.00	394.00	1.00
2	508.00	372.00	508.00	372.00	1.00
3	525.00	360.00	525.00	360.00	1.00
4	542.00	353.00	542.00	353.00	1.00
5	576.00	352.00	576.00	352.00	1.00
6	613.00	355.00	613.00	355.00	1.00
7	644.00	362.00	644.00	362.00	1.00
8	700.00	387.00	700.00	387.00	1.00
9	800.00	420.00	800.00	420.00	1.00

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Simplified Janbu Method * *

Total Number of Trial Surfaces Attempted = 2000

Number of Trial Surfaces With Valid FS = 2000

Statistical Data On All Valid FS Values:
FS Max = 9.585 FS Min = 5.754 FS Ave = 6.492
Standard Deviation = 0.692 Coefficient of Variation = 10.65 %

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	414.538	440.040
2	420.766	435.076
3	438.157	425.199
4	453.456	412.317
5	468.955	399.677
6	488.000	393.570
7	508.000	371.972
8	525.000	360.480
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10	576.000	352.436
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15	813.937	434.833
16	828.076	448.978
17	841.625	463.689
18	855.681	477.918
19	857.189	484.094

Factor of Safety
*** 5.754 ***

Individual data on the 45 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	6.2	2109.1	0.0	0.0	0.	0.	0.0	0.0	0.0
2	17.4	24064.2	0.0	0.0	0.	0.	0.0	0.0	0.0
3	15.3	45332.8	0.0	0.0	0.	0.	0.0	0.0	0.0
4	6.5	27481.2	0.0	0.0	0.	0.	0.0	0.0	0.0
5	9.0	44905.3	0.0	0.0	0.	0.	0.0	0.0	0.0
6	18.4	107622.2	0.0	0.0	0.	0.	0.0	0.0	0.0
7	0.6	4033.2	0.0	0.0	0.	0.	0.0	0.0	0.0
8	20.0	151579.1	0.0	0.0	0.	0.	0.0	0.0	0.0
9	0.0	138.4	0.0	0.0	0.	0.	0.0	0.0	0.0
10	0.9	8298.8	0.0	37.1	0.	0.	0.0	0.0	0.0
11	13.1	125813.4	0.0	8063.4	0.	0.	0.0	0.0	0.0
12	3.0	31212.9	0.0	4194.5	0.	0.	0.0	0.0	0.0
13	8.9	96148.6	0.0	13592.0	0.	0.	0.0	0.0	0.0
14	3.1	35015.5	0.0	5508.7	0.	0.	0.0	0.0	0.0
15	4.0	45286.0	0.0	7614.2	0.	0.	0.0	0.0	0.0
16	1.0	11360.4	0.0	2001.0	0.	0.	0.0	0.0	0.0
17	2.4	27454.7	0.0	4426.3	0.	0.	0.0	0.0	0.0
18	8.6	98599.5	0.0	16123.1	0.	0.	0.0	0.0	0.0
19	2.0	22688.5	0.0	3822.7	0.	0.	0.0	0.0	0.0
20	2.0	22694.6	0.0	3849.3	0.	0.	0.0	0.0	0.0

21	2.1	23806.2	0.0	3991.3	0.	0.	0.0	0.0	0.0
22	16.9	200741.6	0.0	33903.8	0.	0.	0.0	0.0	0.0
23	4.0	48597.1	0.0	8267.4	0.	0.	0.0	0.0	0.0
24	6.0	73509.0	0.0	12406.8	0.	0.	0.0	0.0	0.0
25	24.0	296914.7	0.0	52222.9	0.	0.	0.0	0.0	0.0
26	3.0	37300.7	0.0	6819.9	0.	0.	0.0	0.0	0.0
27	3.0	37249.7	0.0	7001.2	0.	0.	0.0	0.0	0.0
28	10.0	122528.7	0.0	23169.4	0.	0.	0.0	0.0	0.0
29	18.0	213507.7	0.0	41054.1	0.	0.	0.0	0.0	0.0
30	36.0	385823.1	0.0	75503.2	0.	0.	0.0	0.0	0.0
31	20.0	186098.3	0.0	32197.5	0.	0.	0.0	0.0	0.0
32	2.0	17496.3	0.0	2743.4	0.	0.	0.0	0.0	0.0
33	46.0	361104.8	0.0	49852.0	0.	0.	0.0	0.0	0.0
34	33.0	210162.3	0.0	19369.8	0.	0.	0.0	0.0	0.0
35	11.0	60898.0	0.0	2891.9	0.	0.	0.0	0.0	0.0
36	6.5	34763.9	0.0	465.9	0.	0.	0.0	0.0	0.0
37	1.5	7830.6	0.0	0.0	0.	0.	0.0	0.0	0.0
38	13.9	66533.4	0.0	0.0	0.	0.	0.0	0.0	0.0
39	11.1	41980.0	0.0	0.0	0.	0.	0.0	0.0	0.0
40	3.1	9463.6	0.0	0.0	0.	0.	0.0	0.0	0.0
41	1.9	4976.4	0.0	0.0	0.	0.	0.0	0.0	0.0
42	4.0	9485.6	0.0	0.0	0.	0.	0.0	0.0	0.0
43	7.6	15506.4	0.0	0.0	0.	0.	0.0	0.0	0.0
44	14.1	16770.3	0.0	0.0	0.	0.	0.0	0.0	0.0
45	1.5	505.4	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	414.538	440.040
2	420.766	435.076
3	438.157	425.199
4	453.456	412.317
5	468.955	399.677
6	488.000	393.570
7	508.000	371.972
8	525.000	360.480
9	542.000	352.561
10	576.000	352.436
11	613.000	355.072
12	644.000	362.189
13	700.000	387.255
14	800.000	420.488
15	813.937	434.833
16	828.076	448.978
17	841.625	463.689
18	855.681	477.918
19	857.189	484.094

Factor of Safety
*** 5.754 ***

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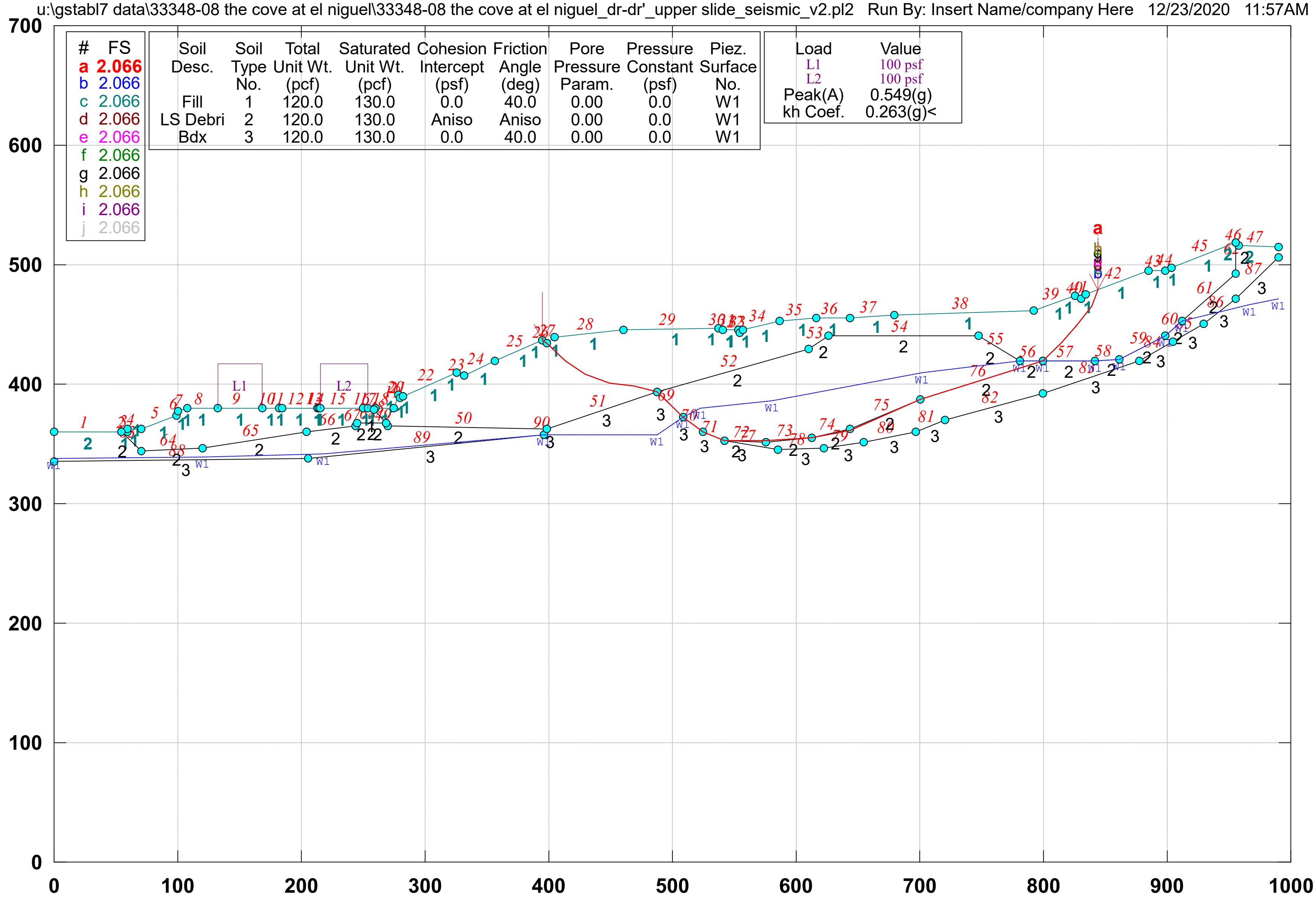
Factor of Safety
*** 5.754 ***

**** END OF GSTABL7 OUTPUT ****

33348-08 The Cove at El Niguel Section DR-DR'_Upper Slide_Seismic

u:\gstabl7 data\33348-08 the cove at el niguel\33348-08 the cove at el niguel_dr-dr'_upper slide_seismic_v2.pl2 Run By: Insert Name/company Here 12/23/2020 11:57AM



#	FS
a	2.066
b	2.066
c	2.066
d	2.066
e	2.066
f	2.066
g	2.066
h	2.066
i	2.066
j	2.066

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	120.0	130.0	0.0	40.0	0.00	0.0	W1
LS Debri	2	120.0	130.0	Aniso	Aniso	0.00	0.0	W1
Bdx	3	120.0	130.0	0.0	40.0	0.00	0.0	W1

Load	Value
L1	100 psf
L2	100 psf
Peak(A)	0.549(g)
kh Coef.	0.263(g)<

GSTABL7 v.2 FSmin=2.066

Safety Factors Are Calculated By The Simplified Janbu Method

*** 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 **

****

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: 12/23/2020
Time of Run: 11:57AM
Run By: Insert Name/company Here
Input Data Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_upper slide_seismic_v2.in
Output Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_upper slide_seismic_v2.OUT
Unit System: English

Plotted Output Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_upper slide_seismic_v2.PLT

PROBLEM DESCRIPTION: 33348-08 The Cove at El Niguel
Section DR-DR'_Upper Slide_Seismic

BOUNDARY COORDINATES

47 Top Boundaries
90 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	360.00	55.00	360.00	2
2	55.00	360.00	59.00	360.00	1
3	59.00	360.00	59.01	362.00	1
4	59.01	362.00	70.00	362.00	1
5	70.00	362.00	99.00	373.50	1
6	99.00	373.50	100.00	377.00	1
7	100.00	377.00	108.00	379.95	1
8	108.00	379.95	132.00	379.95	1
9	132.00	379.95	168.00	379.95	1
10	168.00	379.95	182.00	379.95	1
11	182.00	379.95	184.00	379.50	1
12	184.00	379.50	213.00	379.50	1
13	213.00	379.50	214.00	380.19	1
14	214.00	380.19	215.00	380.19	1
15	215.00	380.19	250.00	380.19	1
16	250.00	380.19	254.00	380.19	1
17	254.00	380.19	260.00	380.19	1
18	260.00	380.19	275.00	380.19	1
19	275.00	380.19	278.00	390.44	1
20	278.00	390.44	280.00	389.00	1
21	280.00	389.00	282.00	390.00	1

22	282.00	390.00	326.00	410.00	1
23	326.00	410.00	332.00	407.00	1
24	332.00	407.00	357.00	420.00	1
25	357.00	420.00	395.00	437.00	1
26	395.00	437.00	399.00	434.00	1
27	399.00	434.00	405.00	439.00	1
28	405.00	439.00	460.00	445.00	1
29	460.00	445.00	537.00	447.00	1
30	537.00	447.00	541.00	445.00	1
31	541.00	445.00	553.00	446.00	1
32	553.00	446.00	555.00	443.00	1
33	555.00	443.00	557.00	446.00	1
34	557.00	446.00	586.00	453.00	1
35	586.00	453.00	616.00	456.00	1
36	616.00	456.00	644.00	456.00	1
37	644.00	456.00	680.00	458.00	1
38	680.00	458.00	792.00	462.00	1
39	792.00	462.00	825.00	474.00	1
40	825.00	474.00	830.00	471.00	1
41	830.00	471.00	834.00	475.00	1
42	834.00	475.00	885.00	495.00	1
43	885.00	495.00	899.00	495.00	1
44	899.00	495.00	903.00	498.00	1
45	903.00	498.00	955.00	519.00	1
46	955.00	519.00	958.00	516.00	2
47	958.00	516.00	990.00	515.00	2
48	259.00	379.00	268.00	368.00	1
49	268.00	368.00	270.00	365.00	2
50	270.00	365.00	398.00	363.00	2
51	398.00	363.00	488.00	394.00	3
52	488.00	394.00	610.00	430.00	2
53	610.00	430.00	626.00	440.00	2
54	626.00	440.00	748.00	440.00	2
55	748.00	440.00	781.00	420.00	2
56	781.00	420.00	800.00	420.00	2
57	800.00	420.00	841.00	419.00	2
58	841.00	419.00	861.00	421.00	2
59	861.00	421.00	898.00	440.00	2
60	898.00	440.00	912.00	453.00	2
61	912.00	453.00	954.99	493.00	2
62	954.99	493.00	955.00	519.00	2
63	55.00	360.00	70.00	344.00	2
64	70.00	344.00	120.00	346.00	2
65	120.00	346.00	204.00	360.00	2
66	204.00	360.00	244.00	365.00	2
67	244.00	365.00	245.00	368.00	2
68	245.00	368.00	268.00	368.00	2
69	488.00	394.00	508.00	372.00	3
70	508.00	372.00	525.00	360.00	3
71	525.00	360.00	542.00	353.00	3
72	542.00	353.00	576.00	352.00	2
73	576.00	352.00	613.00	355.00	2
74	613.00	355.00	644.00	362.00	2
75	644.00	362.00	700.00	387.00	2
76	700.00	387.00	800.00	420.00	2
77	542.00	353.00	585.00	345.00	3
78	585.00	345.00	622.00	347.00	3
79	622.00	347.00	655.00	351.00	3
80	655.00	351.00	697.00	360.00	3
81	697.00	360.00	720.00	370.00	3
82	720.00	370.00	800.00	392.00	3
83	800.00	392.00	877.00	420.00	3
84	877.00	420.00	905.00	435.00	3
85	905.00	435.00	929.00	450.00	3
86	929.00	450.00	955.00	471.00	3
87	955.00	471.00	990.00	506.00	3
88	0.00	335.00	205.00	338.00	3
89	205.00	338.00	396.00	358.00	3
90	396.00	358.00	398.00	363.00	3

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	130.0	0.0	40.0	0.00	0.0	1
2	120.0	130.0	0.0	17.3	0.00	0.0	1
3	120.0	130.0	0.0	40.0	0.00	0.0	1

ANISOTROPIC STRENGTH PARAMETERS
1 soil type(s)

Soil Type 2 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	0.0	0.00	40.00
2	30.0	0.00	17.33
3	90.0	0.00	40.00

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

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 17 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	338.00
2	120.00	339.00
3	218.00	342.00
4	396.00	358.00
5	488.00	358.00

6	508.00	372.00
7	522.00	380.00
8	580.00	386.00
9	702.00	409.00
10	781.00	420.00
11	800.00	420.00
12	841.00	419.00
13	861.00	421.00
14	898.00	440.00
15	912.00	453.00
16	966.00	466.00
17	990.00	471.00

1

BOUNDARY LOAD(S)

2 Load(s) Specified

Load No.	X-Left (ft)	X-Right (ft)	Intensity (psf)	Deflection (deg)
1	132.00	168.00	100.0	0.0
2	215.00	254.00	100.0	0.0

NOTE - Intensity Is Specified As A Uniformly Distributed Force Acting On A Horizontally Projected Surface.

Specified Peak Ground Acceleration Coefficient (A) = 0.549(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.263(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)

Specified Seismic Pore-Pressure Factor = 0.000

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

2000 Trial Surfaces Have Been Generated.

9 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 20.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	488.00	394.00	488.00	394.00	1.00
2	508.00	372.00	508.00	372.00	1.00
3	525.00	360.00	525.00	360.00	1.00
4	542.00	353.00	542.00	353.00	1.00
5	576.00	352.00	576.00	352.00	1.00
6	613.00	355.00	613.00	355.00	1.00
7	644.00	362.00	644.00	362.00	1.00
8	700.00	387.00	700.00	387.00	1.00
9	800.00	420.00	800.00	420.00	1.00

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Simplified Janbu Method * *

Total Number of Trial Surfaces Attempted = 2000

Number of Trial Surfaces With Valid FS = 2000

Statistical Data On All Valid FS Values:
FS Max = 2.327 FS Min = 2.066 FS Ave = 2.156
Standard Deviation = 0.050 Coefficient of Variation = 2.31 %

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	394.722	436.876
2	399.254	433.293
3	413.892	419.664
4	430.000	407.810
5	448.789	400.957
6	468.662	398.701
7	488.000	393.600
8	508.000	372.179
9	525.000	359.599
10	542.000	352.702
11	576.000	352.082
12	613.000	355.083
13	644.000	361.800
14	700.000	387.075
15	800.000	419.502
16	813.722	434.052
17	826.360	449.553
18	838.583	465.383
19	844.283	479.032

Factor of Safety
*** 2.066 ***

Individual data on the 51 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	0.3	5.7	0.0	0.0	0.	0.	1.5	0.0	0.0
2	4.0	204.1	0.0	0.0	0.	0.	53.7	0.0	0.0
3	0.3	21.7	0.0	0.0	0.	0.	5.7	0.0	0.0
4	5.7	4128.5	0.0	0.0	0.	0.	1085.8	0.0	0.0
5	8.9	16732.0	0.0	0.0	0.	0.	4400.5	0.0	0.0
6	16.1	52406.5	0.0	0.0	0.	0.	13782.9	0.0	0.0
7	18.8	86511.6	0.0	0.0	0.	0.	22752.6	0.0	0.0
8	11.2	59284.1	0.0	0.0	0.	0.	15591.7	0.0	0.0
9	8.7	47728.4	0.0	0.0	0.	0.	12552.6	0.0	0.0
10	18.7	110355.7	0.0	0.0	0.	0.	29023.5	0.0	0.0
11	0.7	4110.9	0.0	0.0	0.	0.	1081.2	0.0	0.0
12	13.8	99007.2	0.0	0.0	0.	0.	26038.9	0.0	0.0
13	6.2	52427.2	0.0	0.0	0.	0.	13788.4	0.0	0.0
14	0.1	1214.9	0.0	0.0	0.	0.	319.5	0.0	0.0

15	5.1	46822.2	0.0	1165.1	0.	0.	12314.2	0.0	0.0
16	8.8	86509.9	0.0	7414.1	0.	0.	22752.1	0.0	0.0
17	3.0	31519.0	0.0	4504.8	0.	0.	8289.5	0.0	0.0
18	12.0	131975.8	0.0	19103.7	0.	0.	34709.6	0.0	0.0
19	4.0	45306.7	0.0	7459.2	0.	0.	11915.7	0.0	0.0
20	1.0	11346.1	0.0	1950.1	0.	0.	2984.0	0.0	0.0
21	1.8	20252.0	0.0	3253.6	0.	0.	5326.3	0.0	0.0
22	9.2	105715.7	0.0	17257.8	0.	0.	27803.2	0.0	0.0
23	2.0	22697.4	0.0	3827.5	0.	0.	5969.4	0.0	0.0
24	2.0	22711.0	0.0	3857.7	0.	0.	5973.0	0.0	0.0
25	11.7	136898.0	0.0	23118.5	0.	0.	36004.2	0.0	0.0
26	7.3	88182.3	0.0	15037.0	0.	0.	23192.0	0.0	0.0
27	4.0	48770.7	0.0	8356.8	0.	0.	12826.7	0.0	0.0
28	6.0	73730.8	0.0	12521.1	0.	0.	19391.2	0.0	0.0
29	24.0	297340.2	0.0	52463.7	0.	0.	78200.5	0.0	0.0
30	3.0	37302.0	0.0	6825.6	0.	0.	9810.4	0.0	0.0
31	3.0	37252.7	0.0	6983.5	0.	0.	9797.5	0.0	0.0
32	6.1	75338.8	0.0	14190.7	0.	0.	19814.1	0.0	0.0
33	3.9	47309.1	0.0	8972.9	0.	0.	12442.3	0.0	0.0
34	18.0	214144.9	0.0	41249.6	0.	0.	56320.1	0.0	0.0
35	36.0	387326.9	0.0	76388.0	0.	0.	*******	0.0	0.0
36	4.8	46654.5	0.0	8443.1	0.	0.	12270.1	0.0	0.0
37	15.2	140007.9	0.0	24091.8	0.	0.	36822.1	0.0	0.0
38	2.0	17545.2	0.0	2761.1	0.	0.	4614.4	0.0	0.0
39	11.0	94188.6	0.0	14520.7	0.	0.	24771.6	0.0	0.0
40	35.0	269196.4	0.0	36352.7	0.	0.	70798.6	0.0	0.0
41	33.0	213163.9	0.0	20824.8	0.	0.	56062.1	0.0	0.0
42	11.0	62153.1	0.0	3518.3	0.	0.	16346.3	0.0	0.0
43	8.0	43583.4	0.0	942.1	0.	0.	11462.4	0.0	0.0
44	0.5	2494.6	0.0	10.4	0.	0.	656.1	0.0	0.0
45	13.3	64405.6	0.0	0.0	0.	0.	16938.7	0.0	0.0
46	11.3	41928.1	0.0	0.0	0.	0.	11027.1	0.0	0.0
47	1.4	4059.6	0.0	0.0	0.	0.	1067.7	0.0	0.0
48	3.6	8815.3	0.0	0.0	0.	0.	2318.4	0.0	0.0
49	4.0	7748.6	0.0	0.0	0.	0.	2037.9	0.0	0.0
50	4.6	7415.8	0.0	0.0	0.	0.	1950.3	0.0	0.0
51	5.7	3903.4	0.0	0.0	0.	0.	1026.6	0.0	0.0

Failure Surface Specified By 19 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	394.722	436.876
2	399.254	433.293
3	413.892	419.664
4	430.000	407.810
5	448.789	400.957
6	468.662	398.701
7	488.000	393.600
8	508.000	372.179
9	525.000	359.599
10	542.000	352.702
11	576.000	352.082
12	613.000	355.083
13	644.000	361.800
14	700.000	387.075
15	800.000	419.502
16	813.722	434.052
17	826.360	449.553
18	838.583	465.383
19	844.283	479.032

Factor of Safety
*** 2.066 ***

1

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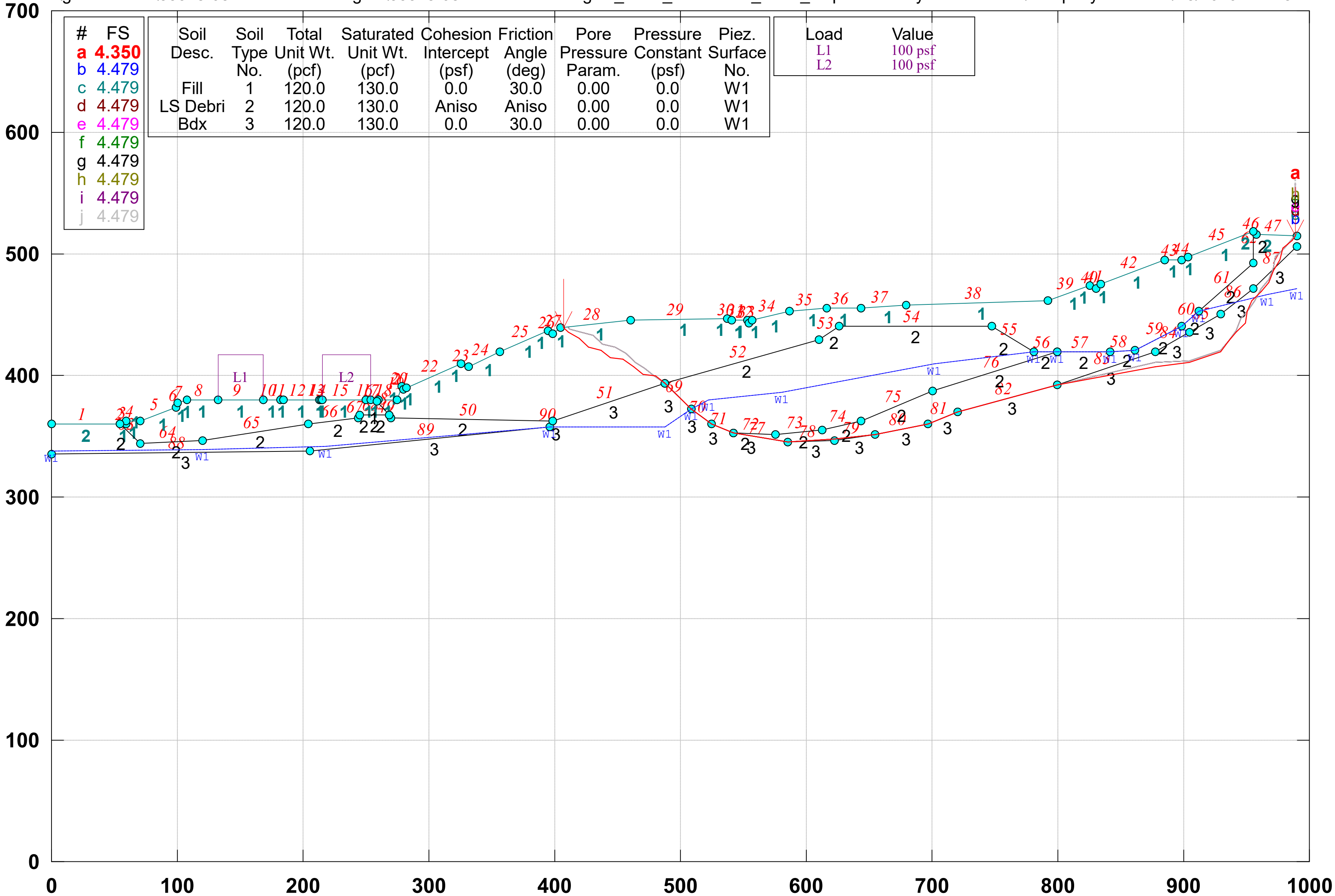
15	800.000	419.502
16	813.722	434.052
17	826.360	449.553
18	838.583	465.383
19	844.283	479.032

Factor of Safety
*** 2.066 ***

**** END OF GSTABL7 OUTPUT ****

33348-08 The Cove at El Niguel Section DR-DR'_Lower Slide_Static

u:\gstabl7 data\33348-08 the cove at el niguel\33348-08 the cove at el niguel_dr-dr'_lower slide_static_v1.pl2 Run By: Insert Name/company Here 12/23/2020 11:25AM



GSTABL7 v.2 FSmin=4.350

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

*** 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 **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*********************************************************************************
                    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: 12/23/2020
Time of Run: 11:25AM
Run By: Insert Name/company Here
Input Data Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_lower slide_static_v1.in
Output Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_lower slide_static_v1.OUT
Unit System: English

Plotted Output Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_lower slide_static_v1.PLT

PROBLEM DESCRIPTION: 33348-08 The Cove at El Niguel
Section DR-DR'_Lower Slide_Static

BOUNDARY COORDINATES

47 Top Boundaries
90 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	360.00	55.00	360.00	2
2	55.00	360.00	59.00	360.00	1
3	59.00	360.00	59.01	362.00	1
4	59.01	362.00	70.00	362.00	1
5	70.00	362.00	99.00	373.50	1
6	99.00	373.50	100.00	377.00	1
7	100.00	377.00	108.00	379.95	1
8	108.00	379.95	132.00	379.95	1
9	132.00	379.95	168.00	379.95	1
10	168.00	379.95	182.00	379.95	1
11	182.00	379.95	184.00	379.50	1
12	184.00	379.50	213.00	379.50	1
13	213.00	379.50	214.00	380.19	1
14	214.00	380.19	215.00	380.19	1
15	215.00	380.19	250.00	380.19	1
16	250.00	380.19	254.00	380.19	1
17	254.00	380.19	260.00	380.19	1
18	260.00	380.19	275.00	380.19	1
19	275.00	380.19	278.00	390.44	1
20	278.00	390.44	280.00	389.00	1
21	280.00	389.00	282.00	390.00	1

22	282.00	390.00	326.00	410.00	1
23	326.00	410.00	332.00	407.00	1
24	332.00	407.00	357.00	420.00	1
25	357.00	420.00	395.00	437.00	1
26	395.00	437.00	399.00	434.00	1
27	399.00	434.00	405.00	439.00	1
28	405.00	439.00	460.00	445.00	1
29	460.00	445.00	537.00	447.00	1
30	537.00	447.00	541.00	445.00	1
31	541.00	445.00	553.00	446.00	1
32	553.00	446.00	555.00	443.00	1
33	555.00	443.00	557.00	446.00	1
34	557.00	446.00	586.00	453.00	1
35	586.00	453.00	616.00	456.00	1
36	616.00	456.00	644.00	456.00	1
37	644.00	456.00	680.00	458.00	1
38	680.00	458.00	792.00	462.00	1
39	792.00	462.00	825.00	474.00	1
40	825.00	474.00	830.00	471.00	1
41	830.00	471.00	834.00	475.00	1
42	834.00	475.00	885.00	495.00	1
43	885.00	495.00	899.00	495.00	1
44	899.00	495.00	903.00	498.00	1
45	903.00	498.00	955.00	519.00	1
46	955.00	519.00	958.00	516.00	2
47	958.00	516.00	990.00	515.00	2
48	259.00	379.00	268.00	368.00	1
49	268.00	368.00	270.00	365.00	2
50	270.00	365.00	398.00	363.00	2
51	398.00	363.00	488.00	394.00	3
52	488.00	394.00	610.00	430.00	2
53	610.00	430.00	626.00	440.00	2
54	626.00	440.00	748.00	440.00	2
55	748.00	440.00	781.00	420.00	2
56	781.00	420.00	800.00	420.00	2
57	800.00	420.00	841.00	419.00	2
58	841.00	419.00	861.00	421.00	2
59	861.00	421.00	898.00	440.00	2
60	898.00	440.00	912.00	453.00	2
61	912.00	453.00	954.99	493.00	2
62	954.99	493.00	955.00	519.00	2
63	55.00	360.00	70.00	344.00	2
64	70.00	344.00	120.00	346.00	2
65	120.00	346.00	204.00	360.00	2
66	204.00	360.00	244.00	365.00	2
67	244.00	365.00	245.00	368.00	2
68	245.00	368.00	268.00	368.00	2
69	488.00	394.00	508.00	372.00	3
70	508.00	372.00	525.00	360.00	3
71	525.00	360.00	542.00	353.00	3
72	542.00	353.00	576.00	352.00	2
73	576.00	352.00	613.00	355.00	2
74	613.00	355.00	644.00	362.00	2
75	644.00	362.00	700.00	387.00	2
76	700.00	387.00	800.00	420.00	2
77	542.00	353.00	585.00	345.00	3
78	585.00	345.00	622.00	347.00	3
79	622.00	347.00	655.00	351.00	3
80	655.00	351.00	697.00	360.00	3
81	697.00	360.00	720.00	370.00	3
82	720.00	370.00	800.00	392.00	3
83	800.00	392.00	877.00	420.00	3
84	877.00	420.00	905.00	435.00	3
85	905.00	435.00	929.00	450.00	3
86	929.00	450.00	955.00	471.00	3
87	955.00	471.00	990.00	506.00	3
88	0.00	335.00	205.00	338.00	3
89	205.00	338.00	396.00	358.00	3
90	396.00	358.00	398.00	363.00	3

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	130.0	0.0	30.0	0.00	0.0	1
2	120.0	130.0	0.0	13.0	0.00	0.0	1
3	120.0	130.0	0.0	30.0	0.00	0.0	1

ANISOTROPIC STRENGTH PARAMETERS
1 soil type(s)

Soil Type 2 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	0.0	0.00	30.00
2	30.0	0.00	13.00
3	90.0	0.00	30.00

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

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 17 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	338.00
2	120.00	339.00
3	218.00	342.00
4	396.00	358.00
5	488.00	358.00

6	508.00	372.00
7	522.00	380.00
8	580.00	386.00
9	702.00	409.00
10	781.00	420.00
11	800.00	420.00
12	841.00	419.00
13	861.00	421.00
14	898.00	440.00
15	912.00	453.00
16	966.00	466.00
17	990.00	471.00

1

BOUNDARY LOAD(S)

2 Load(s) Specified

Load No.	X-Left (ft)	X-Right (ft)	Intensity (psf)	Deflection (deg)
1	132.00	168.00	100.0	0.0
2	215.00	254.00	100.0	0.0

NOTE - Intensity Is Specified As A Uniformly Distributed Force Acting On A Horizontally Projected Surface.

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

1000 Trial Surfaces Have Been Generated.

13 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 10.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	488.00	394.00	488.00	394.00	1.00
2	508.00	372.00	508.00	372.00	1.00
3	525.00	360.00	525.00	360.00	1.00
4	542.00	353.00	542.00	353.00	1.00
5	585.00	345.00	585.00	345.00	1.00
6	622.00	347.00	622.00	347.00	1.00
7	655.00	351.00	655.00	351.00	1.00
8	697.00	360.00	697.00	360.00	1.00
9	720.00	370.00	720.00	370.00	1.00
10	800.00	392.00	800.00	392.00	1.00
11	877.00	420.00	877.00	420.00	1.00
12	905.00	435.00	905.00	435.00	1.00
13	929.00	450.00	929.00	450.00	1.00

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 (0-1) * *

Selected ki function = Constant (1.0)

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 = 1000

WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.

Number of Trial Surfaces with Non-Converged FS = 460

Number of Trial Surfaces with Misleading FS = 122

Number of Trial Surfaces With Valid FS = 418

Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Attempted = 58.2 %

Statistical Data On All Valid FS Values:
FS Max = 37.024 FS Min = 4.350 FS Ave = 19.363
Standard Deviation = 10.829 Coefficient of Variation = 55.92 %

((Simplified Janbu FS for Critical Surface = 3.527))

Failure Surface Specified By 35 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	406.928	439.210
2	411.460	435.486
3	420.065	430.391
4	427.548	423.758
5	437.243	421.306
6	444.768	414.720
7	454.684	413.427
8	462.241	406.878
9	469.978	400.543
10	479.963	399.985
11	488.000	394.035
12	508.000	371.923
13	525.000	360.245
14	542.000	352.594
15	585.000	345.314
16	622.000	347.499
17	655.000	350.993
18	697.000	360.389
19	720.000	370.031
20	800.000	392.013
21	877.000	406.928
22	905.000	411.460
23	929.000	420.065
24	935.376	427.768

25	941.685	435.528
26	948.649	442.704
27	950.158	452.590
28	954.435	461.629
29	960.967	469.201
30	967.176	477.040
31	971.287	486.155
32	976.684	494.574
33	978.717	504.365
34	985.774	511.450
35	988.404	515.050

*** FOS = 4.350 Theta (ki=1.0) = 5.35 ***
Lambda = 0.094

Individual data on the 80 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	4.5	1147.2	0.0	0.0	0.	0.	0.0	0.0	0.0
2	8.6	7471.1	0.0	0.0	0.	0.	0.0	0.0	0.0
3	7.5	12551.4	0.0	0.0	0.	0.	0.0	0.0	0.0
4	9.7	22634.7	0.0	0.0	0.	0.	0.0	0.0	0.0
5	7.5	22499.0	0.0	0.0	0.	0.	0.0	0.0	0.0
6	9.9	35466.7	0.0	0.0	0.	0.	0.0	0.0	0.0
7	5.3	21424.0	0.0	0.0	0.	0.	0.0	0.0	0.0
8	2.2	9998.1	0.0	0.0	0.	0.	0.0	0.0	0.0
9	7.7	38485.1	0.0	0.0	0.	0.	0.0	0.0	0.0
10	10.0	54065.9	0.0	0.0	0.	0.	0.0	0.0	0.0
11	8.0	46885.4	0.0	0.0	0.	0.	0.0	0.0	0.0
12	0.0	153.4	0.0	0.0	0.	0.	0.0	0.0	0.0
13	6.2	40844.9	0.0	0.0	0.	0.	0.0	0.0	0.0
14	13.8	109840.4	0.0	0.0	0.	0.	0.0	0.0	0.0
15	0.0	381.2	0.0	0.1	0.	0.	0.0	0.0	0.0
16	4.1	37187.3	0.0	715.9	0.	0.	0.0	0.0	0.0
17	9.9	97151.9	0.0	7542.8	0.	0.	0.0	0.0	0.0
18	3.0	31298.2	0.0	4265.3	0.	0.	0.0	0.0	0.0
19	6.4	68814.5	0.0	9493.0	0.	0.	0.0	0.0	0.0
20	5.6	62568.9	0.0	9609.1	0.	0.	0.0	0.0	0.0
21	4.0	45293.7	0.0	7573.0	0.	0.	0.0	0.0	0.0
22	1.0	11357.2	0.0	1987.4	0.	0.	0.0	0.0	0.0
23	11.0	127309.6	0.0	21449.9	0.	0.	0.0	0.0	0.0
24	2.0	23196.6	0.0	4123.2	0.	0.	0.0	0.0	0.0
25	2.0	23288.7	0.0	4191.9	0.	0.	0.0	0.0	0.0
26	9.3	111918.0	0.0	20300.2	0.	0.	0.0	0.0	0.0
27	9.7	122567.2	0.0	22948.0	0.	0.	0.0	0.0	0.0
28	4.0	51757.0	0.0	9894.7	0.	0.	0.0	0.0	0.0
29	5.0	65852.6	0.0	12668.7	0.	0.	0.0	0.0	0.0
30	1.0	13314.1	0.0	2561.5	0.	0.	0.0	0.0	0.0
31	24.0	321597.4	0.0	63861.7	0.	0.	0.0	0.0	0.0
32	3.0	40450.2	0.0	8304.9	0.	0.	0.0	0.0	0.0
33	3.0	40505.9	0.0	8376.4	0.	0.	0.0	0.0	0.0
34	6.0	80962.7	0.0	16967.6	0.	0.	0.0	0.0	0.0
35	4.0	53810.3	0.0	11491.8	0.	0.	0.0	0.0	0.0
36	18.0	239791.6	0.0	52722.4	0.	0.	0.0	0.0	0.0
37	10.5	138712.4	0.0	31574.0	0.	0.	0.0	0.0	0.0
38	0.5	6333.5	0.0	1458.3	0.	0.	0.0	0.0	0.0
39	0.8	10299.3	0.0	2418.5	0.	0.	0.0	0.0	0.0
40	24.2	312218.0	0.0	74106.2	0.	0.	0.0	0.0	0.0
41	17.0	211640.3	0.0	51247.3	0.	0.	0.0	0.0	0.0
42	3.0	36546.9	0.0	9440.9	0.	0.	0.0	0.0	0.0
43	2.0	24123.2	0.0	6217.2	0.	0.	0.0	0.0	0.0
44	18.0	208398.6	0.0	53072.8	0.	0.	0.0	0.0	0.0

45	28.0	300107.9	0.0	71032.0	0.	0.	0.0	0.0	0.0
46	33.0	323473.8	0.0	74972.5	0.	0.	0.0	0.0	0.0
47	11.0	100507.1	0.0	22563.1	0.	0.	0.0	0.0	0.0
48	8.0	71966.3	0.0	15058.1	0.	0.	0.0	0.0	0.0
49	0.1	671.2	0.0	132.2	0.	0.	0.0	0.0	0.0
50	24.9	230704.1	0.0	39995.8	0.	0.	0.0	0.0	0.0
51	5.0	46195.5	0.0	6986.3	0.	0.	0.0	0.0	0.0
52	4.0	36738.7	0.0	5339.6	0.	0.	0.0	0.0	0.0
53	7.0	66146.9	0.0	8810.7	0.	0.	0.0	0.0	0.0
54	20.0	195064.2	0.0	22905.5	0.	0.	0.0	0.0	0.0
55	16.0	162854.4	0.0	17971.1	0.	0.	0.0	0.0	0.0
56	8.0	84119.7	0.0	10732.3	0.	0.	0.0	0.0	0.0
57	13.0	136920.1	0.0	20157.6	0.	0.	0.0	0.0	0.0
58	1.0	10382.0	0.0	1460.1	0.	0.	0.0	0.0	0.0
59	4.0	42112.8	0.0	6212.8	0.	0.	0.0	0.0	0.0
60	2.0	21432.1	0.0	3329.9	0.	0.	0.0	0.0	0.0
61	7.0	75497.5	0.0	13206.7	0.	0.	0.0	0.0	0.0
62	17.0	185909.6	0.0	41681.7	0.	0.	0.0	0.0	0.0
63	6.4	67811.5	0.0	20600.6	0.	0.	0.0	0.0	0.0
64	6.1	61111.6	0.0	16420.0	0.	0.	0.0	0.0	0.0
65	0.2	1679.9	0.0	415.0	0.	0.	0.0	0.0	0.0
66	7.0	64919.5	0.0	13272.3	0.	0.	0.0	0.0	0.0
67	1.5	12725.4	0.0	8713.8	0.	0.	0.0	0.0	0.0
68	4.3	31434.6	0.0	3393.6	0.	0.	0.0	0.0	0.0
69	0.6	3801.4	0.0	68.7	0.	0.	0.0	0.0	0.0
70	0.0	67.8	0.0	1.0	0.	0.	0.0	0.0	0.0
71	1.2	7742.7	0.0	57.7	0.	0.	0.0	0.0	0.0
72	1.8	11515.1	0.0	0.0	0.	0.	0.0	0.0	0.0
73	3.0	17256.9	0.0	0.0	0.	0.	0.0	0.0	0.0
74	6.2	31807.9	0.0	0.0	0.	0.	0.0	0.0	0.0
75	4.1	16798.7	0.0	0.0	0.	0.	0.0	0.0	0.0
76	2.0	6746.5	0.0	0.0	0.	0.	0.0	0.0	0.0
77	3.4	9531.7	0.0	0.0	0.	0.	0.0	0.0	0.0
78	2.0	3883.6	0.0	0.0	0.	0.	0.0	0.0	0.0
79	7.1	6211.1	0.0	0.0	0.	0.	0.0	0.0	0.0
80	2.6	581.0	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 35 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	410.858	439.639
2	411.941	438.615
3	421.319	435.143
4	431.126	433.184
5	438.228	426.145
6	447.843	423.396
7	456.575	418.523
8	463.653	411.458
9	471.759	405.602
10	479.440	399.200
11	488.000	394.029
12	508.000	372.416
13	525.000	360.282
14	542.000	353.098
15	585.000	345.082
16	622.000	347.203
17	655.000	351.179
18	697.000	360.479
19	720.000	370.322
20	800.000	391.677
21	877.000	410.858
22	905.000	411.941
23	929.000	421.319
24	935.882	428.575
25	941.480	436.861
26	944.877	446.266
27	951.602	453.667
28	956.980	462.098

29	961.192	471.167
30	967.492	478.934
31	971.118	488.253
32	973.236	498.026
33	980.197	505.205
34	986.601	512.886
35	988.329	515.052

*** FOS = 4.479 Theta (ki=1.0) = 5.44 ***
Lambda = 0.095

1

Failure Surface Specified By 35 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	410.858	439.639
2	411.941	438.615
3	421.319	435.143
4	431.126	433.184
5	438.228	426.145
6	447.843	423.396
7	456.575	418.523
8	463.653	411.458
9	471.759	405.602
10	479.440	399.200
11	488.000	394.029
12	508.000	372.416
13	525.000	360.282
14	542.000	353.098
15	585.000	345.082
16	622.000	347.203
17	655.000	351.179
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35	988.329	515.052


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***  FOS =      4.479   Theta (ki=1.0) =      5.44  ***
                    Lambda =   0.095
```
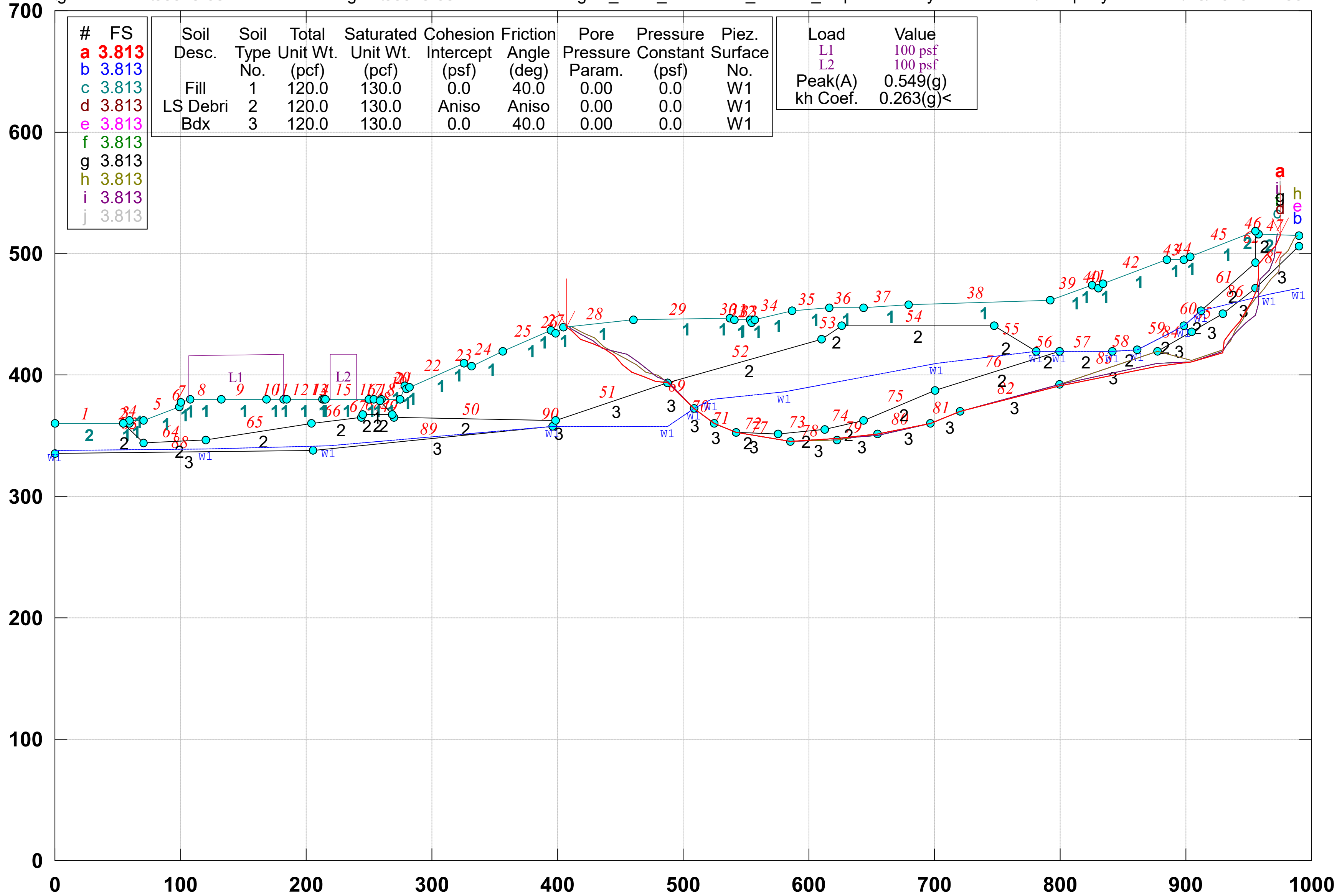


**** END OF GSTABL7 OUTPUT ****

33348-08 The Cove at El Niguel Section DR-DR'_Lower Slide_Seismic

u:\gstabl7 data\33348-08 the cove at el niguel\33348-08 the cove at el niguel_dr-dr'_lower slide_seismic_v2.pl2 Run By: Insert Name/company Here 12/23/2020 11:56AM



GSTABL7 v.2 FSmin=3.813

Safety Factors Are Calculated By GLE (Spencer`s) Method (0-1)

*** 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 **

****
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: 12/23/2020
Time of Run: 11:56AM
Run By: Insert Name/company Here
Input Data Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_lower slide_seismic_v2.in
Output Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_lower slide_seismic_v2.OUT
Unit System: English

Plotted Output Filename: U:\GStabl7 Data\33348-08 The Cove at El Niguel\33348-08 the cove at el niguel_dr-dr'_lower slide_seismic_v2.PLT

PROBLEM DESCRIPTION: 33348-08 The Cove at El Niguel
Section DR-DR'_Lower Slide_Seismic

BOUNDARY COORDINATES

47 Top Boundaries
90 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	0.00	360.00	55.00	360.00	2
2	55.00	360.00	59.00	360.00	1
3	59.00	360.00	59.01	362.00	1
4	59.01	362.00	70.00	362.00	1
5	70.00	362.00	99.00	373.50	1
6	99.00	373.50	100.00	377.00	1
7	100.00	377.00	108.00	379.95	1
8	108.00	379.95	132.00	379.95	1
9	132.00	379.95	168.00	379.95	1
10	168.00	379.95	182.00	379.95	1
11	182.00	379.95	184.00	379.50	1
12	184.00	379.50	213.00	379.50	1
13	213.00	379.50	214.00	380.19	1
14	214.00	380.19	215.00	380.19	1
15	215.00	380.19	250.00	380.19	1
16	250.00	380.19	254.00	380.19	1
17	254.00	380.19	260.00	380.19	1
18	260.00	380.19	275.00	380.19	1
19	275.00	380.19	278.00	390.44	1
20	278.00	390.44	280.00	389.00	1
21	280.00	389.00	282.00	390.00	1

22	282.00	390.00	326.00	410.00	1
23	326.00	410.00	332.00	407.00	1
24	332.00	407.00	357.00	420.00	1
25	357.00	420.00	395.00	437.00	1
26	395.00	437.00	399.00	434.00	1
27	399.00	434.00	405.00	439.00	1
28	405.00	439.00	460.00	445.00	1
29	460.00	445.00	537.00	447.00	1
30	537.00	447.00	541.00	445.00	1
31	541.00	445.00	553.00	446.00	1
32	553.00	446.00	555.00	443.00	1
33	555.00	443.00	557.00	446.00	1
34	557.00	446.00	586.00	453.00	1
35	586.00	453.00	616.00	456.00	1
36	616.00	456.00	644.00	456.00	1
37	644.00	456.00	680.00	458.00	1
38	680.00	458.00	792.00	462.00	1
39	792.00	462.00	825.00	474.00	1
40	825.00	474.00	830.00	471.00	1
41	830.00	471.00	834.00	475.00	1
42	834.00	475.00	885.00	495.00	1
43	885.00	495.00	899.00	495.00	1
44	899.00	495.00	903.00	498.00	1
45	903.00	498.00	955.00	519.00	1
46	955.00	519.00	958.00	516.00	2
47	958.00	516.00	990.00	515.00	2
48	259.00	379.00	268.00	368.00	1
49	268.00	368.00	270.00	365.00	2
50	270.00	365.00	398.00	363.00	2
51	398.00	363.00	488.00	394.00	3
52	488.00	394.00	610.00	430.00	2
53	610.00	430.00	626.00	440.00	2
54	626.00	440.00	748.00	440.00	2
55	748.00	440.00	781.00	420.00	2
56	781.00	420.00	800.00	420.00	2
57	800.00	420.00	841.00	419.00	2
58	841.00	419.00	861.00	421.00	2
59	861.00	421.00	898.00	440.00	2
60	898.00	440.00	912.00	453.00	2
61	912.00	453.00	954.99	493.00	2
62	954.99	493.00	955.00	519.00	2
63	55.00	360.00	70.00	344.00	2
64	70.00	344.00	120.00	346.00	2
65	120.00	346.00	204.00	360.00	2
66	204.00	360.00	244.00	365.00	2
67	244.00	365.00	245.00	368.00	2
68	245.00	368.00	268.00	368.00	2
69	488.00	394.00	508.00	372.00	3
70	508.00	372.00	525.00	360.00	3
71	525.00	360.00	542.00	353.00	3
72	542.00	353.00	576.00	352.00	2
73	576.00	352.00	613.00	355.00	2
74	613.00	355.00	644.00	362.00	2
75	644.00	362.00	700.00	387.00	2
76	700.00	387.00	800.00	420.00	2
77	542.00	353.00	585.00	345.00	3
78	585.00	345.00	622.00	347.00	3
79	622.00	347.00	655.00	351.00	3
80	655.00	351.00	697.00	360.00	3
81	697.00	360.00	720.00	370.00	3
82	720.00	370.00	800.00	392.00	3
83	800.00	392.00	877.00	420.00	3
84	877.00	420.00	905.00	435.00	3
85	905.00	435.00	929.00	450.00	3
86	929.00	450.00	955.00	471.00	3
87	955.00	471.00	990.00	506.00	3
88	0.00	335.00	205.00	338.00	3
89	205.00	338.00	396.00	358.00	3
90	396.00	358.00	398.00	363.00	3

Default Y-Origin = 0.00(ft)

Default X-Plus Value = 0.00(ft)

Default Y-Plus Value = 0.00(ft)

1

ISOTROPIC SOIL PARAMETERS

3 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	130.0	0.0	40.0	0.00	0.0	1
2	120.0	130.0	0.0	17.3	0.00	0.0	1
3	120.0	130.0	0.0	40.0	0.00	0.0	1

ANISOTROPIC STRENGTH PARAMETERS
1 soil type(s)

Soil Type 2 Is Anisotropic

Number Of Direction Ranges Specified = 3

Direction Range No.	Counterclockwise Direction Limit (deg)	Cohesion Intercept (psf)	Friction Angle (deg)
1	0.0	0.00	40.00
2	30.0	0.00	17.33
3	90.0	0.00	40.00

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

1 PIEZOMETRIC SURFACE(S) SPECIFIED

Unit Weight of Water = 62.40 (pcf)

Piezometric Surface No. 1 Specified by 17 Coordinate Points
Pore Pressure Inclination Factor = 0.50

Point No.	X-Water (ft)	Y-Water (ft)
1	0.00	338.00
2	120.00	339.00
3	218.00	342.00
4	396.00	358.00
5	488.00	358.00

6	508.00	372.00
7	522.00	380.00
8	580.00	386.00
9	702.00	409.00
10	781.00	420.00
11	800.00	420.00
12	841.00	419.00
13	861.00	421.00
14	898.00	440.00
15	912.00	453.00
16	966.00	466.00
17	990.00	471.00

1

BOUNDARY LOAD(S)

2 Load(s) Specified

Load No.	X-Left (ft)	X-Right (ft)	Intensity (psf)	Deflection (deg)
1	106.00	182.00	100.0	0.0
2	219.00	240.00	100.0	0.0

NOTE - Intensity Is Specified As A Uniformly Distributed Force Acting On A Horizontally Projected Surface.

Specified Peak Ground Acceleration Coefficient (A) = 0.549(g)
Specified Horizontal Earthquake Coefficient (kh) = 0.263(g)
Specified Vertical Earthquake Coefficient (kv) = 0.000(g)

Specified Seismic Pore-Pressure Factor = 0.000

1

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Sliding Block Surfaces, Has Been Specified.

1000 Trial Surfaces Have Been Generated.

13 Boxes Specified For Generation Of Central Block Base

Length Of Line Segments For Active And Passive Portions Of Sliding Block Is 10.0

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height (ft)
1	488.00	394.00	488.00	394.00	1.00
2	508.00	372.00	508.00	372.00	1.00
3	525.00	360.00	525.00	360.00	1.00
4	542.00	353.00	542.00	353.00	1.00
5	585.00	345.00	585.00	345.00	1.00
6	622.00	347.00	622.00	347.00	1.00
7	655.00	351.00	655.00	351.00	1.00
8	697.00	360.00	697.00	360.00	1.00
9	720.00	370.00	720.00	370.00	1.00
10	800.00	392.00	800.00	392.00	1.00
11	877.00	420.00	877.00	420.00	1.00
12	905.00	435.00	905.00	435.00	1.00
13	929.00	450.00	929.00	450.00	1.00

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 (0-1) * *

Selected ki function = Constant (1.0)

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 = 1000

WARNING! The Factor of Safety Calculation for one or More Trial Surfaces
Did Not Converge in 20 Iterations.

Number of Trial Surfaces with Non-Converged FS = 737

Number of Trial Surfaces with Misleading FS = 239

Number of Trial Surfaces With Valid FS =   24

Percentage of Trial Surfaces With Non-Valid FS Solutions
of the Total Attempted = 97.6 %

Statistical Data On All Valid FS Values:
   FS Max =   3.814   FS Min =   3.813   FS Ave =   3.814
   Standard Deviation =   0.001   Coefficient of Variation =   0.02 %

((Simplified Janbu FS for Critical Surface = 1.788))

Failure Surface Specified By 35 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	407.393	439.261
2	411.349	436.562
3	418.676	429.756
4	427.743	425.540
5	436.934	421.597
6	445.049	415.754
7	452.153	408.716
8	459.311	401.734
9	468.793	398.556
10	478.070	394.822
11	488.000	393.644
12	508.000	372.027
13	525.000	359.983
14	542.000	352.733
15	585.000	345.244
16	622.000	347.236
17	655.000	351.176
18	697.000	360.285
19	720.000	369.531
20	800.000	391.632

21	877.000	407.393
22	905.000	411.349
23	929.000	418.676
24	930.881	428.498
25	936.408	436.831
26	943.462	443.919
27	948.119	452.768
28	952.994	461.499
29	956.582	470.834
30	957.261	480.811
31	958.312	490.755
32	964.621	498.514
33	971.669	505.608
34	974.708	515.135
35	974.839	515.474

*** FOS = 3.813 Theta (ki=1.0) = 22.50 ***
Lambda = 0.414

Individual data on the 78 slices

Slice No.	Width (ft)	Weight (lbs)	Water Force Top (lbs)	Water Force Bot (lbs)	Tie Force Norm (lbs)	Tie Force Tan (lbs)	Earthquake Force Hor (lbs)	Earthquake Force Ver (lbs)	Surcharge Load (lbs)
1	1.3	108.0	0.0	0.0	0.	0.	28.4	0.0	0.0
2	8.8	4419.4	0.0	0.0	0.	0.	1162.3	0.0	0.0
3	8.7	10366.4	0.0	0.0	0.	0.	2726.4	0.0	0.0
4	7.8	15378.1	0.0	0.0	0.	0.	4044.4	0.0	0.0
5	9.7	25410.6	0.0	0.0	0.	0.	6683.0	0.0	0.0
6	9.3	28880.8	0.0	0.0	0.	0.	7595.6	0.0	0.0
7	4.8	17241.7	0.0	0.0	0.	0.	4534.6	0.0	0.0
8	3.8	14900.1	0.0	0.0	0.	0.	3918.7	0.0	0.0
9	7.8	34638.8	0.0	0.0	0.	0.	9110.0	0.0	0.0
10	7.1	37264.4	0.0	0.0	0.	0.	9800.5	0.0	0.0
11	9.3	54786.0	0.0	0.0	0.	0.	14408.7	0.0	0.0
12	0.3	1617.7	0.0	0.0	0.	0.	425.4	0.0	0.0
13	19.7	148963.1	0.0	0.0	0.	0.	39177.3	0.0	0.0
14	0.1	826.4	0.0	0.0	0.	0.	217.4	0.0	0.0
15	3.5	31653.8	0.0	542.0	0.	0.	8325.0	0.0	0.0
16	10.4	102168.7	0.0	8090.2	0.	0.	26870.4	0.0	0.0
17	3.0	31540.7	0.0	4517.5	0.	0.	8295.2	0.0	0.0
18	12.0	132086.9	0.0	19178.6	0.	0.	34738.9	0.0	0.0
19	4.0	45354.8	0.0	7490.9	0.	0.	11928.3	0.0	0.0
20	1.0	11358.9	0.0	1958.5	0.	0.	2987.4	0.0	0.0
21	11.0	127354.9	0.0	21497.5	0.	0.	33494.3	0.0	0.0
22	2.0	23216.9	0.0	4138.0	0.	0.	6106.0	0.0	0.0
23	2.0	23312.7	0.0	4208.5	0.	0.	6131.2	0.0	0.0
24	19.0	234897.8	0.0	43500.2	0.	0.	61778.1	0.0	0.0
25	4.0	51886.6	0.0	9969.4	0.	0.	13646.2	0.0	0.0
26	3.5	45526.0	0.0	8772.0	0.	0.	11973.3	0.0	0.0
27	1.5	20509.2	0.0	3999.4	0.	0.	5393.9	0.0	0.0
28	1.0	13352.8	0.0	2580.0	0.	0.	3511.8	0.0	0.0
29	24.0	322471.5	0.0	64280.8	0.	0.	84810.0	0.0	0.0
30	3.0	40552.0	0.0	8353.7	0.	0.	10665.2	0.0	0.0
31	3.0	40606.0	0.0	8424.5	0.	0.	10679.4	0.0	0.0
32	6.0	81158.0	0.0	17061.5	0.	0.	21344.5	0.0	0.0
33	4.0	53944.9	0.0	11548.2	0.	0.	14187.5	0.0	0.0
34	7.6	101886.0	0.0	22156.8	0.	0.	26796.0	0.0	0.0
35	10.4	138677.3	0.0	30897.4	0.	0.	36472.1	0.0	0.0
36	11.0	145651.0	0.0	33297.9	0.	0.	38306.2	0.0	0.0
37	25.0	324107.8	0.0	77254.2	0.	0.	85240.3	0.0	0.0
38	17.0	212836.1	0.0	51799.1	0.	0.	55975.9	0.0	0.0
39	3.0	36756.4	0.0	9605.6	0.	0.	9666.9	0.0	0.0
40	2.0	24252.1	0.0	6321.0	0.	0.	6378.3	0.0	0.0

41	18.0	209172.5	0.0	53793.0	0.	0.	55012.4	0.0	0.0
42	28.0	300616.5	0.0	71337.9	0.	0.	79062.1	0.0	0.0
43	26.4	261970.9	0.0	60966.7	0.	0.	68898.3	0.0	0.0
44	6.6	61709.7	0.0	14165.7	0.	0.	16229.7	0.0	0.0
45	11.0	100481.7	0.0	22567.8	0.	0.	26426.7	0.0	0.0
46	8.0	71918.1	0.0	15045.7	0.	0.	18914.5	0.0	0.0
47	0.5	4658.7	0.0	920.1	0.	0.	1225.2	0.0	0.0
48	24.5	225208.5	0.0	38728.6	0.	0.	59229.8	0.0	0.0
49	5.0	45577.5	0.0	6727.9	0.	0.	11986.9	0.0	0.0
50	4.0	36168.4	0.0	5093.9	0.	0.	9512.3	0.0	0.0
51	7.0	64986.4	0.0	8297.4	0.	0.	17091.4	0.0	0.0
52	20.0	190611.0	0.0	20873.9	0.	0.	50130.7	0.0	0.0
53	16.0	158113.7	0.0	15982.2	0.	0.	41583.9	0.0	0.0
54	8.0	81951.8	0.0	9665.9	0.	0.	21553.3	0.0	0.0
55	13.0	135403.2	0.0	19263.3	0.	0.	35611.0	0.0	0.0
56	1.0	10368.4	0.0	1437.8	0.	0.	2726.9	0.0	0.0
57	4.0	42203.6	0.0	6174.1	0.	0.	11099.5	0.0	0.0
58	2.0	21564.8	0.0	3340.7	0.	0.	5671.5	0.0	0.0
59	7.0	76034.5	0.0	13461.4	0.	0.	19997.1	0.0	0.0
60	17.0	186986.8	0.0	42344.1	0.	0.	49177.5	0.0	0.0
61	5.0	52639.8	0.0	20450.6	0.	0.	13844.3	0.0	0.0
62	7.0	68836.7	0.0	16516.9	0.	0.	18104.0	0.0	0.0
63	0.6	5359.4	0.0	1178.1	0.	0.	1409.5	0.0	0.0
64	6.4	59770.6	0.0	12022.3	0.	0.	15719.7	0.0	0.0
65	7.0	60912.5	0.0	9886.3	0.	0.	16020.0	0.0	0.0
66	0.0	405.8	0.0	131.7	0.	0.	106.7	0.0	0.0
67	0.0	83.7	0.0	27.0	0.	0.	22.0	0.0	0.0
68	2.9	22387.1	0.0	5388.8	0.	0.	5887.8	0.0	0.0
69	0.1	454.6	0.0	418.5	0.	0.	119.6	0.0	0.0
70	0.1	719.5	0.0	266.9	0.	0.	189.2	0.0	0.0
71	0.2	1161.2	0.0	0.0	0.	0.	305.4	0.0	0.0
72	0.4	1889.7	0.0	0.0	0.	0.	497.0	0.0	0.0
73	0.3	1451.6	0.0	0.0	0.	0.	381.8	0.0	0.0
74	6.9	27465.0	0.0	0.0	0.	0.	7223.3	0.0	0.0
75	3.7	10891.1	0.0	0.0	0.	0.	2864.4	0.0	0.0
76	1.9	3412.0	0.0	0.0	0.	0.	897.4	0.0	0.0
77	1.3	751.7	0.0	0.0	0.	0.	197.7	0.0	0.0
78	0.0	0.0	0.0	0.0	0.	0.	0.0	0.0	0.0

Failure Surface Specified By 34 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	412.597	439.829
2	420.463	433.909
3	429.884	430.556
4	437.060	423.591
5	445.513	418.247
6	454.060	413.057
7	463.191	408.979
8	470.475	402.128
9	480.268	400.100
10	488.000	393.759
11	508.000	372.257
12	525.000	359.563
13	542.000	353.271
14	585.000	344.726
15	622.000	347.020
16	655.000	350.998
17	697.000	360.254
18	720.000	370.130
19	800.000	391.722
20	877.000	419.872
21	905.000	412.597
22	929.000	420.463
23	934.938	428.510
24	939.542	437.387
25	945.515	445.407
26	949.265	454.677

27	955.721	462.314
28	961.803	470.252
29	967.558	478.430
30	974.450	485.675
31	975.213	495.646
32	981.956	503.031
33	986.642	511.865
34	988.529	515.046

*** FOS = 3.813 Theta (ki=1.0) = 22.50 ***
Lambda = 0.414

1

Failure Surface Specified By 35 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	409.485	439.489
2	410.826	438.293
3	419.656	433.599
4	428.325	428.614
5	436.090	422.313
6	445.816	419.990
7	455.163	416.434
8	463.818	411.426
9	471.643	405.200
10	478.747	398.161
11	488.000	394.368
12	508.000	372.122
13	525.000	359.543
14	542.000	352.601
15	585.000	345.015
16	622.000	347.253
17	655.000	350.533
18	697.000	359.826
19	720.000	369.849
20	800.000	392.071
21	877.000	409.485
22	905.000	410.826
23	929.000	419.656
24	933.971	428.333
25	940.949	435.496
26	947.958	442.629
27	954.942	449.786
28	957.932	459.328
29	958.308	469.321
30	958.977	479.299
31	965.920	486.496
32	969.616	495.787
33	971.523	505.604
34	972.777	515.525
35	972.790	515.538

*** FOS = 3.813 Theta (ki=1.0) = 22.50 ***
Lambda = 0.414

Failure Surface Specified By 35 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)

1	407.393	439.261
2	411.349	436.562
3	418.676	429.756
4	427.743	425.540
5	436.934	421.597
6	445.049	415.754
7	452.153	408.716
8	459.311	401.734
9	468.793	398.556
10	478.070	394.822
11	488.000	393.644
12	508.000	372.027
13	525.000	359.983
14	542.000	352.733
15	585.000	345.244
16	622.000	347.236
17	655.000	351.176
18	697.000	360.285
19	720.000	369.531
20	800.000	391.632
21	877.000	407.393
22	905.000	411.349
23	929.000	418.676
24	930.881	428.498
25	936.408	436.831
26	943.462	443.919
27	948.119	452.768
28	952.994	461.499
29	956.582	470.834
30	957.261	480.811
31	958.312	490.755
32	964.621	498.514
33	971.669	505.608
34	974.708	515.135
35	974.839	515.474

*** FOS = 3.813 Theta (ki=1.0) = 22.50 ***
Lambda = 0.414

1

Failure Surface Specified By 34 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	412.597	439.829
2	420.463	433.909
3	429.884	430.556
4	437.060	423.591
5	445.513	418.247
6	454.060	413.057
7	463.191	408.979
8	470.475	402.128
9	480.268	400.100
10	488.000	393.759
11	508.000	372.257
12	525.000	359.563
13	542.000	353.271
14	585.000	344.726
15	622.000	347.020
16	655.000	350.998
17	697.000	360.254
18	720.000	370.130
19	800.000	391.722
20	877.000	419.872

21	905.000	412.597
22	929.000	420.463
23	934.938	428.510
24	939.542	437.387
25	945.515	445.407
26	949.265	454.677
27	955.721	462.314
28	961.803	470.252
29	967.558	478.430
30	974.450	485.675
31	975.213	495.646
32	981.956	503.031
33	986.642	511.865
34	988.529	515.046

*** FOS = 3.813 Theta (ki=1.0) = 22.50 ***
Lambda = 0.414

Failure Surface Specified By 35 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	409.485	439.489
2	410.826	438.293
3	419.656	433.599
4	428.325	428.614
5	436.090	422.313
6	445.816	419.990
7	455.163	416.434
8	463.818	411.426
9	471.643	405.200
10	478.747	398.161
11	488.000	394.368
12	508.000	372.122
13	525.000	359.543
14	542.000	352.601
15	585.000	345.015
16	622.000	347.253
17	655.000	350.533
18	697.000	359.826
19	720.000	369.849
20	800.000	392.071
21	877.000	409.485
22	905.000	410.826
23	929.000	419.656
24	933.971	428.333
25	940.949	435.496
26	947.958	442.629
27	954.942	449.786
28	957.932	459.328
29	958.308	469.321
30	958.977	479.299
31	965.920	486.496
32	969.616	495.787
33	971.523	505.604
34	972.777	515.525
35	972.790	515.538

*** FOS = 3.813 Theta (ki=1.0) = 22.50 ***
Lambda = 0.414

Failure Surface Specified By 35 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	407.393	439.261
2	411.349	436.562
3	418.676	429.756
4	427.743	425.540
5	436.934	421.597
6	445.049	415.754
7	452.153	408.716
8	459.311	401.734
9	468.793	398.556
10	478.070	394.822
11	488.000	393.644
12	508.000	372.027
13	525.000	359.983
14	542.000	352.733
15	585.000	345.244
16	622.000	347.236
17	655.000	351.176
18	697.000	360.285
19	720.000	369.531
20	800.000	391.632
21	877.000	407.393
22	905.000	411.349
23	929.000	418.676
24	930.881	428.498
25	936.408	436.831
26	943.462	443.919
27	948.119	452.768
28	952.994	461.499
29	956.582	470.834
30	957.261	480.811
31	958.312	490.755
32	964.621	498.514
33	971.669	505.608
34	974.708	515.135
35	974.839	515.474

*** FOS = 3.813 Theta (ki=1.0) = 22.50 ***
Lambda = 0.414

Failure Surface Specified By 34 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	412.597	439.829
2	420.463	433.909
3	429.884	430.556
4	437.060	423.591
5	445.513	418.247
6	454.060	413.057
7	463.191	408.979
8	470.475	402.128
9	480.268	400.100
10	488.000	393.759
11	508.000	372.257
12	525.000	359.563
13	542.000	353.271
14	585.000	344.726
15	622.000	347.020

16	655.000	350.998
17	697.000	360.254
18	720.000	370.130
19	800.000	391.722
20	877.000	419.872
21	905.000	412.597
22	929.000	420.463
23	934.938	428.510
24	939.542	437.387
25	945.515	445.407
26	949.265	454.677
27	955.721	462.314
28	961.803	470.252
29	967.558	478.430
30	974.450	485.675
31	975.213	495.646
32	981.956	503.031
33	986.642	511.865
34	988.529	515.046

*** FOS = 3.813 Theta (ki=1.0) = 22.50 ***
Lambda = 0.414

1

Failure Surface Specified By 35 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	409.485	439.489
2	410.826	438.293
3	419.656	433.599
4	428.325	428.614
5	436.090	422.313
6	445.816	419.990
7	455.163	416.434
8	463.818	411.426
9	471.643	405.200
10	478.747	398.161
11	488.000	394.368
12	508.000	372.122
13	525.000	359.543
14	542.000	352.601
15	585.000	345.015
16	622.000	347.253
17	655.000	350.533
18	697.000	359.826
19	720.000	369.849
20	800.000	392.071
21	877.000	409.485
22	905.000	410.826
23	929.000	419.656
24	933.971	428.333
25	940.949	435.496
26	947.958	442.629
27	954.942	449.786
28	957.932	459.328
29	958.308	469.321
30	958.977	479.299
31	965.920	486.496
32	969.616	495.787
33	971.523	505.604
34	972.777	515.525
35	972.790	515.538

*** FOS = 3.813 Theta (ki=1.0) = 22.50 ***
Lambda = 0.414

Failure Surface Specified By 35 Coordinate Points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	407.393	439.261
2	411.349	436.562
3	418.676	429.756
4	427.743	425.540
5	436.934	421.597
6	445.049	415.754
7	452.153	408.716
8	459.311	401.734
9	468.793	398.556
10	478.070	394.822
11	488.000	393.644
12	508.000	372.027
13	525.000	359.983
14	542.000	352.733
15	585.000	345.244
16	622.000	347.236
17	655.000	351.176
18	697.000	360.285
19	720.000	369.531
20	800.000	391.632
21	877.000	407.393
22	905.000	411.349
23	929.000	418.676
24	930.881	428.498
25	936.408	436.831
26	943.462	443.919
27	948.119	452.768
28	952.994	461.499
29	956.582	470.834
30	957.261	480.811
31	958.312	490.755
32	964.621	498.514
33	971.669	505.608
34	974.708	515.135
35	974.839	515.474

*** FOS = 3.813 Theta (ki=1.0) = 22.50 ***
Lambda = 0.414

**** END OF GSTABL7 OUTPUT ****

Surficial Slope Stability Calculations

$$F.S. = \frac{F_R}{F_D} = \frac{d\,(\gamma_s - 62.4)\cos^2\alpha\,\tan\phi + C}{d\,\gamma_s\,\sin\alpha\,\cos\alpha}$$

Soil Parameters:

γ_s = Total saturated weight of soil =	120	pcf
ϕ = Angle of internal friction =	28	degree
C = Cohesion =	250	psf

Factor of Safety, F.S.

Depth, d (ft)	Slope Ratio (horizontal:vertical) 2:1
1	5.72
2	3.11
3	2.25
4	1.81
5	1.55

SURFICIAL SLOPE STABILITY ANALYSIS		TABLE
AMERICAN GEOTECHNICAL	F.N. 33348.08	F1

APPENDIX H

STABILITY ANALYSIS OF 15.5-FOOT HIGH MECHANICALLY STABILIZED EARTH (MSE) WALL

AASHTO DESIGN METHOD
The Cove at El Niguel

PROJECT IDENTIFICATION

Title:	The Cove at El Niguel
Project Number:	33348 08
Client:	Laguna-Niguel Properties, Inc.
Designer:	JH
Station Number:	

Description:

Cross-Section J-J' 15.5-ft High & 1.5H Width MSE Wall

Company's information:

Name: American Geotechnical, Inc.
Street: 22725 Old Canal Road

Yorba Linda, CA 92887

Telephone #:	714-685-3900
Fax #:	714-685-3909
E-Mail:	

Original file path and name: U:\MSEW 3.0 Data\33348-08 The Cove at El Niguel\Section.....
.....5 ft MSE Wall_R1.BEN

Original date and time of creating this file: Tue Apr 10 11:45:27 2012

PROGRAM MODE: ANALYSIS
of a SIMPLE STRUCTURE
using GEOGRID as reinforcing material.

SOIL DATA

REINFORCED SOIL

Unit weight, γ	120.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	30.0 °

RETAINED SOIL

Unit weight, γ	120.0 lb/ft ^{3}
Design value of internal angle of friction, ϕ	30.0 °

FOUNDATION SOIL (Considered as an equivalent uniform soil)

Equivalent unit weight, $\gamma_{equiv.}$	120.0 lb/ft ^{3}
Equivalent internal angle of friction, $\phi_{equiv.}$	30.0 °
Equivalent cohesion, $c_{equiv.}$	0.0 lb/ft ^{2}

Water table does not affect bearing capacity

LATERAL EARTH PRESSURE COEFFICIENTS

Ka (internal stability) = 0.3333 (if batter is less than 10°, Ka is calculated from eq. 15. Otherwise, eq. 38 is utilized)
Inclination of internal slip plane, ψ = 60.00° (see Fig. 28 in DEMO 82).
Ka (external stability) = 0.5476 (eq. 17 is utilized to calculate Ka for all batters)
(For external stability user specified δ = 30.00°)

BEARING CAPACITY

Bearing capacity coefficients (calculated by MSEW): Nc = 30.14 N γ= 22.40

SEISMICITY

Note: specified α_o:ombined with I and δroduced a square root of -0.17 in eq. 37a.
MSEW set this square root to ZERO so that the Kae could be calculated. Be aware that the end results are likely erroneous and ARE PROVIDED FOR INFORMATION ONLY

Maximum ground acceleration coefficient, A = 0.549
Design acceleration coefficient in Internal Stability: Kh = Am = 0.549
Design acceleration coefficient in External Stability: Kh = 0.248 (Am = 0.248)
(Kh in External Stability is based on allowable displacement, d = 100 mm. using FHWA-NHI-00-043 equation)

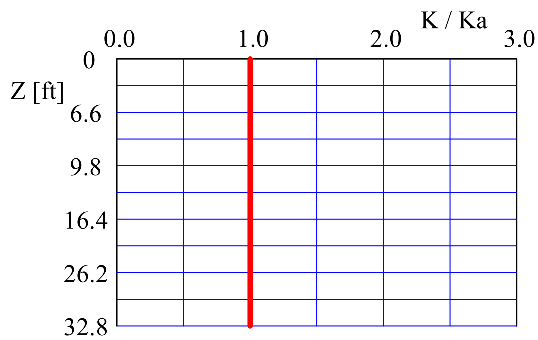
Kae (Kh > 0) = 1.3211 Kae (Kh = 0) = 0.5476 Δ Kae = 0.7734 (see eq. 37 in DEMO 82)
Seismic soil-geogrid friction coefficient, F* is 80.0% of its specified static value.

INPUT DATA: Geogrids
(Analysis)

D A T A	Geogrid type #1	Geogrid type #2	Geogrid type #3	Geogrid type #4	Geogrid type #5
Tult [lb/ft]	13705.0				
Durability reduction factor, RFd	1.10				
Installation-damage reduction factor, RFid	1.05				
Creep reduction factor, RFc	1.44	N/A	N/A	N/A	N/A
Fs-overall for strength	N/A				
Coverage ratio, Rc	1.000				
Friction angle along geogrid-soil interface, ρ	30.00				
Pullout resistance factor, F*	0.80·tanφ	N/A	N/A	N/A	N/A
Scale-effect correction factor, α	1.0				

Variation of Lateral Earth Pressure Coefficient With Depth

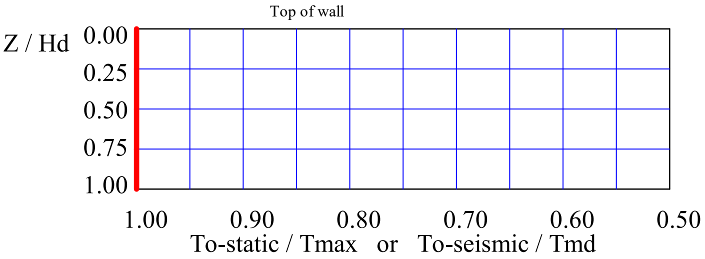
Z	K / Ka
0 ft	1.00
3.3 ft	1.00
6.6 ft	1.00
9.8 ft	1.00
13.1 ft	1.00
16.4 ft	1.00
19.7 ft	1.00



INPUT DATA: Facia and Connection (Analysis)

FACIA type: Segmental precast concrete panels.
Depth of panel is 1.31 ft. Horizontal distance to Center of Gravity of panel is 0.66 ft.
Average unit weight of panel is γ_f = 152.78 lb/ft ³

Z / Hd	To-static / Tmax or To-seismic / Tmd
0.00	1.00
0.25	1.00
0.50	1.00
0.75	1.00
1.00	1.00



D A T A (for connection only)	Type #1	Type #2	Type #3	Type #4	Type #5
Product Name	Miragrid ..	N/A	N/A	N/A	N/A
Durability reduction factor, RFd	1.10	N/A	N/A	N/A	N/A
Creep reduction factor, RFc	1.57	N/A	N/A	N/A	N/A
Overall factor of safety: connection break, Fs	N/A	N/A	N/A	N/A	N/A
Overall factor of safety: connection pullout, Fs	N/A	N/A	N/A	N/A	N/A
CRu = Tult-connection/Tult-geogrid	0.90	N/A	N/A	N/A	N/A

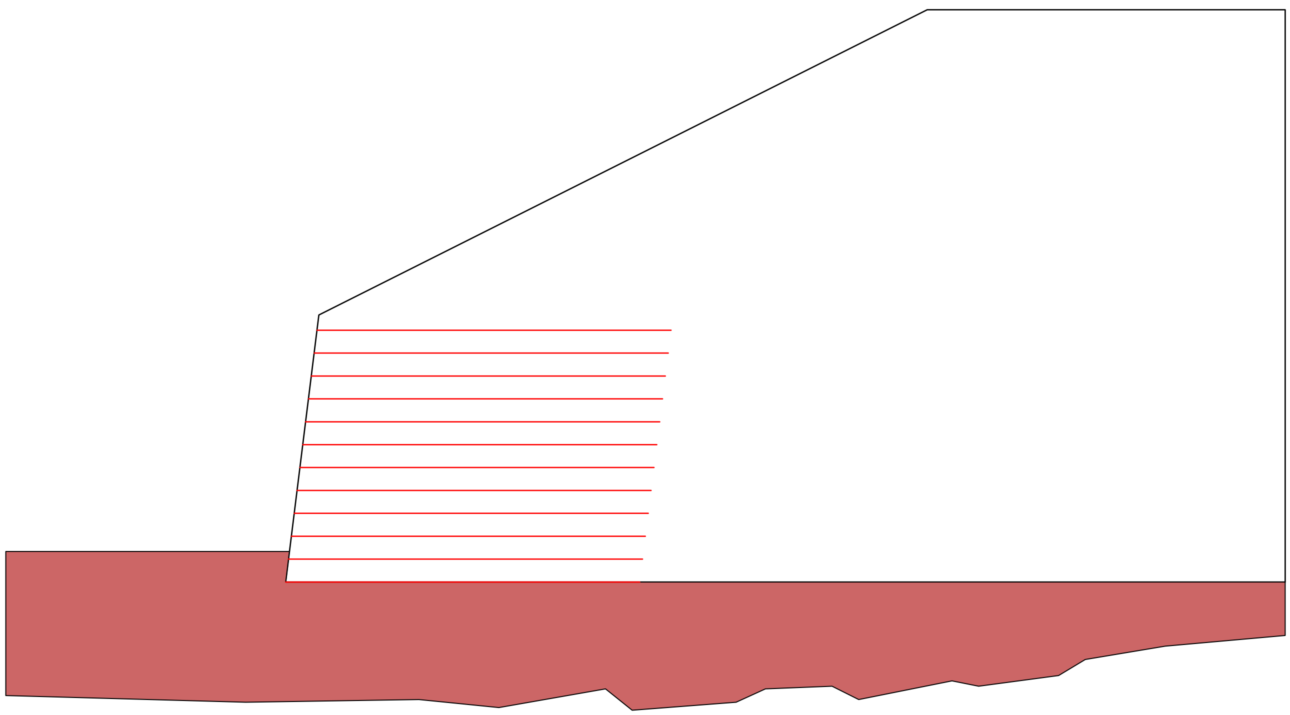
INPUT DATA: Geometry and Surcharge loads (of a SIMPLE STRUCTURE)

Design height, Hd	17.50	[ft]	{ Embedded depth is E = 2.00 ft, and height above top of finished bottom grade is H = 15.50 ft }
Batter, ω	7.1	[deg]	
Backslope, β	26.6	[deg]	
Backslope rise	20.0	[ft]	Broken back equivalent angle, I = 26.60° (see Fig. 25 in DEMO 82)

U N I F O R M   S U R C H A R G E

Uniformly distributed dead load is 0.0 [lb/ft ²]

ANALYZED REINFORCEMENT LAYOUT:



SCALE:

0 2 4 6 8 10 [ft]

ANALYSIS: CALCULATED FACTORS (Static conditions)

Bearing capacity, Fs = 9.35, Meyerhof stress = 3235 lb/ft².

Foundation Interface: Direct sliding, Fs = 1.877, Eccentricity, e/L = -0.0162, Fs-overturning = 5.17

GEOGRID				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.00	23.25	1	N/A	11.79	13.60	13.603	91.272	1.877	-0.0162	Miragrid 2..
2	1.50	23.25	1	N/A	6.24	7.20	7.203	45.269	1.942	-0.0246	Miragrid 2..
3	3.00	23.25	1	N/A	6.78	7.82	7.818	45.398	2.012	-0.0330	Miragrid 2..
4	4.50	23.25	1	N/A	7.41	8.55	8.548	45.168	2.086	-0.0414	Miragrid 2..
5	6.00	23.25	1	N/A	8.17	9.43	9.428	45.007	2.165	-0.0500	Miragrid 2..
6	7.50	23.25	1	N/A	9.11	10.51	10.510	45.046	2.248	-0.0590	Miragrid 2..
7	9.00	23.25	1	N/A	10.29	11.87	11.873	45.346	2.336	-0.0685	Miragrid 2..
8	10.50	23.25	1	N/A	11.82	13.64	13.642	46.064	2.427	-0.0791	Miragrid 2..
9	12.00	23.25	1	N/A	13.89	16.03	16.030	47.395	2.520	-0.0912	Miragrid 2..
10	13.50	23.25	1	N/A	16.84	19.43	19.433	49.729	2.610	-0.1057	Miragrid 2..
11	15.00	23.25	1	N/A	21.38	24.67	24.669	53.878	2.692	-0.1241	Miragrid 2..
12	16.50	23.25	1	N/A	25.88	29.86	29.861	54.694	2.750	-0.1491	Miragrid 2..

ANALYSIS: CALCULATED FACTORS (Seismic conditions)

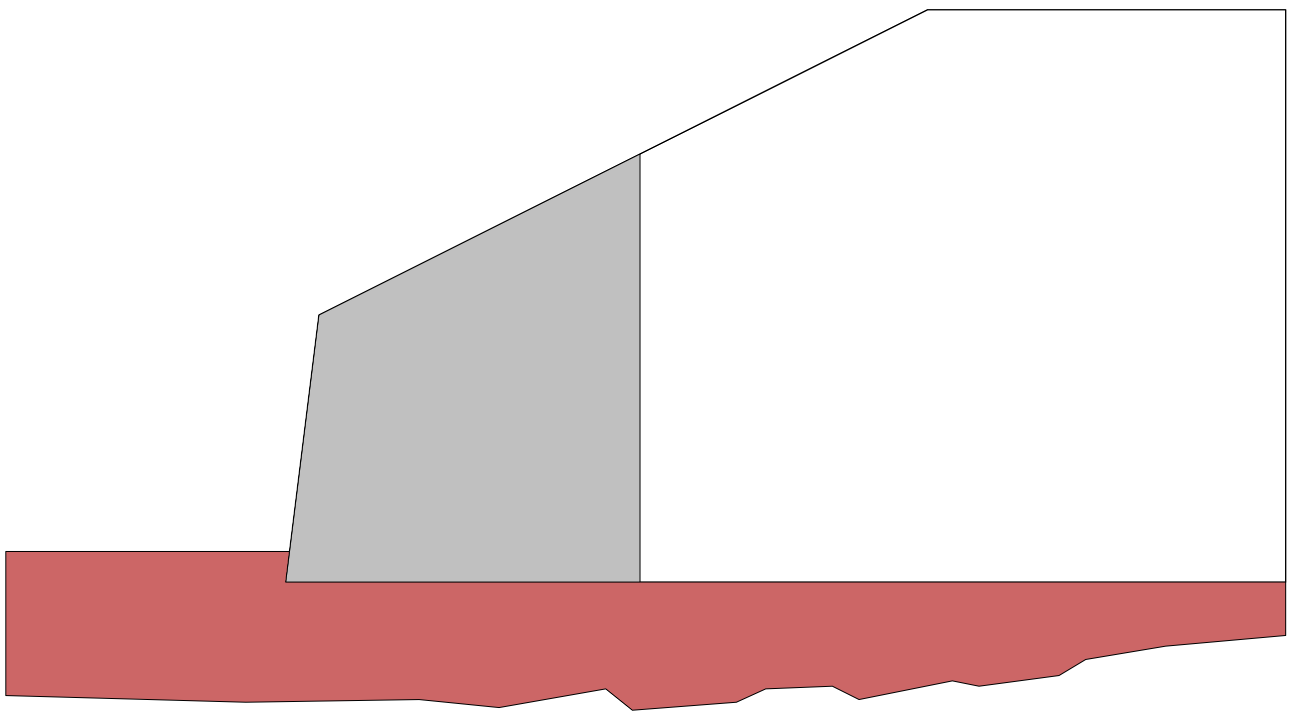
Bearing capacity, Fs = 6.05, Meyerhof stress = 4173 lb/ft².

Foundation Interface: Direct sliding, Fs = 1.199, Eccentricity, e/L = 0.0861, Fs-overturning = 2.93

GEOGRID				CONNECTION							
#	Elevation [ft]	Length [ft]	Type #	Fs-overall [pullout resistance]	Fs-overall [connection break]	Fs-overall [geogrid strength]	Geogrid strength Fs	Pullout resistance Fs	Direct sliding Fs	Eccentricity e/L	Product name
1	0.00	23.25	1	N/A	7.24	8.07	8.074	36.765	1.199	0.0861	Miragrid 2..
2	1.50	23.25	1	N/A	4.72	5.33	5.327	24.033	1.270	0.0631	Miragrid 2..
3	3.00	23.25	1	N/A	5.06	5.70	5.704	23.681	1.350	0.0411	Miragrid 2..
4	4.50	23.25	1	N/A	5.45	6.14	6.138	23.084	1.441	0.0199	Miragrid 2..
5	6.00	23.25	1	N/A	5.90	6.64	6.643	22.452	1.546	-0.0004	Miragrid 2..
6	7.50	23.25	1	N/A	6.44	7.24	7.239	21.831	1.668	-0.0201	Miragrid 2..
7	9.00	23.25	1	N/A	7.09	7.95	7.952	21.215	1.808	-0.0393	Miragrid 2..
8	10.50	23.25	1	N/A	7.88	8.82	8.822	20.623	1.970	-0.0583	Miragrid 2..
9	12.00	23.25	1	N/A	8.86	9.90	9.904	20.054	2.154	-0.0776	Miragrid 2..
10	13.50	23.25	1	N/A	10.14	11.29	11.290	19.514	2.356	-0.0979	Miragrid 2..
11	15.00	23.25	1	N/A	11.84	13.13	13.126	19.018	2.560	-0.1208	Miragrid 2..
12	16.50	23.25	1	N/A	13.37	14.78	14.778	17.717	2.722	-0.1485	Miragrid 2..

BEARING CAPACITY for GIVEN LAYOUT

	STATIC	SEISMIC	UNITS
(Water table does not affect bearing capacity)			
Ultimate bearing capacity, q-ult	30241	25235	[lb/ft ²]
Meyerhof stress, σ_V	3235.0	4173	[lb/ft ²]
Eccentricity, e	-0.38	2.24	[ft]
Eccentricity, e/L	-0.016	0.096	
Fs calculated	9.35	6.05	
Base length	23.25	23.25	[ft]



SCALE:

0 2 4 6 8 10 [ft]

DIRECT SLIDING for GIVEN LAYOUT
(for GEOGRID reinforcements)

Along reinforced and foundation soils interface: Fs-static = 1.877 and Fs-seismic = 1.199

#	Geogrid Elevation [ft]	Geogrid Length [ft]	Fs Static	Fs Seismic	Geogrid Type #	Product name
1	0.00	23.25	1.877	1.199	1	Miragrid 20XT
2	1.50	23.25	1.942	1.270	1	Miragrid 20XT
3	3.00	23.25	2.012	1.350	1	Miragrid 20XT
4	4.50	23.25	2.086	1.441	1	Miragrid 20XT
5	6.00	23.25	2.165	1.546	1	Miragrid 20XT
6	7.50	23.25	2.248	1.668	1	Miragrid 20XT
7	9.00	23.25	2.336	1.808	1	Miragrid 20XT
8	10.50	23.25	2.427	1.970	1	Miragrid 20XT
9	12.00	23.25	2.520	2.154	1	Miragrid 20XT
10	13.50	23.25	2.610	2.356	1	Miragrid 20XT
11	15.00	23.25	2.692	2.560	1	Miragrid 20XT
12	16.50	23.25	2.750	2.722	1	Miragrid 20XT

ECCENTRICITY for GIVEN LAYOUT

At interface with foundation: e/L static = -0.0162, e/L seismic = 0.0861; Overturning: Fs-static = 5.17, Fs-seismic = 2.93

#	Geogrid Elevation [ft]	Geogrid Length [ft]	e / L Static	e / L Seismic	Geogrid Type #	Product name
1	0.00	23.25	-0.0162	0.0861	1	Miragrid 20XT
2	1.50	23.25	-0.0246	0.0631	1	Miragrid 20XT
3	3.00	23.25	-0.0330	0.0411	1	Miragrid 20XT
4	4.50	23.25	-0.0414	0.0199	1	Miragrid 20XT
5	6.00	23.25	-0.0500	-0.0004	1	Miragrid 20XT
6	7.50	23.25	-0.0590	-0.0201	1	Miragrid 20XT
7	9.00	23.25	-0.0685	-0.0393	1	Miragrid 20XT
8	10.50	23.25	-0.0791	-0.0583	1	Miragrid 20XT
9	12.00	23.25	-0.0912	-0.0776	1	Miragrid 20XT
10	13.50	23.25	-0.1057	-0.0979	1	Miragrid 20XT
11	15.00	23.25	-0.1241	-0.1208	1	Miragrid 20XT
12	16.50	23.25	-0.1491	-0.1485	1	Miragrid 20XT

RESULTS for STRENGTH   Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Tavailable [lb/ft]	Tmax [lb/ft]	Tmd [lb/ft]	Specified minimum Fs-overall static	Actual calculated Fs-overall static	Specified minimum Fs-overall seismic	Actual calculated Fs-overall seismic	Product name
1	0.00	8240	605.76	597.32	N/A	13.603	N/A	8.074	Miragrid 2..
2	1.50	8240	1144.03	579.89	N/A	7.203	N/A	5.327	Miragrid 2..
3	3.00	8240	1054.03	562.45	N/A	7.818	N/A	5.704	Miragrid 2..
4	4.50	8240	964.03	545.02	N/A	8.548	N/A	6.138	Miragrid 2..
5	6.00	8240	874.03	527.59	N/A	9.428	N/A	6.643	Miragrid 2..
6	7.50	8240	784.03	510.16	N/A	10.510	N/A	7.239	Miragrid 2..
7	9.00	8240	694.03	492.73	N/A	11.873	N/A	7.952	Miragrid 2..
8	10.50	8240	604.03	475.29	N/A	13.642	N/A	8.822	Miragrid 2..
9	12.00	8240	514.03	457.86	N/A	16.030	N/A	9.904	Miragrid 2..
10	13.50	8240	424.03	440.43	N/A	19.433	N/A	11.290	Miragrid 2..
11	15.00	8240	334.03	423.00	N/A	24.669	N/A	13.126	Miragrid 2..
12	16.50	8240	275.95	405.57	N/A	29.861	N/A	14.778	Miragrid 2..

RESULTS for PULLOUT   Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Coverage Ratio	Tmax [lb/ft]	Tmd [lb/ft]	Le [ft]	La [ft]	Avail.Static Pullout, Pr [lb/ft]	Specified Static Fs	Actual Static Fs	Avail.Seism. Pullout, Pr [lb/ft]	Specified Seismic Fs	Actual Seismic Fs
1	0.00	1.000	605.8	597.3	23.25	0.00	55289.7	N/A	91.272	44231.7	N/A	36.765
2	1.50	1.000	1144.0	579.9	22.57	0.68	51788.9	N/A	45.269	41431.1	N/A	24.033
3	3.00	1.000	1054.0	562.5	21.89	1.36	47850.5	N/A	45.398	38280.4	N/A	23.681
4	4.50	1.000	964.0	545.0	21.21	2.04	43543.1	N/A	45.168	34834.5	N/A	23.084
5	6.00	1.000	874.0	527.6	20.54	2.71	39337.3	N/A	45.007	31469.9	N/A	22.452
6	7.50	1.000	784.0	510.2	19.86	3.39	35317.4	N/A	45.046	28253.9	N/A	21.831
7	9.00	1.000	694.0	492.7	19.18	4.07	31471.4	N/A	45.346	25177.1	N/A	21.215
8	10.50	1.000	604.0	475.3	18.50	4.75	27824.0	N/A	46.064	22259.2	N/A	20.623
9	12.00	1.000	514.0	457.9	17.82	5.43	24362.4	N/A	47.395	19490.0	N/A	20.054
10	13.50	1.000	424.0	440.4	17.14	6.11	21086.7	N/A	49.729	16869.4	N/A	19.514
11	15.00	1.000	334.0	423.0	16.46	6.79	17996.8	N/A	53.878	14397.5	N/A	19.018
12	16.50	1.000	276.0	405.6	15.79	7.46	15092.8	N/A	54.694	12074.2	N/A	17.717

RESULTS for CONNECTION (static conditions)

Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Connection force, To [lb/ft]	Reduction factor for connection break, CRu	Reduction factor for connection pullout, CRs	Available connection strength, Tc-break criterion [lb/ft]	Available connection strength, Tc-pullout criterion [lb/ft]	Available Geogrid strength, Tavailable [lb/ft]	Fs-overall connection break Specified	Fs-overall connection break Actual	Fs-overall connection pullout Specified	Fs-overall connection pullout Actual	Fs-overall Geogrid strength Specified	Fs-overall Geogrid strength Actual	Product name
1	0.00	606	0.90	N/A	7142	N/A	8240	N/A	11.79	N/A	N/A	N/A	13.60	Miragrid 2..
2	1.50	1144	0.90	N/A	7142	N/A	8240	N/A	6.24	N/A	N/A	N/A	7.20	Miragrid 2..
3	3.00	1054	0.90	N/A	7142	N/A	8240	N/A	6.78	N/A	N/A	N/A	7.82	Miragrid 2..
4	4.50	964	0.90	N/A	7142	N/A	8240	N/A	7.41	N/A	N/A	N/A	8.55	Miragrid 2..
5	6.00	874	0.90	N/A	7142	N/A	8240	N/A	8.17	N/A	N/A	N/A	9.43	Miragrid 2..
6	7.50	784	0.90	N/A	7142	N/A	8240	N/A	9.11	N/A	N/A	N/A	10.51	Miragrid 2..
7	9.00	694	0.90	N/A	7142	N/A	8240	N/A	10.29	N/A	N/A	N/A	11.87	Miragrid 2..
8	10.50	604	0.90	N/A	7142	N/A	8240	N/A	11.82	N/A	N/A	N/A	13.64	Miragrid 2..
9	12.00	514	0.90	N/A	7142	N/A	8240	N/A	13.89	N/A	N/A	N/A	16.03	Miragrid 2..
10	13.50	424	0.90	N/A	7142	N/A	8240	N/A	16.84	N/A	N/A	N/A	19.43	Miragrid 2..
11	15.00	334	0.90	N/A	7142	N/A	8240	N/A	21.38	N/A	N/A	N/A	24.67	Miragrid 2..
12	16.50	276	0.90	N/A	7142	N/A	8240	N/A	25.88	N/A	N/A	N/A	29.86	Miragrid 2..

RESULTS for CONNECTION (seismic conditions)

Live Load included in calculating Tmax

#	Geogrid Elevation [ft]	Connection force, To [lb/ft]	Reduction factor for connection break, CRu	Reduction factor for connection pullout, CRs	Available connection strength, Tc-break criterion [lb/ft]	Available connection strength, Tc-pullout criterion [lb/ft]	Available Geogrid strength, Tavailable [lb/ft]	Fs-overall connection break Specified	Fs-overall connection break Actual	Fs-overall connection pullout Specified	Fs-overall connection pullout Actual	Fs-overall Geogrid strength Specified	Fs-overall Geogrid strength Actual	Product name
1	0.00	1203	0.90	N/A	7142	N/A	8240	N/A	7.24	N/A	N/A	N/A	8.07	Miragrid 2..
2	1.50	1724	0.90	N/A	7142	N/A	8240	N/A	4.72	N/A	N/A	N/A	5.33	Miragrid 2..
3	3.00	1616	0.90	N/A	7142	N/A	8240	N/A	5.06	N/A	N/A	N/A	5.70	Miragrid 2..
4	4.50	1509	0.90	N/A	7142	N/A	8240	N/A	5.45	N/A	N/A	N/A	6.14	Miragrid 2..
5	6.00	1402	0.90	N/A	7142	N/A	8240	N/A	5.90	N/A	N/A	N/A	6.64	Miragrid 2..
6	7.50	1294	0.90	N/A	7142	N/A	8240	N/A	6.44	N/A	N/A	N/A	7.24	Miragrid 2..
7	9.00	1187	0.90	N/A	7142	N/A	8240	N/A	7.09	N/A	N/A	N/A	7.95	Miragrid 2..
8	10.50	1079	0.90	N/A	7142	N/A	8240	N/A	7.88	N/A	N/A	N/A	8.82	Miragrid 2..
9	12.00	972	0.90	N/A	7142	N/A	8240	N/A	8.86	N/A	N/A	N/A	9.90	Miragrid 2..
10	13.50	864	0.90	N/A	7142	N/A	8240	N/A	10.14	N/A	N/A	N/A	11.29	Miragrid 2..
11	15.00	757	0.90	N/A	7142	N/A	8240	N/A	11.84	N/A	N/A	N/A	13.13	Miragrid 2..
12	16.50	682	0.90	N/A	7142	N/A	8240	N/A	13.37	N/A	N/A	N/A	14.78	Miragrid 2..

33348-08 The Cove at El Niguel

PROJECT IDENTIFICATION

Title:	33348-08 The Cove at El Niguel
Project Number:	33348 - 08
Client:	Laguna-Niguel Properties, Inc.
Designer:	JH

Description:

Company's information:

Name:	American Geotechnical, Inc.
Street:	22725 Old Canal Road Yorba Linda, CA 92887
Telephone #:	714-685-3900
Fax #:	714-685-3909
E-Mail:	

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Original date and time of creating this file: Fri Dec 18 16:01:54 2020

PROGRAM MODE: Analysis of a General Slope using GEOSYNTHETIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

=========== Soil Layer #: ===========	Unit weight, γ [lb/ft ³]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ²]
....1..	120.0	30.0	0.0
....2..	120.0	30.0	0.0

REINFORCEMENT

Reinforcement Type #	Geosynthetic Designated Name	Ultimate Strength, Tult [lb/ft]	Reduction Factor for Installation Damage, RFid	Reduction Factor for Durability, RFd	Reduction Factor for Creep, RFc	Additional Reduction Factor, RFa	Coverage Ratio, Rc
1	Miragrid 20XT	13705.00	1.05	1.10	1.44	1.00	1.00

Interaction Parameters Type #	Geosynthetic Designated Name	== Direct Sliding == Cds-phi	Cds-c	==== Pullout ==== Ci	Alpha
1	Miragrid 20XT	0.80	0.00	0.80	0.80

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER

Unit weight of water = 62.45 [lb/ft ³]
Water pressure is defined by phreatic surface in Effective Stress Analysis.

SEISMICITY

Not Applicable

DRAWING OF SPECIFIED GEOMETRY - COMPLEX - Quick Input

-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.
-- Xw,Yw represents the coordinates of phreatic surface.

GEOMETRY

Soil profile contains 2 layers (see details in next page)

WATER GEOMETRY

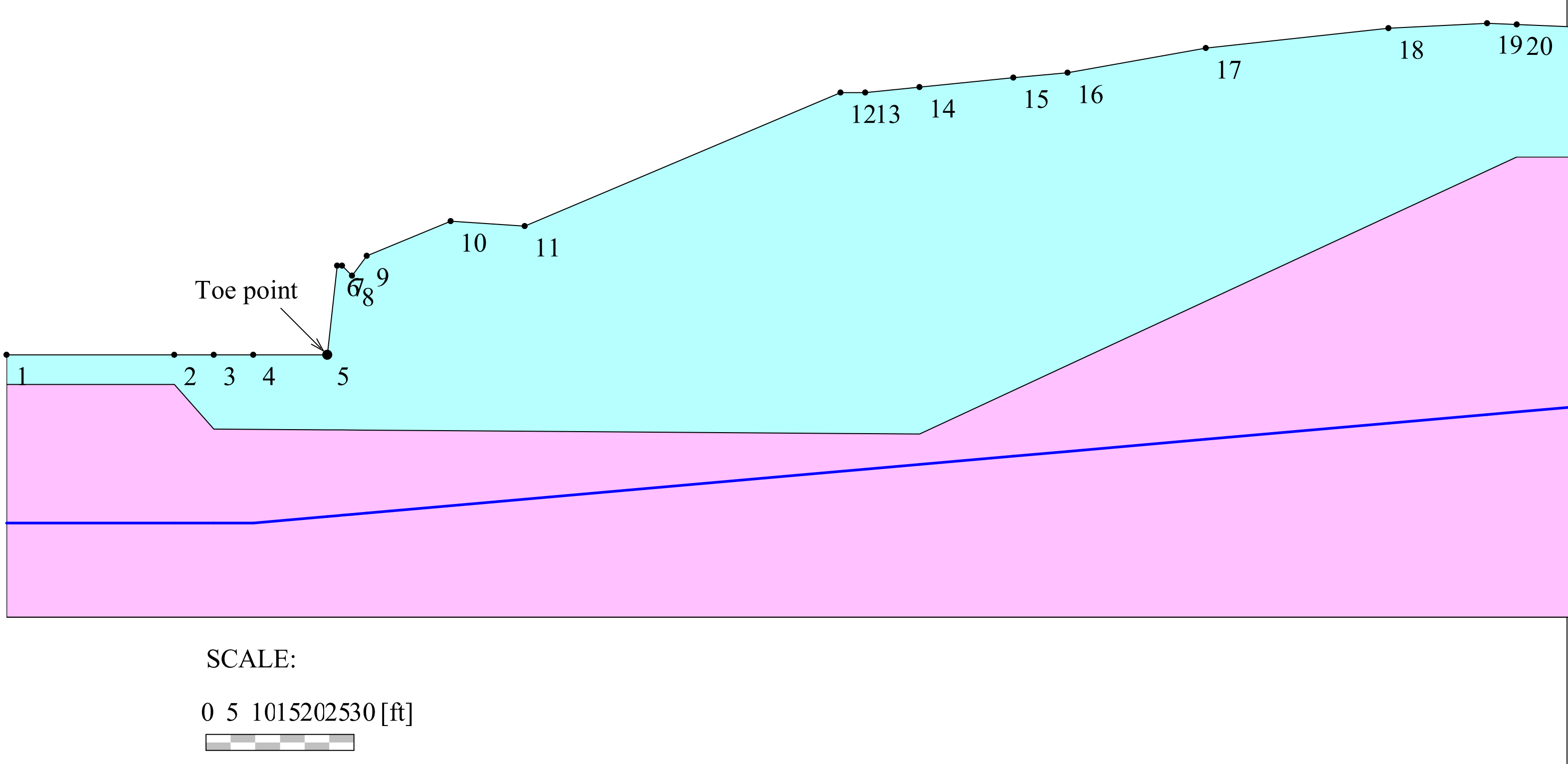
Phreatic line was specified.

UNIFORM SURCHARGE

Surcharge load, Q1 .... None
Surcharge load, Q2 .... None
Surcharge load, Q3 .... None

STRIP LOAD

............................None..................................



TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 2 layers. Coordinates in [ft.]
Water was described by phreatic line.

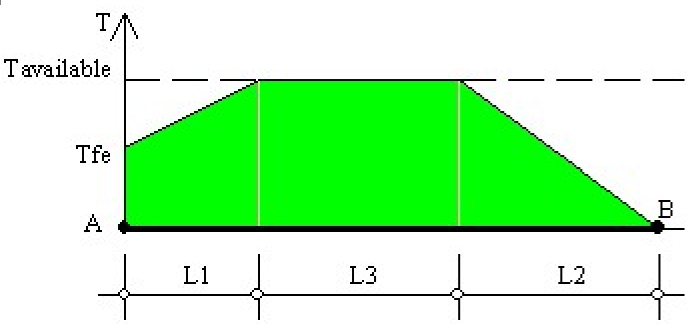
	#	Xi	Yi
Top of Layer 1	1	0.00	381.00
	2	65.00	381.00
	3	67.00	399.00
	4	68.00	399.00
	5	70.00	397.00
	6	73.00	401.00
	7	90.00	408.00
	8	105.00	407.00
	9	169.00	434.00
	10	174.00	434.00
	11	204.00	437.00
	12	215.00	438.00
	13	243.00	443.00
	14	280.00	447.00
	15	300.00	448.00
	16	324.00	447.00
	17	335.00	447.00
Top of Layer 2	18	0.00	375.00
	19	34.00	375.00
	20	42.00	366.00
	21	185.00	365.00
	22	306.00	421.00
	23	335.00	421.00
Top of Phreatic Line	25	0.00	347.00
	26	50.00	347.00
	27	335.00	372.00

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 2 layers. Coordinates in [ft.]
Water was described by phreatic line. Y values are tabulated in the right most column.

#	X	Y1	Y2	(phreatic) Yw
1	0.00	381.00	375.00	347.00
2	34.00	381.00	375.00	347.00
3	42.00	381.00	366.00	347.00
4	50.00	381.00	365.94	347.00
5	65.00	381.00	365.84	348.32
6	67.00	399.00	365.83	348.49
7	68.00	399.00	365.82	348.58
8	70.00	397.00	365.80	348.75
9	73.00	401.00	365.78	349.02
10	90.00	408.00	365.66	350.51
11	105.00	407.00	365.56	351.82
12	169.00	434.00	365.11	357.44
13	174.00	434.00	365.08	357.88
14	185.00	435.10	365.00	358.84
15	204.00	437.00	373.79	360.51
16	215.00	438.00	378.88	361.47
17	243.00	443.00	391.84	363.93
18	280.00	447.00	408.97	367.18
19	300.00	448.00	418.22	368.93
20	306.00	447.75	421.00	369.46
21	324.00	447.00	421.00	371.04
22	335.00	447.00	421.00	372.00

DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER



A = Front-end of reinforcement (at face of slope)
B = Rear-end of reinforcement
AB = L1 + L2 + L3 = Embedded length of reinforcement

Tavailable = Long-term strength of reinforcement
Tfe = Available front-end strength (e.g., connection to facing)

L1 = Front-end 'pullout' length
L2 = Rear-end pullout length
Tavailable prevails along L3

Factor of safety on resistance to pullout on either end of reinforcement, Fs-po = 1.50

Reinforcement Layer #	Designated Name	Height Relative to Toe [ft]	L [ft]	L1 [ft]	L2 [ft]	L3 [ft]	Tfe [lb/ft]	Tavailable [lb/ft]
1	Miragrid 20XT	0.00	23.25	0.00	5.58	17.67	8240.14	8240.14
2	Miragrid 20XT	1.50	23.25	0.00	5.91	17.34	8240.14	8240.14
3	Miragrid 20XT	3.00	23.25	0.00	6.33	16.92	8240.14	8240.14
4	Miragrid 20XT	4.50	23.25	0.00	6.79	16.46	8240.14	8240.14
5	Miragrid 20XT	6.00	23.25	0.00	7.35	15.90	8240.14	8240.14
6	Miragrid 20XT	7.50	23.25	0.00	8.01	15.24	8240.14	8240.14
7	Miragrid 20XT	9.00	23.25	0.00	8.79	14.46	8240.14	8240.14
8	Miragrid 20XT	10.50	23.25	0.00	9.81	13.44	8240.14	8240.14
9	Miragrid 20XT	12.00	23.25	0.00	11.12	12.13	8240.14	8240.14
10	Miragrid 20XT	13.50	23.25	0.00	13.02	10.23	8240.14	8240.14
11	Miragrid 20XT	-2.00	25.00	0.00	4.99	20.01	8240.14	8240.14

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 3.31

Critical Circle: Xc = 65.95[ft], Yc = 407.71[ft], R = 31.85[ft]. (Number of slices used = 54)

Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Minimum Factor of Safety = 1.55

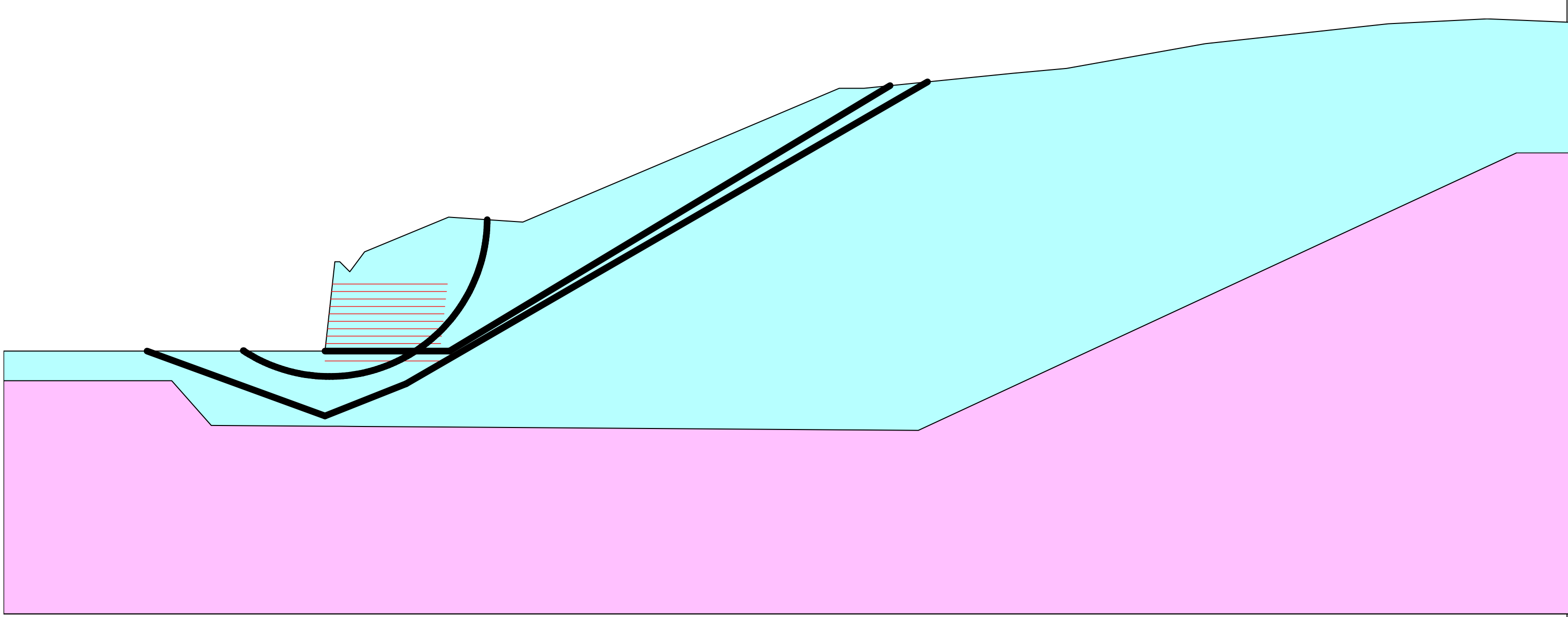
Critical Two-Part Wedge: (Xa = 65.00, Ya = 381.00) [ft]
(Xb = 90.19, Yb = 381.00) [ft]
(Xc = 179.27, Yc = 434.53) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 24.23 [degrees]

Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.61

Critical Three-Part Wedge: (X2 = 29.01, Y2 = 381.00) [ft]
(X-left = 65.00, Y-left = 367.90) [ft]
(X-right = 81.40, Y-right = 374.40) [ft]
(X1 = 186.86, Y1 = 435.29) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 19.66 [degrees]

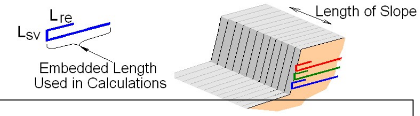
REINFORCEMENT LAYOUT: DRAWING



SCALE:

0 5 10 15 20 25 30 [ft]

REINFORCEMENT LAYOUT: TABULATED DATA & QUANTITIES



Layer #	Reinf. Type #	Geosynthetic Designated Name	Height Relative to Toe [ft]	Embedded Length [ft]	Covergae Ratio, Rc	(X, Y) front [ft]		(X, Y) rear [ft]		Lsv * [ft]	Lre [ft]
1	1	Miragrid 20XT	0.00	23.25	1.00	213.25	1250.00	236.50	1250.00	0.00	0.00
2	1	Miragrid 20XT	1.50	23.25	1.00	213.42	1251.50	236.67	1251.50	0.00	0.00
3	1	Miragrid 20XT	3.00	23.25	1.00	213.59	1253.00	236.84	1253.00	0.00	0.00
4	1	Miragrid 20XT	4.50	23.25	1.00	213.75	1254.50	237.00	1254.50	0.00	0.00
5	1	Miragrid 20XT	6.00	23.25	1.00	213.92	1256.00	237.17	1256.00	0.00	0.00
6	1	Miragrid 20XT	7.50	23.25	1.00	214.09	1257.50	237.34	1257.50	0.00	0.00
7	1	Miragrid 20XT	9.00	23.25	1.00	214.25	1259.00	237.50	1259.00	0.00	0.00
8	1	Miragrid 20XT	10.50	23.25	1.00	214.42	1260.50	237.67	1260.50	0.00	0.00
9	1	Miragrid 20XT	12.00	23.25	1.00	214.59	1262.00	237.84	1262.00	0.00	0.00
10	1	Miragrid 20XT	13.50	23.25	1.00	214.75	1263.50	238.00	1263.50	0.00	0.00
11	1	Miragrid 20XT	-2.00	25.00	1.00	213.25	1248.00	238.25	1248.00	0.00	0.00

* Vertical distance between layers.

QUANTITIES

Reinf. Type #	Designated Name	Coverage Ratio	Area of reinforcemnt [ft²] / length of slope [ft]
1	Miragrid 20XT	1.00	257.50

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33348-08 The Cove at El Niguel

PROJECT IDENTIFICATION

Title:	33348-08 The Cove at El Niguel
Project Number:	33348 - 08
Client:	Laguna-Niguel Properties, Inc.
Designer:	JH

Description:

Company's information:

Name:	American Geotechnical, Inc.
Street:	22725 Old Canal Road Yorba Linda, CA 92887
Telephone #:	714-685-3900
Fax #:	714-685-3909
E-Mail:	

Original file path and name: U:\ReSSA 3 ..... iguel_15.5' High & 1.5H Width MSE Wall_Seismic.MSE
Original date and time of creating this file: Fri Dec 18 16:01:54 2020

PROGRAM MODE: Analysis of a General Slope using GEOSYNTHETIC as reinforcing material.

INPUT DATA (EXCLUDING REINFORCEMENT LAYOUT)

SOIL DATA

========== Soil Layer #: ==========	Unit weight, γ [lb/ft ³]	Internal angle of friction, ϕ [deg.]	Cohesion, c [lb/ft ²]
....1..	120.0	40.0	0.0
....2..	120.0	40.0	0.0

REINFORCEMENT

Reinforcement Type #	Geosynthetic Designated Name	Ultimate Strength, Tult [lb/ft]	Reduction Factor for Installation Damage, RFid	Reduction Factor for Durability, RFd	Reduction Factor for Creep, RFc	Additional Reduction Factor, RFa	Coverage Ratio, Rc
1	Miragrid 20XT	13705.00	1.05	1.10	1.44	1.00	1.00

Interaction Parameters Type #	Geosynthetic Designated Name	== Direct Sliding == Cds-phi	Cds-c	==== Pullout ==== Ci	Alpha
1	Miragrid 20XT	0.80	0.00	0.80	0.80

Relative Orientation of Reinforcement Force, ROR = 0.00. Assigned Factor of Safety to resist pullout, Fs-po = 1.50
Design method for Global Stability: Comprehensive Bishop.

WATER

Unit weight of water = 62.45 [lb/ft ³]
Water pressure is defined by phreatic surface in Effective Stress Analysis.

SEISMICITY

Horizontal peak ground acceleration coefficient, Ao = 0.549
Design horizontal seismic coefficient, kh = Am = 0.5 x Ao = 0.275 & design vertical seismic coefficient, kv (down) = 0.000 x kh = 0.000

DRAWING OF SPECIFIED GEOMETRY - COMPLEX - Quick Input

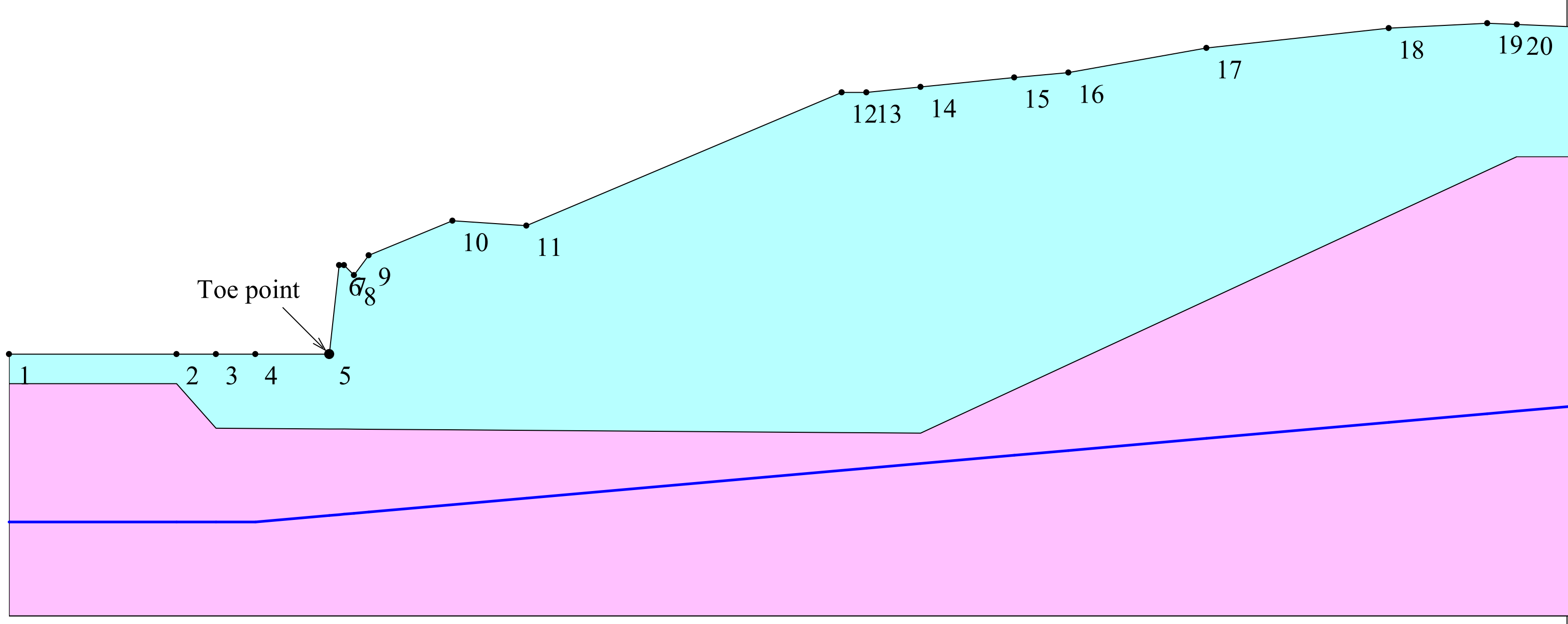
-- Problem geometry is defined along sections selected by user at x,y coordinates.
-- X1,Y1 represents the coordinates of soil surface. X2,Y2 represent the coordinates of the end of soil layer 1 and start of soil layer 2, and so on.
-- Xw,Yw represents the coordinates of phreatic surface.

GEOMETRY
Soil profile contains 2 layers (see details in next page)

WATER GEOMETRY
Phreatic line was specified.

UNIFORM SURCHARGE
Surcharge load, Q1 .... None
Surcharge load, Q2 .... None
Surcharge load, Q3 .... None

STRIP LOAD
............................None..................................



SCALE:

0 5 1015202530 [ft]

TABULATED DETAILS OF QUICK SPECIFIED GEOMETRY

Soil profile contains 2 layers. Coordinates in [ft.]
Water was described by phreatic line.

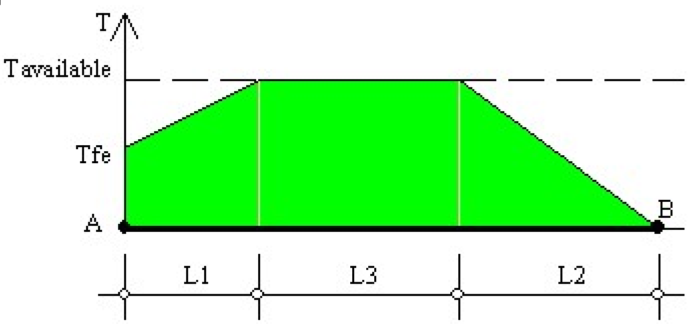
	#	Xi	Yi
Top of Layer 1	1	0.00	381.00
	2	65.00	381.00
	3	67.00	399.00
	4	68.00	399.00
	5	70.00	397.00
	6	73.00	401.00
	7	90.00	408.00
	8	105.00	407.00
	9	169.00	434.00
	10	174.00	434.00
	11	204.00	437.00
	12	215.00	438.00
	13	243.00	443.00
	14	280.00	447.00
	15	300.00	448.00
	16	324.00	447.00
	17	335.00	447.00
Top of Layer 2	18	0.00	375.00
	19	34.00	375.00
	20	42.00	366.00
	21	185.00	365.00
	22	306.00	421.00
	23	335.00	421.00
Top of Phreatic Line	25	0.00	347.00
	26	50.00	347.00
	27	335.00	372.00

TABULATED DETAILS OF SPECIFIED GEOMETRY

Soil profile contains 2 layers. Coordinates in [ft.]
Water was described by phreatic line. Y values are tabulated in the right most column.

#	X	Y1	Y2	(phreatic) Yw
1	0.00	381.00	375.00	347.00
2	34.00	381.00	375.00	347.00
3	42.00	381.00	366.00	347.00
4	50.00	381.00	365.94	347.00
5	65.00	381.00	365.84	348.32
6	67.00	399.00	365.83	348.49
7	68.00	399.00	365.82	348.58
8	70.00	397.00	365.80	348.75
9	73.00	401.00	365.78	349.02
10	90.00	408.00	365.66	350.51
11	105.00	407.00	365.56	351.82
12	169.00	434.00	365.11	357.44
13	174.00	434.00	365.08	357.88
14	185.00	435.10	365.00	358.84
15	204.00	437.00	373.79	360.51
16	215.00	438.00	378.88	361.47
17	243.00	443.00	391.84	363.93
18	280.00	447.00	408.97	367.18
19	300.00	448.00	418.22	368.93
20	306.00	447.75	421.00	369.46
21	324.00	447.00	421.00	371.04
22	335.00	447.00	421.00	372.00

DISTRIBUTION OF AVAILABLE STRENGTH ALONG EACH REINFORCEMENT LAYER



A = Front-end of reinforcement (at face of slope)
B = Rear-end of reinforcement
AB = L1 + L2 + L3 = Embedded length of reinforcement

Tavailable = Long-term strength of reinforcement
Tfe = Available front-end strength (e.g., connection to facing)

L1 = Front-end 'pullout' length
L2 = Rear-end pullout length
Tavailable prevails along L3

Factor of safety on resistance to pullout on either end of reinforcement, Fs-po = 1.50

Reinforcement Layer #	Designated Name	Height Relative to Toe [ft]	L [ft]	L1 [ft]	L2 [ft]	L3 [ft]	Tfe [lb/ft]	Tavailable [lb/ft]
1	Miragrid 20XT	0.00	23.25	0.00	3.77	19.48	8240.14	8240.14
2	Miragrid 20XT	1.50	23.25	0.00	4.00	19.25	8240.14	8240.14
3	Miragrid 20XT	3.00	23.25	0.00	4.27	18.98	8240.14	8240.14
4	Miragrid 20XT	4.50	23.25	0.00	4.56	18.69	8240.14	8240.14
5	Miragrid 20XT	6.00	23.25	0.00	4.92	18.33	8240.14	8240.14
6	Miragrid 20XT	7.50	23.25	0.00	5.35	17.90	8240.14	8240.14
7	Miragrid 20XT	9.00	23.25	0.00	5.84	17.41	8240.14	8240.14
8	Miragrid 20XT	10.50	23.25	0.00	6.43	16.82	8240.14	8240.14
9	Miragrid 20XT	12.00	23.25	0.00	7.22	16.03	8240.14	8240.14
10	Miragrid 20XT	13.50	23.25	0.00	8.20	15.05	8240.14	8240.14
11	Miragrid 20XT	-2.00	25.00	0.00	3.41	21.59	8240.14	8240.14

RESULTS OF ROTATIONAL STABILITY ANALYSIS

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each entry point (considering all specified exit points)

Entry Point #	Entry Point X [ft]	Entry Point Y [ft]	Exit Point X [ft]	Exit Point Y [ft]	Critical Circle Xc [ft]	Critical Circle Yc [ft]	Critical Circle R [ft]	Fs	STATUS
1	65.00	381.00	65.00	381.00	65.00	381.00	0.00	N/A	#10 - Overhanging Cliff
2	65.00	381.00	65.00	381.00	65.00	381.00	0.00	N/A	#10 - Overhanging Cliff
3	65.00	381.00	65.00	381.00	65.00	381.00	0.00	N/A	#10 - Overhanging Cliff
4	65.00	381.00	65.00	381.00	65.00	381.00	0.00	N/A	#10 - Overhanging Cliff
5	65.00	381.00	65.00	381.00	65.00	381.00	0.00	N/A	#10 - Overhanging Cliff
6	65.00	381.00	65.00	381.00	65.00	381.00	0.00	N/A	#10 - Overhanging Cliff
7	65.00	381.00	65.00	381.00	65.00	381.00	0.00	N/A	#10 - Overhanging Cliff
8	65.00	381.00	65.00	381.00	65.00	381.00	0.00	N/A	#10 - Overhanging Cliff
9	94.52	407.70	48.47	381.07	63.59	408.05	30.93	19.37	
10	96.16	407.59	48.48	381.08	64.77	407.91	31.39	4.93	
11	97.80	407.48	48.48	381.08	65.95	407.71	31.85	2.92	. On extreme X-entry

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-entry' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

****

Results in the tables below represent critical circles identified between specified points on entry and exit. (Theta-exit set to 50.00 deg.)
The most critical circle is obtained from a search considering all the combinations of input entry and exit points.

Critical circles for each exit point (considering all specified entry points)

Exit Point #	Exit Point X [ft]	Exit Point Y [ft]	Entry Point X [ft]	Entry Point Y [ft]	Critical Circle Xc [ft]	Critical Circle Yc [ft]	Critical Circle R [ft]	Fs	STATUS
. 1	48.48	381.08	97.80	407.48	65.95	407.71	31.85	2.92	. On extreme X-exit
2	49.77	381.29	97.80	407.48	66.41	407.91	31.39	3.37	
3	51.43	381.26	97.80	407.48	67.19	407.51	30.61	3.53	
4	53.12	381.22	97.80	407.48	67.67	407.61	30.13	4.18	
5	54.84	381.16	97.80	407.48	68.16	407.64	29.64	4.86	
6	56.56	381.09	97.80	407.48	67.27	409.77	30.61	5.03	
7	58.39	381.02	97.80	407.48	68.83	408.06	28.98	4.04	
8	59.71	381.13	97.80	407.48	69.35	407.91	28.46	4.56	
9	61.60	381.04	97.80	407.48	69.88	407.70	27.92	5.11	
10	63.03	381.08	97.80	407.48	70.07	407.91	27.73	6.44	
11	64.95	381.01	97.80	407.48	70.65	407.56	27.15	7.00	

Note: In the 'Status' column, OK means the critical circle was identified within the specified search domain. 'On extreme X-exit' means that the critical result is on the edge of the search domain; a lower Fs may result if the search domain is expanded.

CRITICAL RESULTS OF ROTATIONAL AND TRANSLATIONAL STABILITY ANALYSES

Rotational (Circular Arc; Bishop) Stability Analysis

Minimum Factor of Safety = 2.92
Critical Circle: Xc = 65.95[ft], Yc = 407.71[ft], R = 31.85[ft]. (Number of slices used = 54)

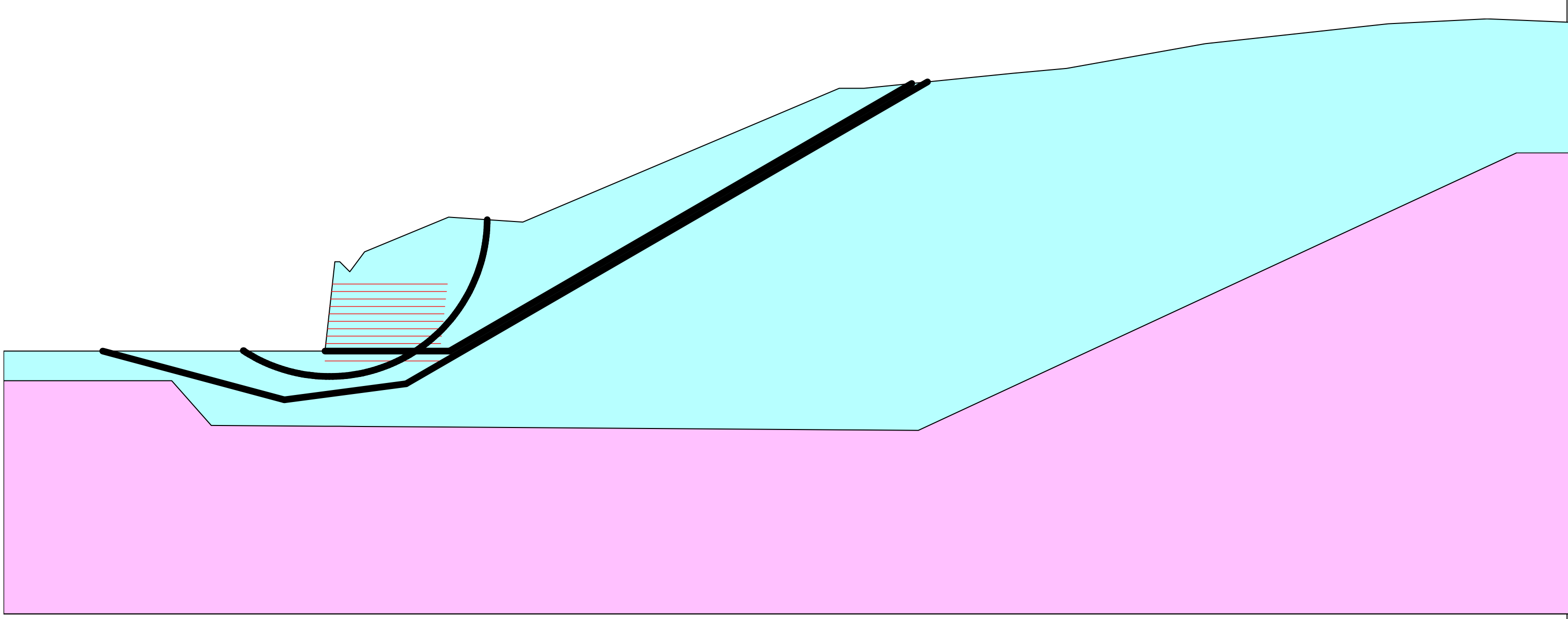
Translational (2-Part Wedge; Spencer), Direct Sliding, Stability Analysis

Minimum Factor of Safety = 1.18
Critical Two-Part Wedge: (Xa = 65.00, Ya = 381.00) [ft]
(Xb = 90.19, Yb = 381.00) [ft]
(Xc = 183.66, Yc = 434.97) [ft]
(Number of slices used = 30)
Interslice resultant force inclination = 23.91 [degrees]

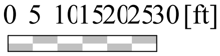
Three-Part Wedge Stability Analysis

Minimum Factor of Safety = 1.31
Critical Three-Part Wedge: (X2 = 20.04, Y2 = 381.00) [ft]
(X-left = 56.80, Y-left = 371.15) [ft]
(X-right = 81.40, Y-right = 374.40) [ft]
(X1 = 186.86, Y1 = 435.29) [ft]
(Number of slices used = 45)
Interslice resultant force inclination = 18.88 [degrees]

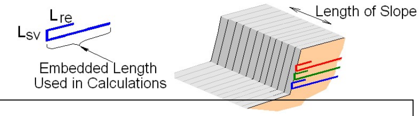
REINFORCEMENT LAYOUT: DRAWING



SCALE:



REINFORCEMENT LAYOUT: TABULATED DATA & QUANTITIES



Layer #	Reinf. Type #	Geosynthetic Designated Name	Height Relative to Toe [ft]	Embedded Length [ft]	Covergae Ratio, Rc	(X, Y) front [ft]		(X, Y) rear [ft]		Lsv * [ft]	Lre [ft]
1	1	Miragrid 20XT	0.00	23.25	1.00	213.25	1250.00	236.50	1250.00	0.00	0.00
2	1	Miragrid 20XT	1.50	23.25	1.00	213.42	1251.50	236.67	1251.50	0.00	0.00
3	1	Miragrid 20XT	3.00	23.25	1.00	213.59	1253.00	236.84	1253.00	0.00	0.00
4	1	Miragrid 20XT	4.50	23.25	1.00	213.75	1254.50	237.00	1254.50	0.00	0.00
5	1	Miragrid 20XT	6.00	23.25	1.00	213.92	1256.00	237.17	1256.00	0.00	0.00
6	1	Miragrid 20XT	7.50	23.25	1.00	214.09	1257.50	237.34	1257.50	0.00	0.00
7	1	Miragrid 20XT	9.00	23.25	1.00	214.25	1259.00	237.50	1259.00	0.00	0.00
8	1	Miragrid 20XT	10.50	23.25	1.00	214.42	1260.50	237.67	1260.50	0.00	0.00
9	1	Miragrid 20XT	12.00	23.25	1.00	214.59	1262.00	237.84	1262.00	0.00	0.00
10	1	Miragrid 20XT	13.50	23.25	1.00	214.75	1263.50	238.00	1263.50	0.00	0.00
11	1	Miragrid 20XT	-2.00	25.00	1.00	213.25	1248.00	238.25	1248.00	0.00	0.00

* Vertical distance between layers.

QUANTITIES

Reinf. Type #	Designated Name	Coverage Ratio	Area of reinforcemnt [ft²] / length of slope [ft]
1	Miragrid 20XT	1.00	257.50

APPENDIX I

STANDARD GUIDELINES FOR GRADING PROJECTS

TABLE OF CONTENTS

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GEOTECHNICAL GUIDELINES FOR GRADING PROJECTS

A. GENERAL

Al The guidelines contained herein and the standard details attached hereto represent this firm's standard recommendations for grading and other associated operations on construction projects. These guidelines should be considered a portion of the project specifications.

A2 All plates attached hereto shall be considered as part of these guidelines.

A3 The Contractor should not vary from these guidelines without prior recommendation by the Geotechnical Consultant and the approval of the Client or his authorized representative. Recommendations by the Geotechnical Consultant and/or Client should not be considered to preclude requirements for approval by the controlling agency prior to the execution of any changes.

A4 These Standard Grading Guidelines and Standard Details may be modified and/or superseded by recommendations contained in the text of the preliminary geotechnical report and/or subsequent reports.

A5 If disputes arise out of the interpretation of these grading guidelines or standard details, the Geotechnical Consultant shall provide the governing interpretation.

B. DEFINITIONS OF TERMS

Bl ALLUVIUM - unconsolidated detrital deposits resulting from flow of water, including sediments deposited in river beds, canyons, flood plains, lakes, fans at the foot of slopes and estuaries.

B2 AS-GRADED (AS-BUILT) - the surface and subsurface conditions at completion of grading.

B3 BACKCUT - a temporary construction slope at the rear of earth retaining structures such as buttresses, shear keys, stabilization fills or retaining walls.

B4 BACKDRAIN - generally a pipe and gravel or similar drainage system placed behind earth retaining structures such as buttresses, stabilization fills and retaining walls.

B5 BEDROCK - a more or less solid, relatively undisturbed rock in place either at the surface or beneath superficial deposits of soil.

B6 BENCH - a relatively level step and near vertical rise excavated into sloping ground on which fill is to be placed.

B7 BORROW (Import) - any fill material hauled to the project site from off-site areas.

B8 BUTTRESS FILL - a fill mass, the configuration of which is designed by engineering calculations to stabilize a slope exhibiting adverse geologic features. A buttress is generally specified by minimum key width and depth and by maximum backcut angle. A buttress normally contains a backdrain system.

B9 CIVIL ENGINEER - the Registered Civil Engineer or consulting firm responsible for preparation of the grading plans, surveying and verifying as-graded topographic conditions.

B10 CLIENT - the Developer or his authorized representative who is chiefly in charge of the project. He shall have the responsibility of reviewing the findings and recommendations made by the Geotechnical Consultant and shall authorize the Contractor and/or other consultants to perform work and/or provide services.

B11 COLLUVIUM - generally loose deposits usually found near the base of slopes and brought there chiefly by gravity through slow continuous downhill creep (also see Slope Wash).

B12 COMPACTION - is the densification of a fill by mechanical means.

B13 CONTRACTOR - a person or company under contract or otherwise retained by the Client to perform demolition, grading and other site improvements.

B14 DEBRIS - all products of clearing, grubbing, demolition, contaminated soil material unsuitable for reuse as compacted fill and/or any other material so designated by the Geotechnical Consultant.

B15 ENGINEERING GEOLOGIST - a Geologist holding a valid certificate of registration in the specialty of Engineering Geology.

B16 ENGINEERED FILL - a fill of which the Geotechnical Consultant or his representative, during grading, has made sufficient tests to enable him to conclude that the fill has been placed in substantial compliance with the recommendations of the Geotechnical Consultant and the governing agency requirements.

B17 EROSION - the wearing away of the ground surface as a result of the movement of wind, water and/or ice.

B18 EXCAVATION - the mechanical removal of earth materials.

B19 EXISTING GRADE - the ground surface configuration prior to grading.

B20 FILL - any deposits of soil, rock, soil-rock blends or other similar materials placed by man.

B21 FINISH GRADE - the ground surface configuration at which time the surface elevations conform to the approved plan.

B22 GEOFABRIC - any engineering textile utilized in geotechnical applications including subgrade stabilization and filtering.

B23 GEOLOGIST - a representative of the Geotechnical Consultant educated and trained in the field of geology.

B24 GEOTECHNICAL CONSULTANT - the Geotechnical Engineering and Engineering Geology consulting firm retained to provide technical services for the project. For the purpose of these guidelines, observations by the Geotechnical Consultant include observations by the Soil Engineer, Geotechnical Engineer, Engineering Geologist and those performed by persons employed by and responsible to the Geotechnical Consultants.

B25 GEOTECHNICAL ENGINEER - a licensed Civil Engineer who applies scientific methods, engineering principles and professional experience to the acquisition, interpretation and use of knowledge of materials of the earth's crust for the evaluation of engineering problems. Geotechnical Engineering encompasses many of the engineering aspects of soil mechanics, rock mechanics, geology, geophysics, hydrology and related sciences.

B26 GRADING - any operation consisting of excavation, filling or combinations thereof and associated operations.

B27 LANDSLIDE DEBRIS - material, generally porous and of low density, produced from instability of natural or man-made slopes.

B28 MAXIMUM DENSITY - standard laboratory test for maximum dry unit weight. Unless otherwise specified, the maximum dry unit weight shall be determined in accordance with ASTM Method of Test D 1557-78.

B29 OPTIMUM MOISTURE - test moisture content at the maximum density.

B30 RELATIVE COMPACTION - the degree of compaction (expressed as a percentage) of dry unit weight of a material as compared to the maximum dry unit weight of the material.

B31 ROUGH GRADE - the ground surface configuration at which time the surface elevations approximately conform to the approved plan.

B32 SITE - the particular parcel of land where grading is being performed.

B33 SHEAR KEY - similar to buttress, however, it is generally constructed by excavating a slot within a natural slope in order to stabilize the upper portion of the slope without grading encroaching into the lower portion of the slope.

B34 SLOPE - is an inclined ground surface the steepness of which is generally specified as a ratio of horizontal:vertical (e.g., 2:1).

B35 SLOPE WASH - soil and/or rock material that has been transported down a slope by mass wasting assisted by runoff water not confined by channels (also see Colluvium).

B36 SOIL - naturally occurring deposits of sand, silt, clay, etc. or combinations thereof.

B37 SOIL ENGINEER - licensed Civil Engineer experienced in soil mechanics (also see Geotechnical Engineer).

B38 STABILIZATION FILL - a fill mass, the configuration of which is typically related to slope height and is specified by the standards of practice for enhancing the stability of locally adverse conditions. A stabilization fill is normally specified by minimum key width and depth and by maximum backcut angle. A stabilization fill may or may not have a backdrain system specified.

B39 SUBDRAIN - generally a pipe and gravel or similar drainage system placed beneath a fill in the alignment of canyons or former drainage channels.

B40 SLOUGH - loose, noncompacted fill material generated during grading operations.

B41 TAILINGS - non-engineered fill which accumulates on or adjacent to equipment haul-roads.

B42 TERRACE - relatively level step constructed in the face of a graded slope surface for drainage control and maintenance purposes.

B43 TOPSOIL - the presumably fertile upper zone of soil which is usually darker in color and loose.

B44 WINDROW - a string of large rock buried within engineered fill in accordance with guidelines set forth by the Geotechnical Consultant.

C. OBLIGATIONS OF PARTIES

C1 The Geotechnical Consultant should provide observation and testing services and should make evaluations to advise the Client on geotechnical matters. The geotechnical Consultant should report his findings and recommendations to the Client or his authorized representative.

C2 The Client should be chiefly responsible for all aspects of the project. He or his authorized representative has the responsibility of reviewing the findings and recommendations of the Geotechnical Consultant. He shall authorize or cause to have authorized the Contractor and/or other consultants to perform work and/or provide services. During grading the Client or his authorized representative should remain on-site or should remain reasonably accessible to all concerned parties in order to make decisions necessary to maintain the flow of the project.

C3 The Contractor should be responsible for the safety of the project and satisfactory completion of all grading and other associated operations on construction projects, including, but not limited to, earth work in accordance with the project plans, specifications and controlling agency requirements. During grading, the Contractor or his authorized representative should remain on-site. Overnight and on days off, the Contractor should remain accessible.

D. SITE PREPARATION

D1 The Client, prior to any site preparation or grading, should arrange and attend a meeting among the Grading Contractor, the Design Engineer, the Geotechnical Consultant, representatives of the appropriate governing authorities as well as any other concerned parties. All parties should be given at least 48 hours notice.

D2 Clearing and grubbing should consist of the removal of vegetation such as brush, grass, woods, stumps, trees, roots of trees and otherwise deleterious natural materials from the areas to be graded. Clearing and grubbing should extend to the outside of all proposed excavation and fill areas.

D3 Demolition should include removal of buildings, structures, foundations, reservoirs, utilities (including underground pipelines, septic tanks, leach fields, seepage pits, cisterns, mining shafts, tunnels, etc.) and other man-made surface and subsurface improvements from the areas to be graded. Demolition of utilities should include proper capping and/or rerouting pipelines at the project perimeter and cutoff and capping of wells in accordance with the requirements of the governing authorities and the recommendations of the Geotechnical Consultant at the time of demolition.

D4 Trees, plants or man-made improvements not planned to be removed or demolished should be protected by the Contractor from damage.

D5 Debris generated during clearing, grubbing and/or demolition operations should be wasted from areas to be graded and disposed off-site. Clearing, grubbing and demolition operations should be performed under the observation of the Geotechnical Consultant.

D6 The Client or Contractor should obtain the required approvals from the controlling authorities for the project prior, during and/or after demolition, site preparation and removals, etc. The appropriate approvals should be obtained prior to proceeding with grading operations.

E SITE PROTECTION

El Protection of the site during the period of grading should be the responsibility of the Contractor. Unless other provisions are made in writing and agreed upon among the concerned parties, completion of a portion of the project should not be considered to preclude that portion or adjacent areas from the requirements for site protection until such time as the entire project is complete as identified by the Geotechnical Consultant, the Client and the regulating agencies.

E2 The Contractor should be responsible for the stability of all temporary excavations. Recommendations by the Geotechnical Consultant pertaining to temporary excavations (e.g., backcuts) are made in consideration of stability of the completed project and, therefore, should not be considered to preclude the responsibilities of the Contractor. Recommendations by the Geotechnical Consultant should not be considered to preclude more restrictive requirements by the regulating agencies.

E3 Precautions should be taken during the performance of site clearing, excavations and grading to protect the work site from flooding, ponding or inundation by poor or improper surface drainage. Temporary provisions should be made during the rainy season to adequately direct surface drainage away from and off the work site. Where low areas cannot be avoided, pumps should be kept on hand to continually remove water during periods of rainfall.

E4 During periods of rainfall, plastic sheeting should be kept reasonably accessible to prevent unprotected slopes from becoming saturated. Where necessary during periods of rainfall, the Contractor should install checkdams, desilting basins, rip-rap, sand bags or other devices or methods necessary to control erosion and provide safe conditions.

E5 During periods of rainfall, the Geotechnical Consultant should be kept informed by the Contractor as to the nature of remedial or preventative work being performed (e.g., pumping, placement of sandbags or plastic sheeting, other labor, dozing, etc.).

E6 Following periods of rainfall, the Contractor should contact the Geotechnical Consultant and arrange a walkover of the site in order to visually assess rain related damage. The Geotechnical Consultant may also recommend excavations and testing in order to aid in his assessments. At the request of the Geotechnical Consultant, the Contractor shall make excavations in order to evaluate the extent of rain related-damage.

E7 Rain-related damage should be considered to include, but may not be limited to, erosion, silting, saturation, swelling, structural distress and other adverse conditions identified by the Geotechnical Consultant. Soil adversely affected should be classified as Unsuitable Materials and should be subject to overexcavation and replacement with compacted fill or other remedial grading as recommended by the Geotechnical Consultant.

E8 Relatively level areas, where saturated soils and/or erosion gullies exist to depths of greater than 1.0 foot, should be overexcavated to unaffected, competent material. Where less than 1.0 foot in depth, unsuitable materials may be processed in-place to achieve near-optimum moisture conditions, then thoroughly recompacted in accordance with the applicable specifications. If the desired results are not achieved, the affected materials should be overexcavated, then replaced in accordance with the applicable specifications.

E9 In slope areas, where saturated soil and/or erosion gullies exist to depths of greater than 1.0 foot, they should be overexcavated and replaced as compacted fill in accordance with the applicable specifications. Where affected materials exist to depths of 1.0 foot or less below proposed finished grade, remedial grading by moisture conditioning in-place, followed by thorough recompaction in accordance with the applicable grading guidelines herein may be attempted. If the desired results are not achieved, all affected materials should be overexcavated and replaced as compacted fill in accordance with the slope repair recommendations herein. As field conditions dictate, other slope repair procedures may be recommended by the Geotechnical Consultant.

F. EXCAVATIONS

F1 UNSUITABLE MATERIALS

F1.1 Materials which are unsuitable should be excavated under observation and recommendations of the Geotechnical Consultant. Unsuitable materials include, but may not be limited to, dry, loose, soft, wet, organic compressible natural soils and fractured, weathered, soft bedrock and nonengineered or otherwise deleterious fill materials.

F1.2 Material identified by the Geotechnical Consultant as unsatisfactory due to it's moisture condition should be overexcavated, watered or dried, as needed, and thoroughly blended to a uniform near optimum moisture condition prior to placement as compacted fill.

F2 CUT SLOPES

F2.1 Unless otherwise recommended by the Geotechnical Consultant and approved by the regulating agencies, permanent cut slopes should not be steeper than 2:1 (horizontal: vertical).

F2.2 If excavations for cut slopes expose loose, cohesionless, significantly fractured or otherwise unsuitable material, overexcavation and replacement of the unsuitable materials with a compacted stabilization fill should be accomplished as recommended by the Geotechnical Consultant. Unless otherwise specified by the Geotechnical Consultant, stabilization fill construction should conform to the requirements of the Standard Details.

F2.3 The Geotechnical Consultant should review cut slopes during excavation. The Geotechnical Consultant should be notified by the contractor prior to beginning slope excavations.

F2.4 If, during the course of grading, adverse or potentially adverse geotechnical conditions are encountered which were not anticipated in the preliminary report, the Geotechnical Consultant should explore, analyze and make recommendations to treat these problems.

F2.5 When cut slopes are made in the direction of the prevailing drainage, a non-erodible diversion swale (brow ditch) should be provided at the top-of-cut.

F3 PAD AREAS

F3.1 All lot pad areas, including side yard terraces, above stabilization fill or buttresses should be overexcavated to provide for a minimum of 3 feet (refer to Standard Details) of compacted fill over the entire pad area. Pad areas with both fill and cut materials exposed and pad areas containing both very shallow (less than 3 feet) and deeper fill should be overexcavated to provide for a uniform compacted fill blanket with a minimum of 3 feet in thickness (refer to Standard Details). Cut areas exposing significantly varying material types should also be overexcavated to provide for at least a 3-foot thick compacted fill blanket. Geotechnical conditions may require greater depth of overexcavation. The actual depth should be delineated by the Geotechnical Consultant during grading.

F3.2 For pad areas created above cut or natural slopes, positive drainage should be established away from the top-of-slope. This may be accomplished utilizing a berm and/or an appropriate pad gradient. A gradient in soil areas away from the top-of-slopes of 2 percent or greater is recommended.

G. COMPACTED FILL

All fill materials should be compacted as specified below or by other methods specifically recommended by the Geotechnical Consultant. Unless otherwise specified, the minimum degree of compaction (relative compaction) should be 90 percent of the laboratory maximum density.

G1 PLACEMENT

G1.1 Prior to placement of compacted fill, the Contractor should request a review by the Geotechnical Consultant of the exposed ground surface. Unless otherwise recommended, the exposed ground surface should then be scarified (six inches minimum), watered or dried as needed, thoroughly blended to achieve near optimum moisture conditions, then thoroughly compacted to a minimum of 90 percent of the maximum density. The review by the Geotechnical Consultant should not be considered to preclude requirement of inspection and approval by the governing agency.

G1.2 Compacted fill should be placed in thin horizontal lifts not exceeding eight inches in loose thickness prior to compaction. Each lift should be watered or dried as needed, thoroughly blended to achieve near optimum moisture conditions then thoroughly compacted by mechanical methods to a minimum of 90 percent of laboratory maximum dry density. Each lift should be treated in a like manner until the desired finished grades are achieved.

G1.3 The Contractor should have suitable and sufficient mechanical compaction equipment and watering apparatus on the job site to handle the amount of fill being placed in consideration of moisture retention properties of the materials. If necessary, excavation equipment should be "shut down" temporarily in order to permit proper compaction of fills. Earth moving equipment should only be considered a supplement and not substituted for conventional compaction equipment.

G1.4 When placing fill in horizontal lifts adjacent to areas sloping steeper than 5:1 (horizontal: vertical), horizontal keys and vertical benches should be excavated into the adjacent slope area. Keying and benching should be sufficient to provide at least six-foot wide benches and a minimum of four feet of vertical bench height within the firm natural ground, firm bedrock or engineered compacted fill. No compacted fill should be placed in an area subsequent to keying and benching until the area has been reviewed by the Geotechnical Consultant. Material generated by the benching operation should be moved sufficiently away from the bench area to allow for the recommended review of the horizontal bench prior to placement of fill. Typical keying and benching details have been included within the accompanying Standard Details.

G1.5 Within a single fill area where grading procedures dictate two or more separate fills, temporary slopes (false slopes) may be created. When placing fill adjacent to a false slope, benching should be conducted in the same manner as above described. At least a 3-foot vertical bench should be established within the firm core of adjacent approved compacted fill prior to placement of additional fill. Benching should proceed in at least 3-foot vertical increments until the desired finished grades are achieved.

G1.6 Fill should be tested for compliance with the recommended relative compaction and moisture conditions. Field density testing should conform to ASTM Method of Test D1556-64, D2922-78 and/or D2937-71. Tests should be provided for about every two vertical feet or 1,000 cubic yards of fill placed. Actual test interval may vary as field conditions dictate. Fill found not to be in conformance with the grading recommendations should be removed or otherwise handled as recommended by the Geotechnical Consultant.

G1.7 The Contractor should assist the Geotechnical Consultant and/or his representative by digging test pits for removal determinations and/or for testing compacted fill.

G1.8 As recommended by the Geotechnical Consultant, the Contractor should "shut down" or remove grading equipment from an area being tested.

G1.9 The Geotechnical Consultant should maintain a plan with estimated locations of field tests. Unless the client provides for actual surveying of test locations, the estimated locations by the Geotechnical Consultant should only be considered rough estimates and should not be utilized for the purpose of preparing cross sections showing test locations or in any case for the purpose of after-the-fact evaluating of the sequence of fill placement.

G2 MOISTURE

G2.1 For field testing purposes, "near optimum" moisture will vary with material type and other factors including compaction procedure. "Near optimum" may be specifically recommended in Preliminary Investigation Reports and/or may be evaluated during grading. As a preliminary guideline "near optimum" should be considered from one percent below to three percent above optimum.

G2.2 Prior to placement of additional compacted fill following an overnight or other grading delay, the exposed surface or previously compacted fill should be processed by scarification, watered or dried as needed, thoroughly blended to near-optimum moisture conditions, then recompacted to a minimum of 90 percent of laboratory maximum dry density. Where wet or other dry or other unsuitable materials exist to depths of greater than one foot, the unsuitable materials should be over excavated.

G2.3 Following a period of flooding, rainfall or overwatering by other means, no additional fill should be placed until damage assessments have been made and remedial grading performed as described under Section E6 herein.

G3 FILL MATERIAL

G3.1 Excavated on-site materials which are acceptable to the Geotechnical Consultant may be utilized as compacted fill, provided trash, vegetation and other deleterious materials are removed prior to placement.

G3.2 Where import materials are required for use on-site, the Geotechnical Consultant should be notified at least 72 hours in advance of importing, in order to sample and test materials from proposed borrow sites. No import materials should be delivered for use on-site without prior sampling and testing by the Geotechnical Consultant.

G3.3 Where oversized rock or similar irreducible material is generated during grading, it is recommended, where practical, to waste such material off-site or on-site in areas designated as "nonstructural rock disposal areas". Rock placed in disposal areas should be placed with sufficient fines to fill voids. The rock should be compacted in lifts to an unyielding condition. The disposal area should be covered with at least three feet of compacted fill which is free of oversized material. The upper three feet should be placed in accordance with the guidelines for compacted fill herein.

G3.4 Rocks 12 inches in maximum dimension and smaller may be utilized within the compacted fill, provided they are placed in such a manner that nesting of the rock is avoided. Fill should be placed and thoroughly compacted over and around all rock. The amount of rock should not exceed 40 percent by dry weight passing the 3/4-inch sieve size. The 12-inch and 40 percent recommendations herein may vary as field conditions dictate.

G3.5 During the course of grading operations, rocks or similar irreducible materials greater than 12 inches maximum dimension (oversized material), may be generated. These rocks should not be placed within the compacted fill unless placed as recommended by the Geotechnical Consultant.

G3.6 Where rocks or similar irreducible materials of greater than 12 inches but less than four feet of maximum dimension are generated during grading, or otherwise desired to be placed within an engineered fill, special handling in accordance with the accompanying Standard Details is recommended. Rocks greater than four feet should be broken down or disposed off-site. Rocks up to four feet maximum dimension should not be placed in the upper 10 feet of any fill and should not be closer than 20 feet to any slope face. These recommendations could vary as locations of improvements dictate.

Where practical, oversized material should not be placed below areas where structures or deep utilities are proposed. Oversized material should be placed in windrows on a clean, overexcavated or unyielding compacted fill or firm natural ground surface. Select native or imported granular soil (S.E. 30 or higher) should be placed and thoroughly flooded over and around all windrowed rock, such that voids are filled. Windrows of oversized material should be staggered so that successive strata of oversized material are not in the same vertical plane.

The Contractor should be aware that the placement of rock in windrows will significantly slow the grading operation and may require additional equipment and/or special equipment.

G3.7 It may be possible to dispose of individual larger rock as field conditions dictate and as recommended by the Geotechnical Consultant at the time of placement.

G3.8 Material that is considered unsuitable by the Geotechnical Consultant should not be utilized in the compacted fill.

G3.9 During grading operations, placing and mixing the materials from the cut and/or borrow areas may result in soil mixtures which possess unique physical properties. Testing may be required of samples obtained directly from the fill areas in order to verify conformance with the specifications. Processing of these additional samples may take two or more working days. The contractor may elect to move the operation to other areas within the project, or may continue placing compacted fill pending laboratory and field test results. Should he elect the second alternative, fill placed is done so at the Contractor's risk.

G3.10 Any fill placed in areas not previously reviewed and evaluated by the Geotechnical Consultant, and/or in other areas, without prior notification to the Geotechnical Consultant may require removal and recompaction at the Contractor's expense. Determination of overexcavations should be made upon review of field conditions by the Geotechnical Consultant.

G4 FILL SLOPES

G4.1 Unless otherwise recommended by the Geotechnical Consultant and approved by the regulating agencies, permanent fill slopes should not be steeper than 2:1 (horizontal: vertical).

G4.2 Except as specifically recommended otherwise or as otherwise provided for in these grading guidelines (Reference G4.3), compacted fill slopes should be overbuilt and cut back to grade, exposing the firm, compacted fill inner core. The actual amount of overbuilding may vary as field conditions dictate. If the desired results are not achieved, the existing slopes should be overexcavated and reconstructed under the guidelines of the Geotechnical Consultant. The degree of overbuilding shall be increased until the desired compacted slope surface condition is achieved. Care should be taken by the Contractor to provide thorough mechanical compaction to the outer edge of the overbuilt slope surface.

G4.3 Although no construction procedure produces a slope free from risk of future movement, overfilling and cutting back of slope to a compacted inner core is, given no other constraints, the most desirable procedure. Other constraints, however, must often be considered. These constraints may include property line situations, access, the critical nature of the development and cost. Where such constraints are identified, slope face compaction on slopes of 2:1 or flatter may be attempted as a second-best alternative by conventional construction procedures including backrolling techniques upon specific recommendation by the Geotechnical Consultant.

Fill placement should proceed in thin lifts, (i.e., six to eight-inch loose thickness). Each lift should be moisture conditioned and thoroughly compacted. The desired moisture condition should be maintained and/or re-established, where necessary, during the period between successive lifts. Selected lifts should be tested to ascertain that desired compaction is being achieved. Care should be taken to extend compactive effort to the outer edge of the slope.

Each lift should extend horizontally to the desired finished slope surface or more as needed to ultimately establish desired grades. Grade during construction should not be allowed to roll off at the edge of the slope. It may be helpful to elevate slightly the outer edge of the slope. Slough resulting from the placement of individual lifts should not be allowed to drift down over previous lifts. At intervals not exceeding four feet in vertical slope height or the capability of available equipment, whichever is less, fill slopes should be thoroughly backrolled utilizing a conventional sheepsfoot-type roller. Care should be taken to maintain the desired moisture conditions and/or reestablishing same as needed prior to backrolling. Upon achieving final grade, the slopes should again be moisture conditioned and thoroughly backrolled. The use of a side-boom roller will probably be necessary and vibratory methods are strongly recommended. Without delay, so as to avoid (if possible) further moisture conditioning, the slopes should then be grid-rolled to achieve a relatively smooth surface and uniformly compact condition.

In order to monitor slope construction procedures, moisture and density tests should be taken at regular intervals. Failure to achieve the desired results will likely result in a recommendation by the Geotechnical Consultant to overexcavate the slope surfaces followed by reconstruction of the slopes utilizing over-filling and cutting back procedures and/or further attempt at the conventional backrolling approach. Other recommendations may also be provided which would be commensurate with field conditions.

G4.4 Where placement of fill above a natural slope or above a cut slope is proposed, the fill slope configuration as presented in the accompanying Standard Details should be adopted.

G4.5 For pad areas above fill slopes, positive drainage should be established away from the top-of-slope. This may be accomplished utilizing a berm and pad gradients of at least 2 percent in soil areas.

G5 OFF-SITE FILL

G5.1 Off-site fill should be treated in the same manner as recommended in these specifications for site preparation, excavation, drains, compaction, etc.

G5.2 Off-site canyon fill should be placed in preparation for future additional fill, as shown in the accompanying Standard Details.

G5.3 Off-site fill subdrains temporarily terminated (up canyon) should be surveyed for future relocation and connection.

H. DRAINAGE

H1 Canyon subdrain systems specified by the Geotechnical Consultant should be installed in accordance with the Standard Details.

H2 Typical subdrains for compacted fill buttresses, slope stabilizations or sidehill masses, should be installed in accordance with the specifications of the accompanying Standard Details.

H3 Roof, pad and slope drainage should be directed away from slopes and areas of structures to suitable disposal areas via non-erodible devices (i.e., gutters, downspouts, concrete swales).

H4 For drainage over soil areas immediately away from structures, (i.e., within four feet) a minimum of 5 percent gradient should be maintained. Pad drainage of at least 2 percent should be maintained over soil areas.

H5 Drainage patterns established at the time of fine grading should be maintained throughout the life of the project. Property owners should be made aware that altering drainage patterns can be detrimental to slope stability and foundation performance.

GEOTECHNICAL GUIDELINE FOR GRADING PROJECTS

I STAKING

I1 In all fill areas, the fill should be compacted prior to the placement of the stakes. This particularly is important on fill slopes. Slope stakes should not be placed until the slope is thoroughly compacted (backrolled). If stakes must be placed prior to the completion of compaction procedures, it must be recognized that they will be removed and/or demolished at such time as compaction procedures resume.

12 In order to allow for remedial grading operations, which could include overexcavations or slope stabilization, appropriate staking offsets should be provided. For finished slope and stabilization backcut areas, we recommend at least a 10-foot setback from proposed toes and tops-of-cut.

J. MAINTENANCE

J1 LANDSCAPE PLANTS

In order to enhance surficial slope stability, slope planting should be accomplished at the completion of grading. Slope planting should consist of deep-rooting vegetation requiring little watering. Plants native to the southern California area and plants relative to native plants are generally desirable. Plants native to other semi-arid and arid areas may also be appropriate. A Landscape Architect would be the best party to consult regarding actual types of plants and planting configuration.

J2 IRRIGATION

J2.1 Irrigation pipes should be anchored to slope faces, not placed in trenches excavated into slope faces.

J2.2 Slope irrigation should be minimized. If automatic timing devices are utilized on irrigation systems, provisions should be made for interrupting normal irrigation during periods of rainfall.

J2.3 Though not a requirement, consideration should be given to the installation of near-surface moisture monitoring control devices. Such devices can aid in the maintenance of relatively uniform and reasonably constant moisture conditions.

J2.4 Property owners should be made aware that overwatering of slopes is detrimental to slope stability.

J3 MAINTENANCE

J3.1 Periodic inspections of landscaped slope areas should be planned and appropriate measures should be taken to control weeds and enhance growth of the landscape plants. Some areas may require occasional replanting and/or reseeding.

J3.2 Terrace drains and downdrains should be periodically inspected and maintained free of debris. Damage to drainage improvements should be repaired immediately.

J3.3 Property owners should be made aware that burrowing animals can be detrimental to slope stability. A preventative program should be established to control burrowing animals.

J3.4 As a precautionary measure, plastic sheeting should be readily available, or kept on hand, to protect all slope areas from saturation by periods of heavy or prolonged rainfall. This measure is strongly recommended, beginning with the period of time prior to landscape planting.

J4 REPAIRS

J4.1 If slope failures occur, the Geotechnical Consultant should be contacted for a field review of site conditions and development of recommendations for evaluation and repair.

J4.2 If slope failures occur as a result of exposure to periods of heavy rainfall, the failure area and currently unaffected areas should be covered with plastic sheeting to protect against additional saturation.

J4.3 In the accompanying Standard Details, appropriate repair procedures are illustrated for superficial slope failures (i.e., occurring typically within the outer one foot to three feet of a slope face).

K. TRENCH BACKFILL

K1 Utility trench backfill should, unless otherwise recommended, be compacted by mechanical means. Unless otherwise recommended, the degree of compaction should be a minimum of 90 percent of the laboratory maximum density.

K2 As an alternative, granular material (sand equivalent greater than 30) may be thoroughly jetted in-place. Jetting should only be considered to apply to trenches no greater than two feet in width and four feet in depth. Following jetting operations, trench backfill should be thoroughly mechanically compacted and/or wheel rolled from the surface.

K3 Backfill of exterior and interior trenches extending below a 1:1 projection from the outer edge of foundations should be mechanically compacted to a minimum of 90 percent of the laboratory maximum density.

K4 Within slab areas, but outside the influence of foundations, trenches up to one foot wide and two feet deep may be backfilled with sand and consolidated by jetting, flooding or by mechanical means. If on-site materials are utilized, they should be wheel-rolled, tamped or otherwise compacted to a firm condition. For minor interior trenches, density testing may be deleted or spot testing may be elected if deemed necessary, based on review of backfill operations during construction.

K5 If utility contractors indicate that it is undesirable to use compaction equipment in close proximity to a buried-conduit, the Contractor may elect the utilization of light weight mechanical compaction equipment and/or shading of the conduit with clean, granular material, which should be thoroughly jetted in-place above the conduit, prior to initiating mechanical compaction procedures. Other methods of utility trench compaction may also be appropriate, upon review by the Geotechnical Consultant at the time of-construction.

K6 In cases where clean granular materials are proposed for use in lieu of native materials or where flooding or jetting is proposed, the procedures should be considered subject to review by the Geotechnical Consultant.

K7 Clean granular backfill and/or bedding are not recommended in slope areas unless provisions are made for a drainage system to mitigate the potential build-up of seepage forces.

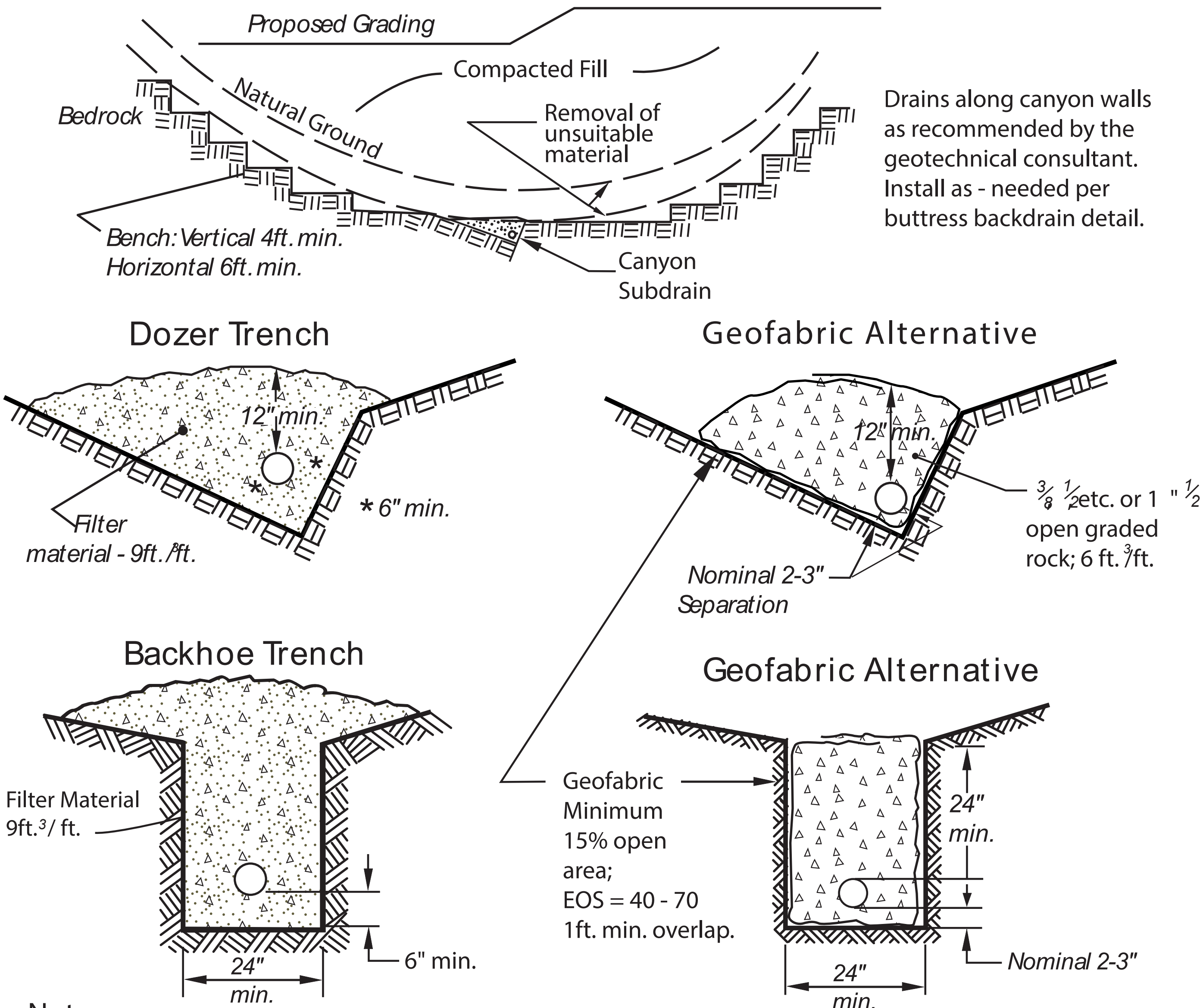
L **STATUS OF GRADING**

Prior to proceeding with any grading operation, the Geotechnical Consultant should be notified at least two working days in advance in order to schedule the necessary observation and testing services.

L1 Prior to any significant expansion or cut back in the grading operation, the Geotechnical Consultant should be provided with adequate notice (i.e., two days) in order to make appropriate adjustments in observation and testing services.

L2 Following completion of grading operations and/or between phases of a grading operation, the Geotechnical Consultant should be provided with at least two working days notice in advance of commencement of additional grading operations.

CANYON SUBDRAIN



Notes:

1- Pipe be 4" min. diameter, 6" min. for runs of 500ft to 1000ft, 8" min. for runs of 1000ft. or greater.

2- Pipe should be schedule 40 PVC or similiar. Upstream ends should be capped.

3- Pipe should have 8 uniformly spaced 3/8" perforations per foot placed at 90 offset on underside of pipe. FInal 20 foot of pipe should be nonperforated.

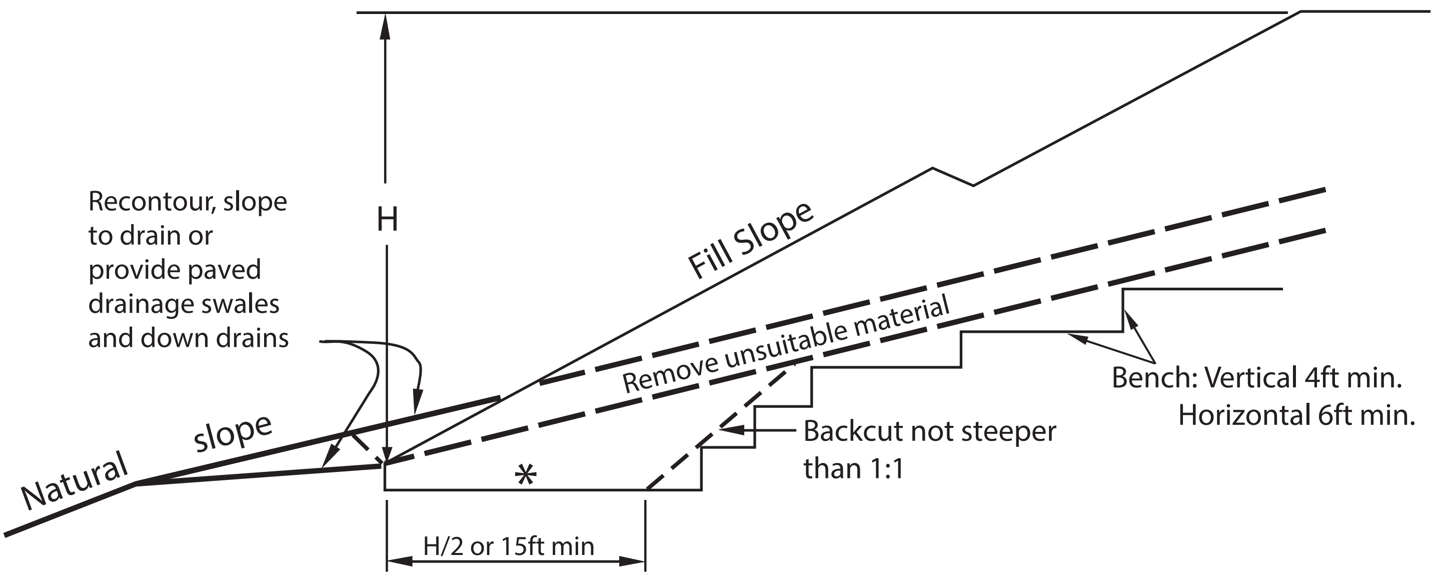
4- Filter material should be Calif. Class 2 Permeable Material.

5- Appropriate gradient should be provided for drainage; 2% minimum is recommended.

6- For the Geofabric Alternatives and gradients of 4% or greater, pipe may be omitted from the upper 500ft. For runs of 500, 1000, and 1500ft or greater 4", 6", and 8" pipe, respectively, should be provided.

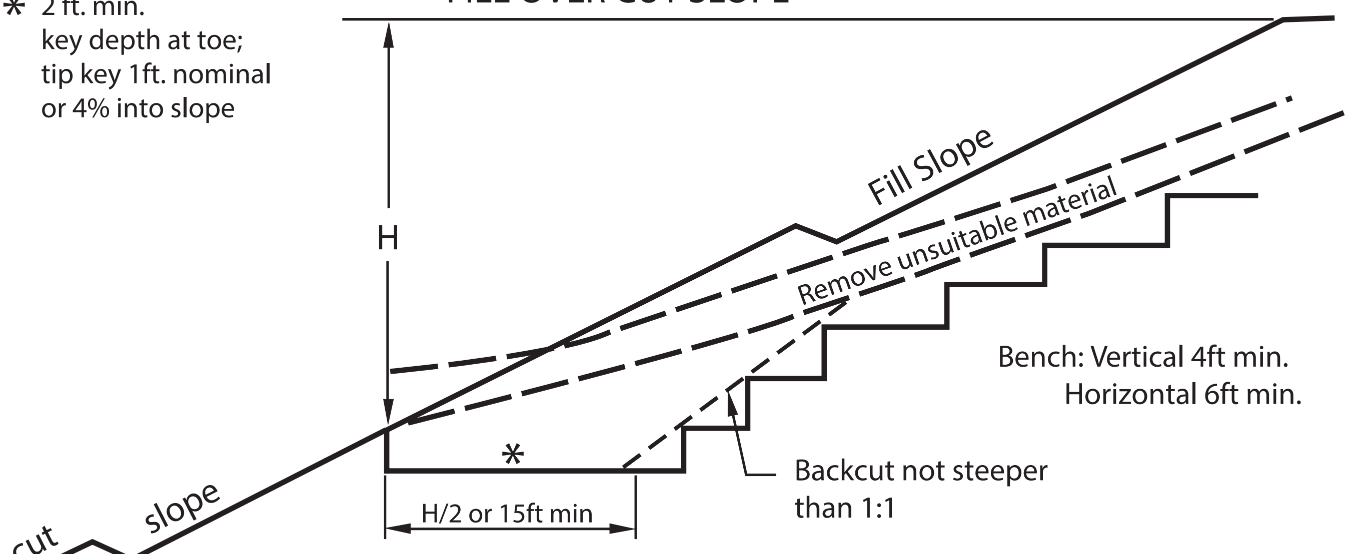
STANDARD DETAIL NO. 1

FILL OVER NATURAL SLOPE



FILL OVER CUT SLOPE

* 2 ft. min. key depth at toe; tip key 1ft. nominal or 4% into slope



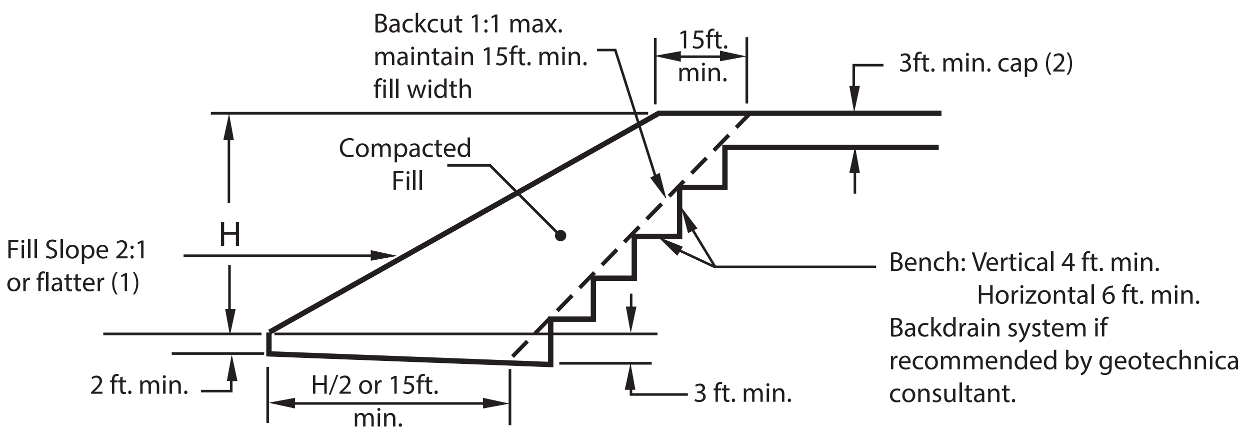
Notes:

1 - If overfilling and cutting back to grade is adopted, 15ft. min. fill width may be reduced to 12ft. min. In no case should the fill width be less than 1/2 the height of fill remaining.

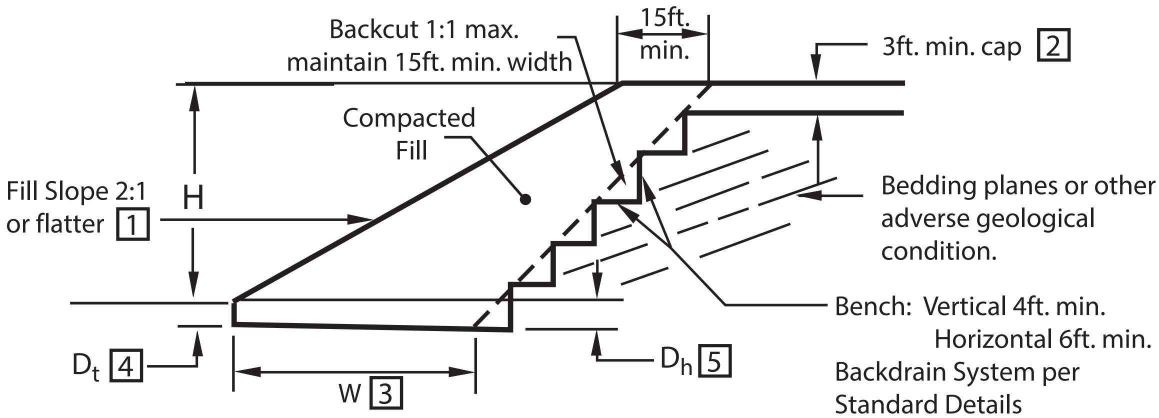
2 - Backdrain as recommended by geotechnical consultant per buttress backdrain detail.

STANDARD DETAIL NO. 2

STABILIZATION FILL



BUTTRESS FILL



Notes:

$\boxed{1}$- If overfilling and cutting back to grade is adopted, 15ft. may be reduced to 12ft. In no case should the fill width be less than 1/2 the fill height remaining.

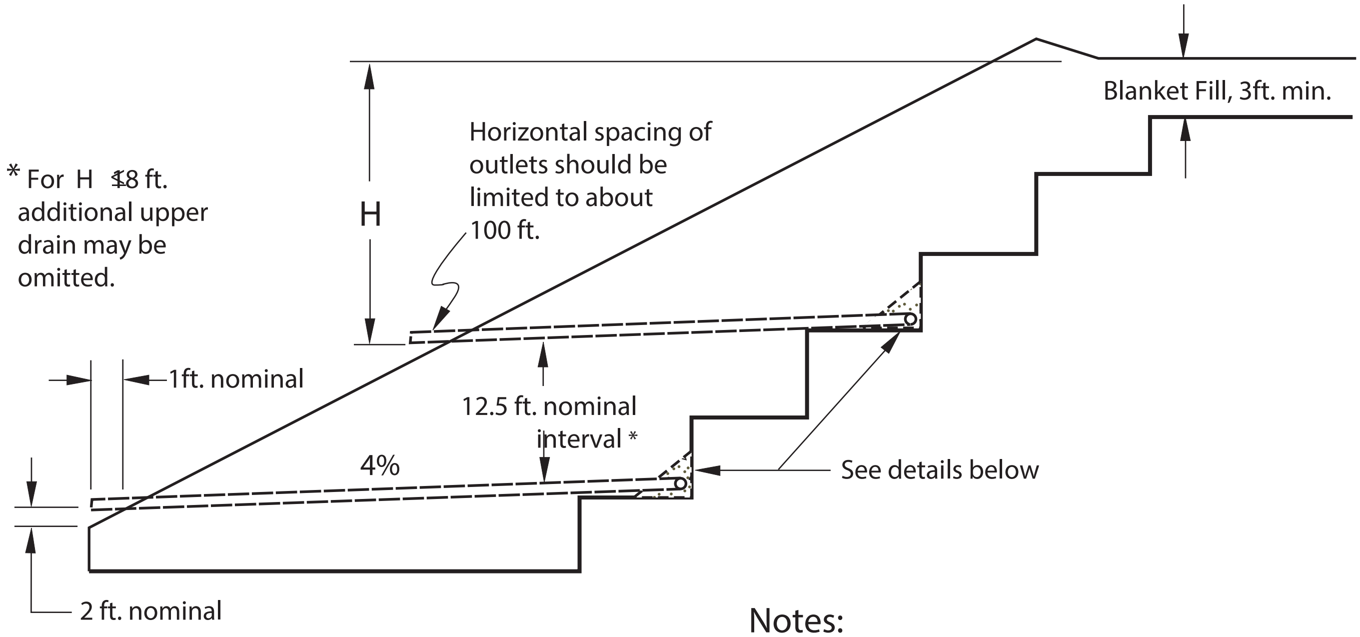
$\boxed{2}$- A 3ft. blanket fill shall be provided above stabilization and buttress fills. The thickness may be greater as recommended by the geotechnical consultant.

$\boxed{3}$- W = designed width of key.

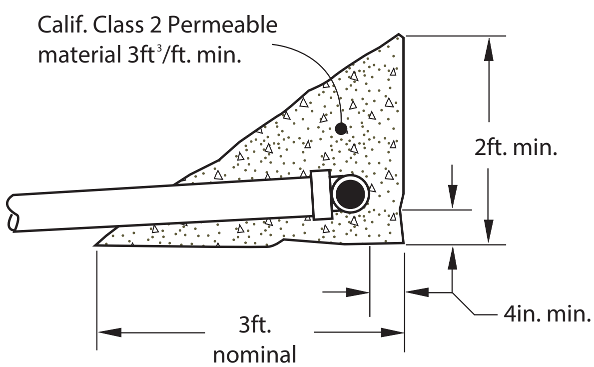
$\boxed{4}$- D_t = designed depth of key at toe

$\boxed{5}$- D_h = depth of key at heel; unless otherwise specified, $D_h = D_t + 1$ft.

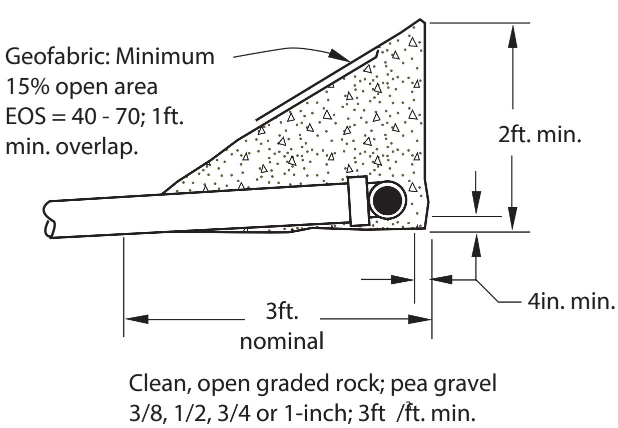
STABILIZATION FILL



Conventional Backdrain



Geofrabic Alternative



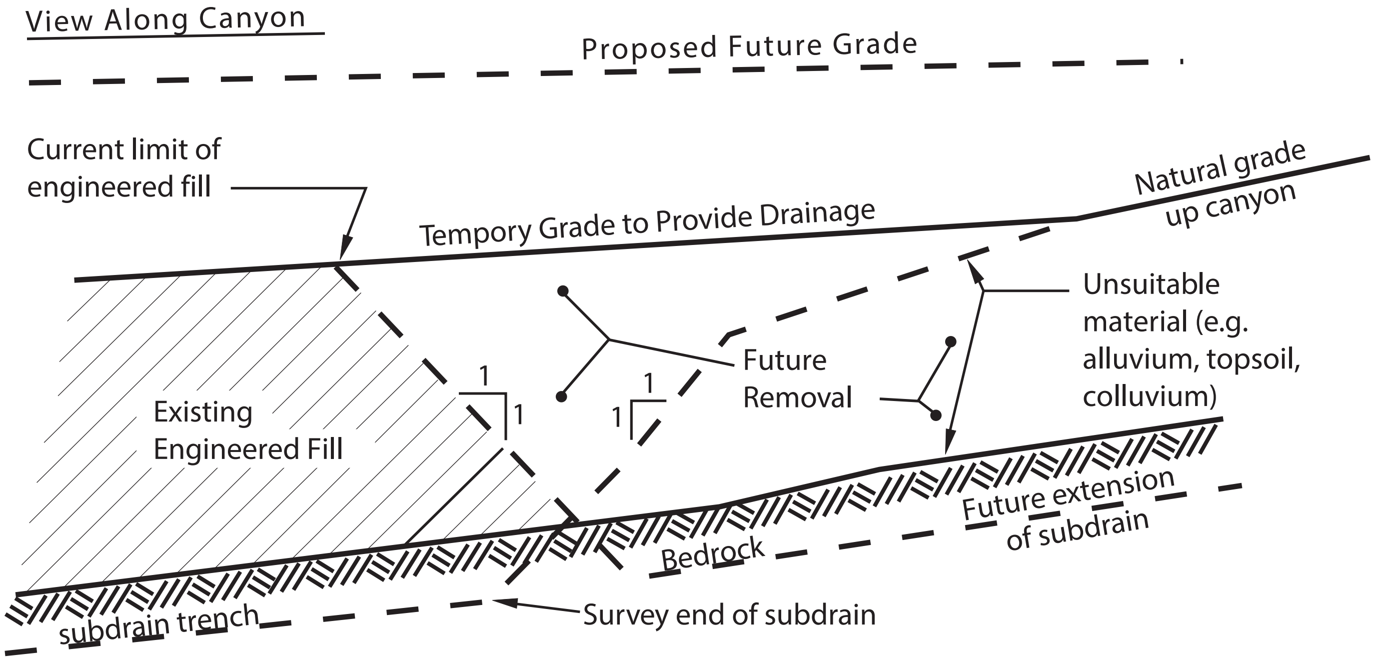
Clean, open graded rock; pea gravel
3/8, 1/2, 3/4 or 1-inch; 3ft³/ft. min.

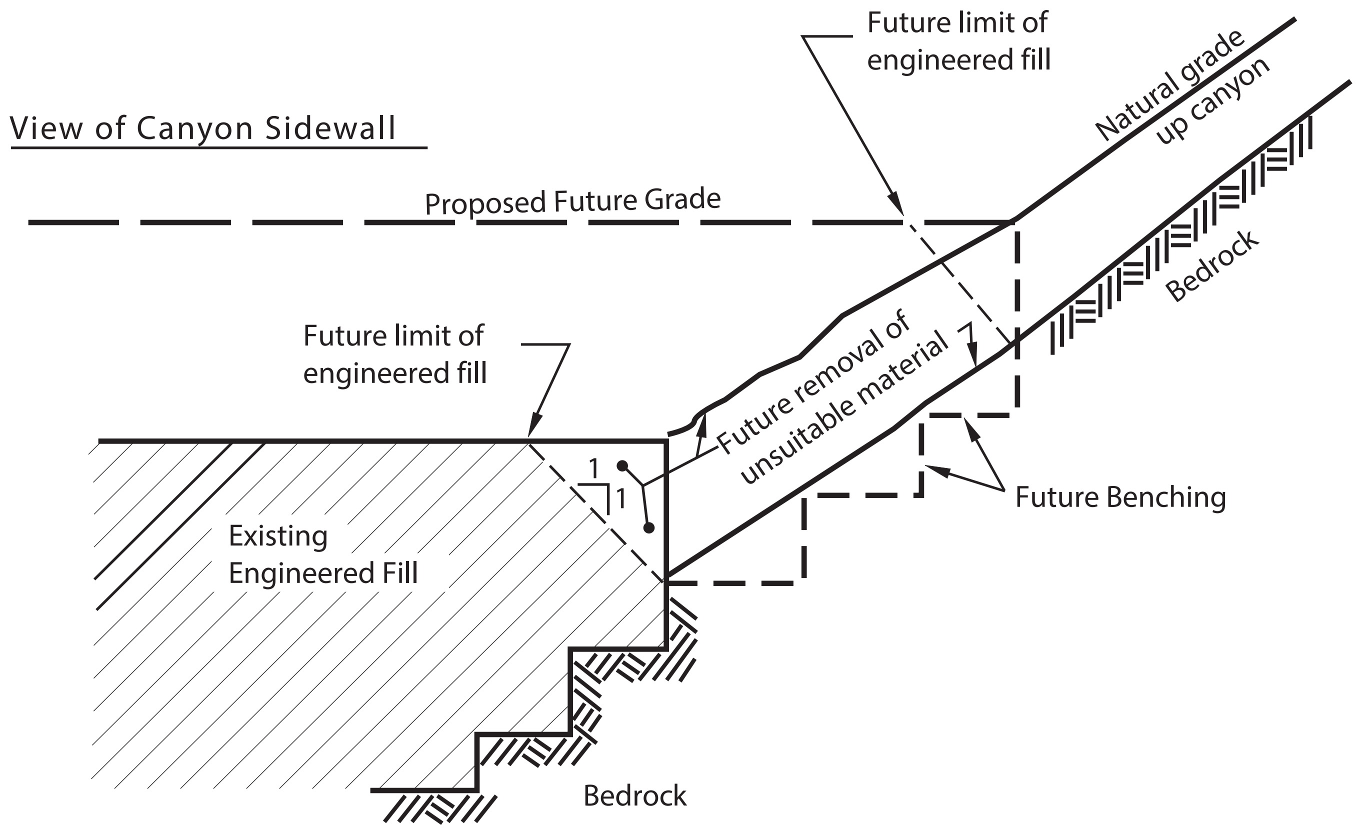
Notes:

1 - Pipe should be 4 inch diameter Schedule 40 PVC or similiar.
2 - Gradients should be 4% or greater.
3 - Cap all upstream ends
4 - Trenches for outlet pipes should be backfilled with compacted native soil.
5 - Backdrain pipe should have 8 uniformly spaced perforations per foot placed 90° offset on underside of pipe. Outlet pipe should be nonperforated.
6 - For the geofrabric alternative the backdrain pipe may be omitted provided at least 20 feet (i.e. 10 ft each side of outlet) of perforated pipe is provided to lead into each outlet.
7 - At each outlet the geofabric should be appropriately overlapped (1ft.) at cuts in fabric or otherwise sealed or taped around the pipe.

STANDARD DETAIL NO. 4

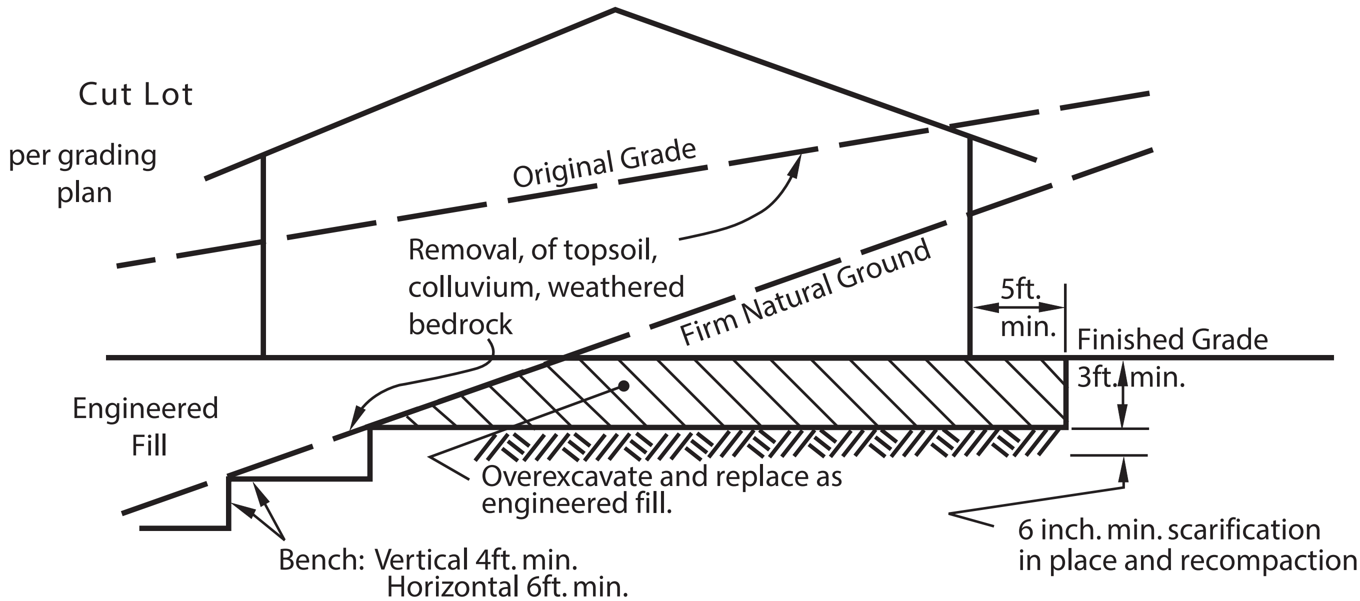
FUTURE CANYON FILL

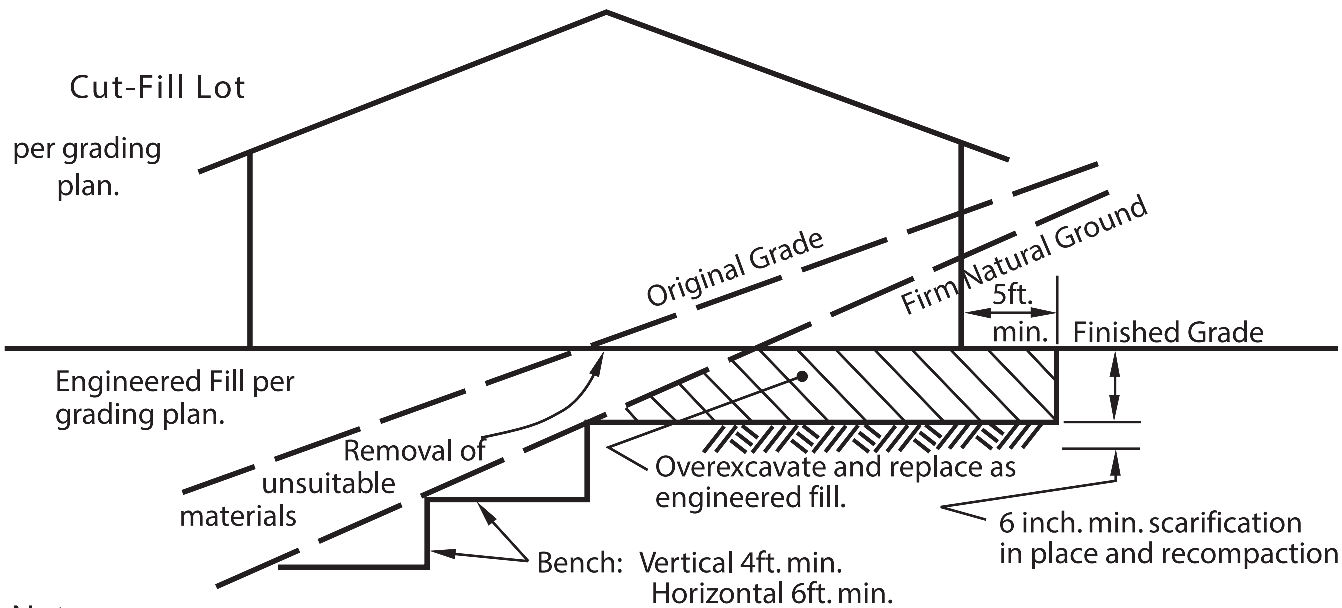




STANDARD DETAIL NO. 5

TRANSITION LOT OVER-EXCAVATION





Notes:

1 - Topsoil, colluvium, weathered bedrock and otherwise unsuitable materials should be removed to firm natural ground as identified by the geotechnical consultant.

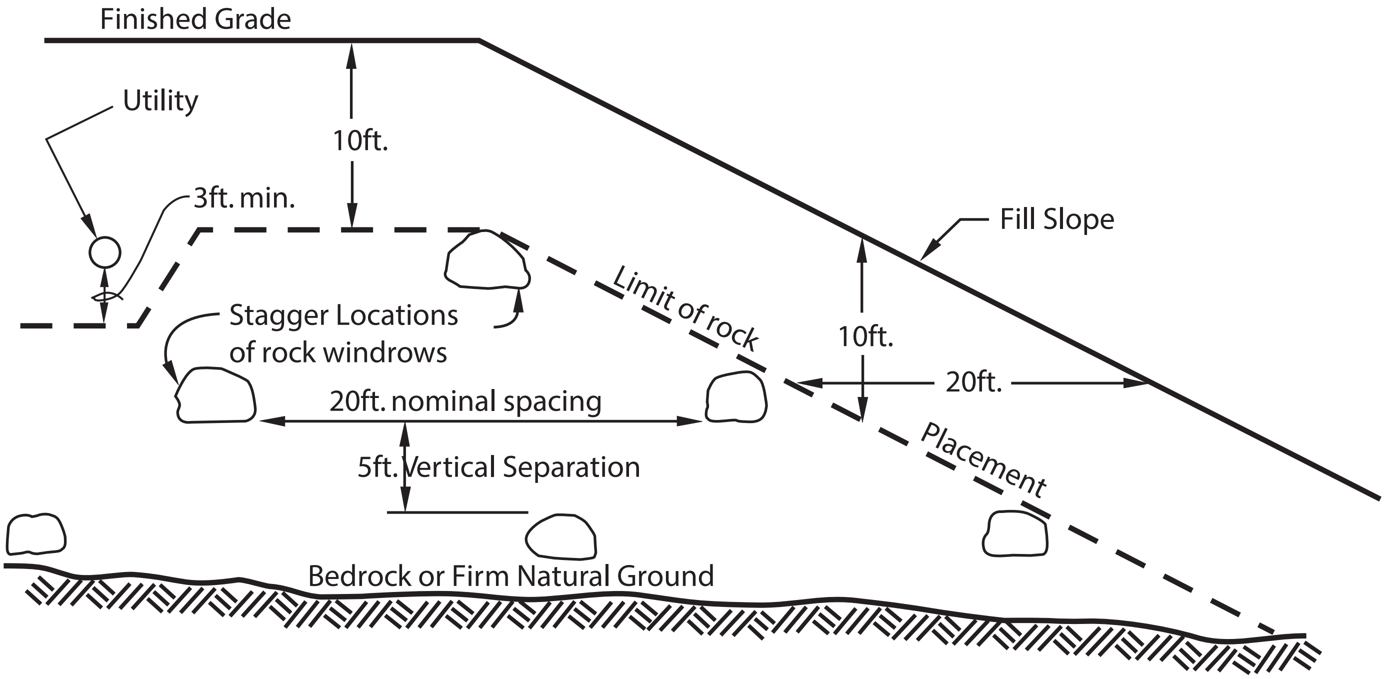
2 - The minimum depth of overexcavation should be considered subject to review by the geotechnical consultant. Steeper transitions may require deeper overexcavation.

3 - The lateral extent of overexcavation should be 5 feet minimum, but may include the entire lot as recommended by the geotechnical consultant.

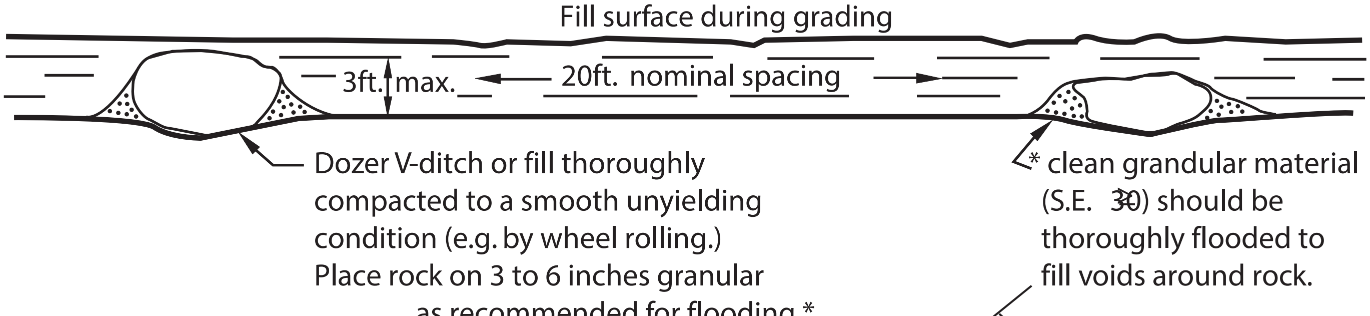
4 - The contractor should notify the geotechnical consultant in advance of achieving final grades (i.e. within 5 ft.) in order to evaluate overexcavation recommendations. Additional staking may be requested to aid in the evaluation of overexcavations.

STANDARD DETAIL NO. 6

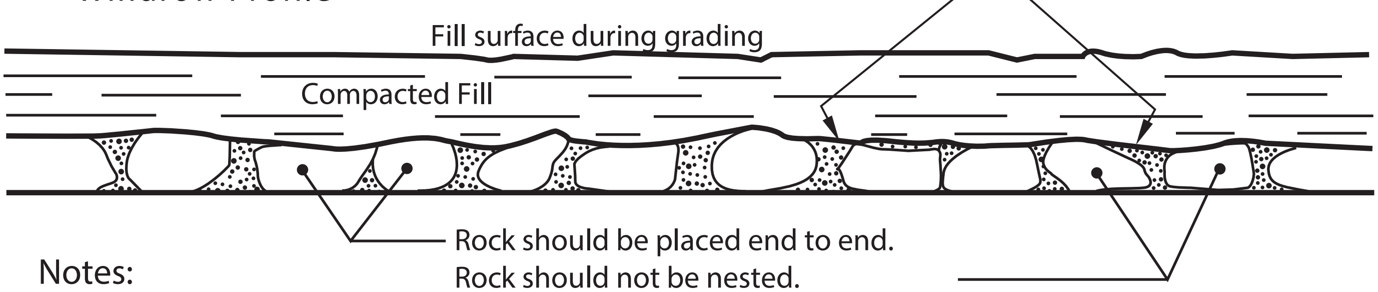
ROCK DISPOSAL



Windrow Section

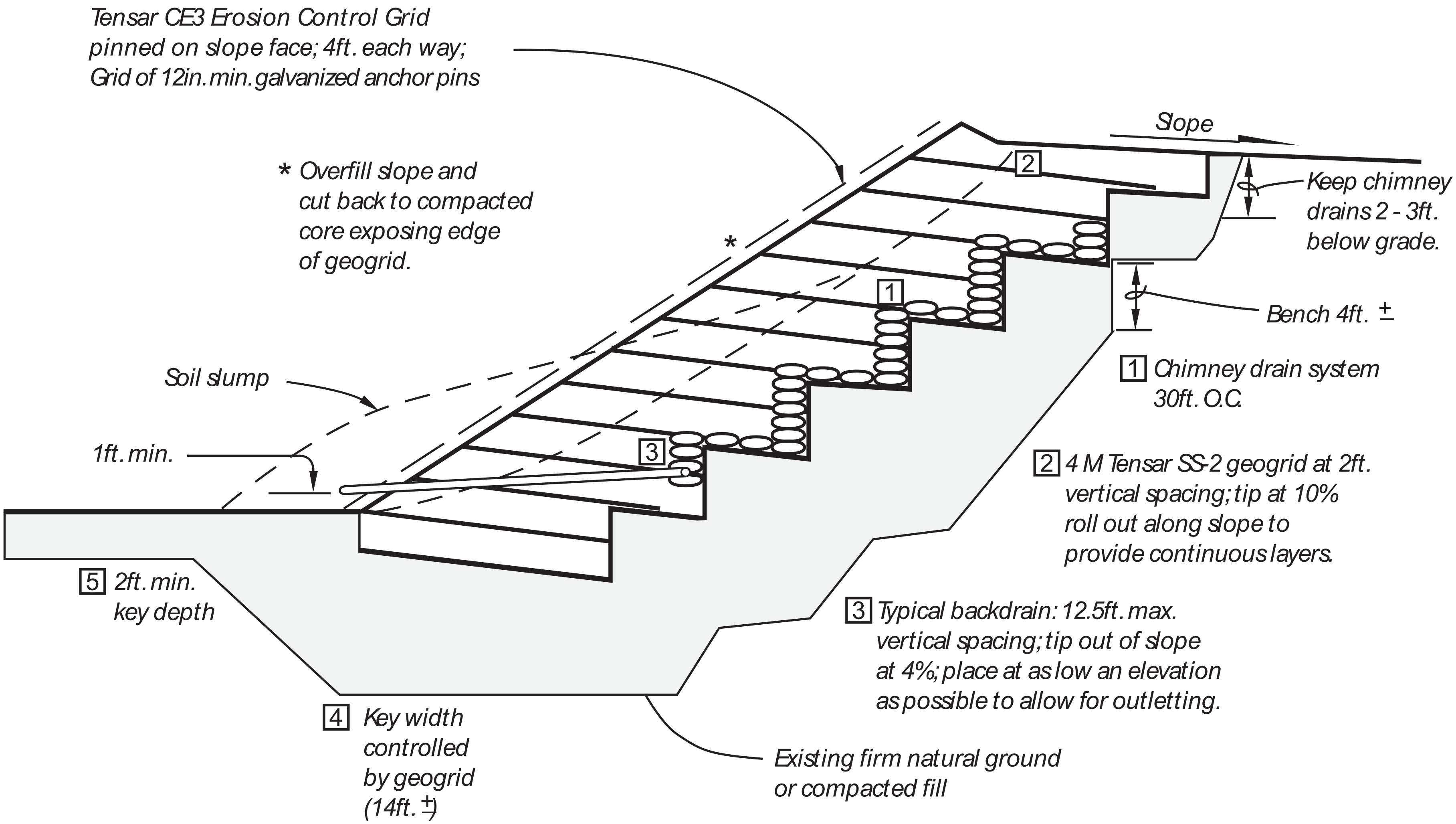


Windrow Profile



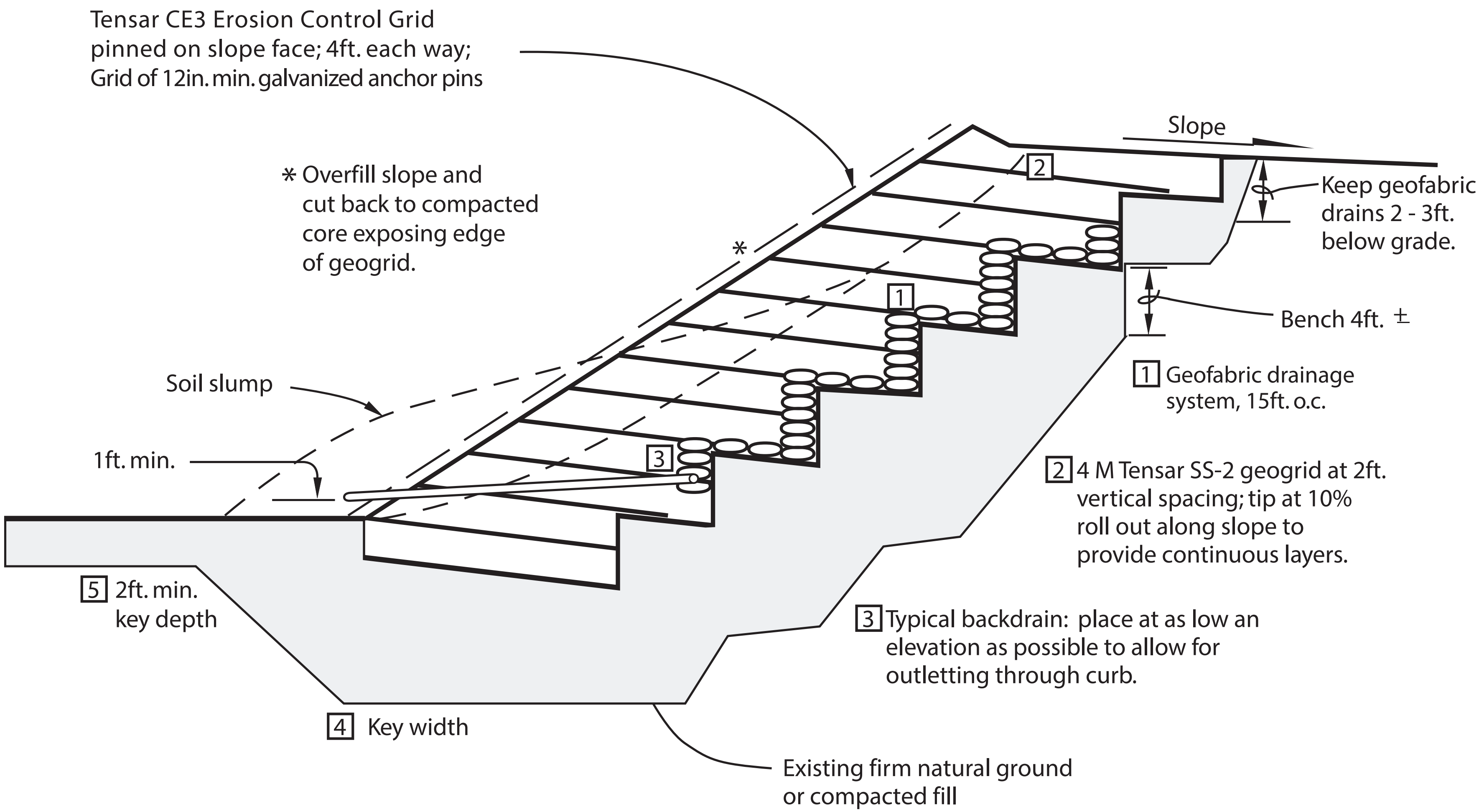
Notes:

1 - Following placement of rock, flooding of granular material, and placement of compacted fill adjacent to windrow, each windrow should be thoroughly compacted from the surface.
2 - The contractor should provide to the geotechnical consultant plans prepared by survey documenting the location of buried rock.
3 - Disposal in streets may be subject to more restrictive requirements by the governing authorities.



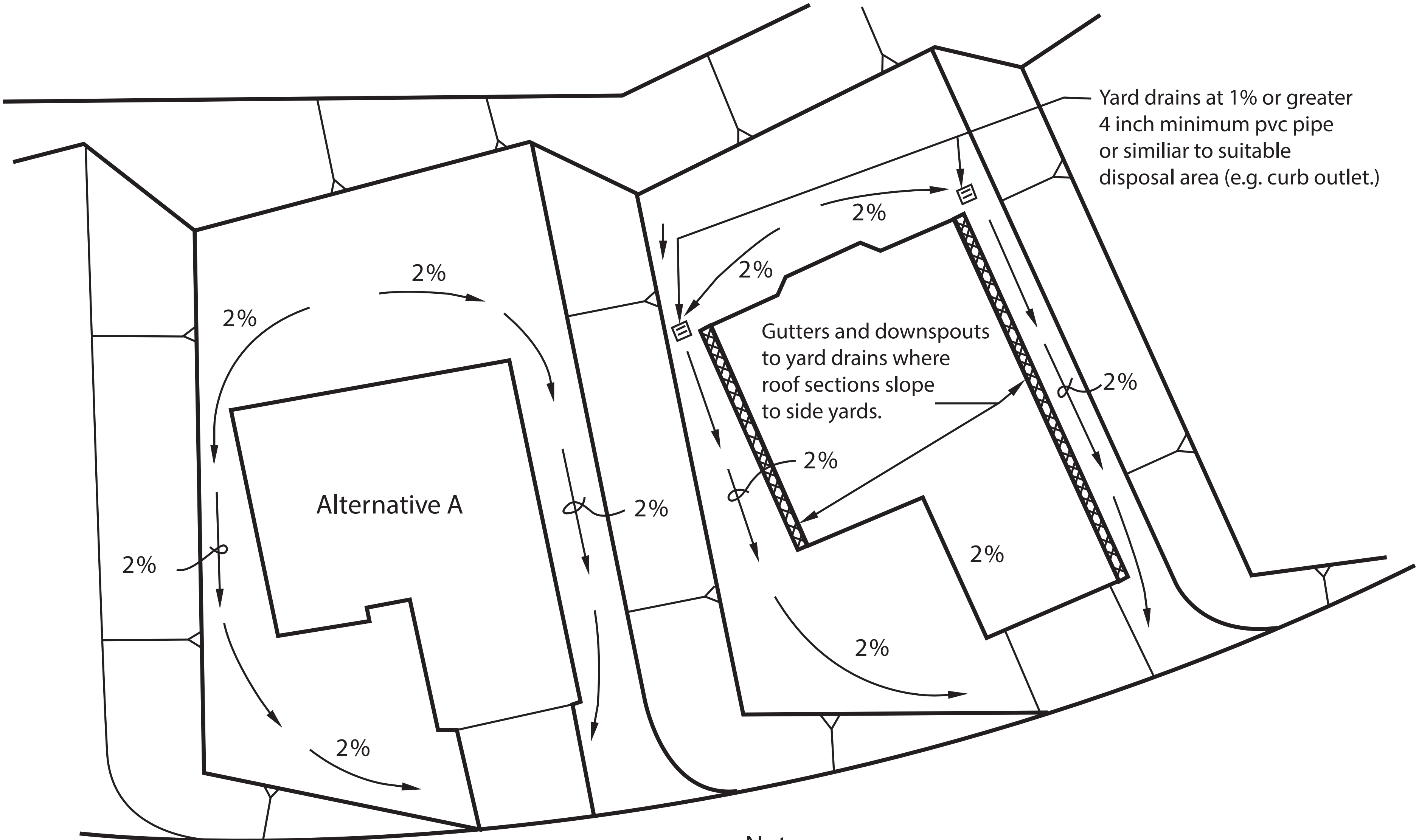
MINOR SLOPE REPAIR

STANDARD DETAIL NO. 8



MINOR SLOPE REPAIR

STANDARD DETAIL NO. 8A



Notes:

1 - Drainage into swale areas should be at 2% gradient.
Directly away from buildings drainage should be at 5%.

LOT DRAINAGE

STANDARD DETAIL NO. 9