Geotechnical Exploration, Inc.

SOIL AND FOUNDATION ENGINEERING ● GROUNDWATER ● ENGINEERING GEOLOGY

12 January 2023

Mr. Pierre Van Der Nerwe
La Jolla Reserve, LLC
10452 Coyote Hill Glen
Escondido, CA 92026

Job No. 10-9977

Subject: **Response to DSD-Geology Comments**
The Reserve – Romero Subdivision
Romero Drive
APN 352-300-11-00
La Jolla, California

Dear Mr. Nerwe:

Per your request as required by the DSD Geology reviewer, we are responding to the following issues presented in project issues report PRJ-1063767 dated November 14, 2022.

DSD-Geology Comment 00061|Page: *The project's geotechnical consultant must provide an addendum geotechnical report or update letter for the purpose of an environmental review that specifically addresses the proposed development plans, tentative map and the following.*

GEI Response: This letter will serve as our geotechnical addendum/update letter that specifically addresses the proposed development plans, tentative map and the following:

DSD-Geology Comment 00062|Page: *Retaining walls are proposed at or adjacent to the limits of grading. Indicate if remedial grading will be necessary outside the limits of grading currently shown on the plans to construct the proposed retaining walls.*

GEI Response: We anticipate remedial grading will be necessary outside the limits of grading shown on the current plans for retaining walls. Grading for retaining walls will be needed up to the property lines. Please see the recommended approximate limits of remedial grading on the attached Plot Plan and Site-Specific Geologic Map, Figure No. I.

<u>DSD-Geology Comment 00063|Page:</u> If remedial grading is recommended beyond the circumscribed limits of grading shown on the plans, circumscribe the limits of anticipated remedial grading on the geologic/geotechnical map to delineate the proposed footprint of the project.

GEI Response: We anticipate remedial grading will necessary outside the circumscribed limits of grading shown on the current plans. Anticipated remedial Grading will be needed up to the property lines. Please see the recommended approximate limits of remedial grading on the attached Plot Plan and Site-Specific Geologic Map, Figure No. I

<u>DSD-Geology Comment 00064|Page</u>: Exploratory Trench T-6, Figure IIIt appears to show offset in the undifferentiated Scripps Formation/Ardath Shale across the gap in the trench log at the chain link fence. Clarify why this offset is not evidence of faulting.

GEI Response: Although it might suggest offset in the undifferentiated Scripps Formation/Ardath Shale formational materials (Tsc/Ta) across the gap in the trench log at the immovable fire road fence as presented in our previously prepared exploratory trench T-6 (Figure No. IIIt), there is not enough supporting evidence to justify designating the topsoil-slopewash (Qsw) and slopewash-fill soil (Qaf) apparent contact offsets across the trenching gap as being due to fault offset.

The apparent cross gap differential of the fill soil-slopewash contact is the same as the slopewash-topsoil contact. There have been no seismic events in the San Diego area that would have offset recently placed fill soils. No jointing or high angle breakage of the undifferentiated Scripps Formation/Ardath Shale formational materials (Tsc/Ta) that would normally be associated with faulting was observed on either side of the trench gap. Strike and dip attitudes of the undifferentiated Scripps Formation/Ardath Shale formational materials (Tsc/Ta), which had very well-defined bedding, were consistent for the full 135-foot length of the trench, including across the 5-foot trench log gap.

In summary, we conclude that the contact elevation differentials across the trench gap are not due to faulting but are a topographic feature with slightly steeper ground surface inclinations descending from the east more than the west, toward the centerline of the trench gap.

We note that we observed a drafting error in the numbering for the referenced exploratory trench T-6 (Figure No. IIIt). The correct figure number for the original exploratory trench T-6 figure should be Figure No. IIIj. Please see that the correction has been made in the attached "Report of Geotechnical Investigation -Third Update" The Reserve – Romero Subdivision, dated October 14, 2022 (Appendix A) with reference to the original geotechnical investigation field work.

DSD-Geology Comment 00065|Page: *In general accordance with the Subdivision Map Act, the project geotechnical consultant should:*

Indicate whether or not there are any soil conditions within the area of the Tentative Map which, if not corrected, would lead to construction defects and

Indicate if rocks or liquids containing deleterious chemicals are present which, if not corrected, could cause construction materials such as concrete, steel, and ductile or cast iron to corrode or deteriorate.

GEI Response: To assess soil corrosivity of the explored on-site soils, resistivity, pH, chloride and soluble sulfate tests were performed by an outside consultant (Clarkson Laboratory and Supply, Inc.) on samples of the currently explored soils at the indicated sampled depths and some explored near surface soils most likely to be in contact with concrete and ferrous metals. The most common factor in determining soils corrosivity to ferrous metals is electrical resistivity. As soil resistivity decreases, its corrosivity to ferrous metals increases. The tested soils yielded resistivities of 3,300 and 850 Ohm-cm, indicating that the soils are moderately to severely corrosive to ferrous metals.

Soils and fluids are considered neutral when pH is measured at 7, acidic when pH is measured at <7 and alkaline when measured at >7. Soils are considered corrosive when the pH gets down to around 5.5 or less. Results of the laboratory testing yielded pH values of 6.8 and 5.2, indicating that the tested soils are mildly acidic, and a factor in soil corrosivity to metals.

Large concentrations of chlorides will adversely affect any ferrous metals such as iron and steel. Soil with a chloride concentration greater than or equal to 500 ppm (0.05 percent) or more is considered corrosive to ferrous metals. The chloride content of the tested soils measured at approximately 200 and 1,080 ppm or 0.02 and 0.108 percent, respectively, indicating that chloride content from B-12 tested soils (5 to 6.5 feet deep) is a factor in corrosion to ferrous metals.

The primary cause of deterioration of concrete in foundations and other below ground structures is the corrosive attack by soluble sulfates present in the soil and groundwater. The results of water-soluble sulfate testing performed on representative samples of the near surface soils in the general area of the proposed structures, yielded soluble sulfate contents of 210 ppm and 2,620 ppm or 0.021 percent and 0.262 percent, indicating that the proposed cement-concrete structures that are in contact with the underlying soils are anticipated to be affected with a S0 to S2 sulfate exposure. Test results should be evaluated by an engineer specializing in soil corrosivity to determine the cement type recommended by the current edition of the CBC (2019) or the American Concrete Institute. Cement type recommendations and concrete specifications should be provided by the structural engineer based on the soil corrosivity test results.

The table below summarizes the laboratory results for chemical testing of the sampled soils:

Sample Location/ Depth (ft)	**pH**	**Soluble Sulfate (PPM)**	**Soluble Chloride (PPM)**	**Soil Resistivity (Ohm-cm)**
B-1/1.5-2.0	6.8	200	210	3,300
B-12/5.0-6.5	5.2	1080	2620	850

The laboratory testing results provide preliminary values for reference at this time. After grading has been completed, additional samples can be taken within each new building pad for final values. It should be noted that ***Geotechnical Exploration Inc***., does not practice corrosion engineering and our assessment here should be construed as an aid to the owner or owner's representative. A corrosion specialist should be consulted for any specific design requirement based on test results. Additional laboratory tests will be performed on representative soil samples close to finish grade elevation on the building pads.

DSD-Geology Comment 00066|Page: *The project's geotechnical consultant should provide a conclusion regarding if the proposed development will destabilize or result in settlement to adjacent property or the right of way.*

GEI Response: Based on the available information at this stage, it is our opinion that the proposed site development would not destabilize or result in settlement to adjacent property or the right of way if designed and constructed in accordance with the recommendations provided in our "Report of Geotechnical Investigation -Third Update" The Reserve – Romero Subdivision, dated October 14, 2022 (Appendix A).

DSD-Geology Comment 00067|Page: *The project's geotechnical consultant must provide a professional opinion that the site will have a factor of safety of 1.5 or greater for both gross and surficial stability following project completion.*

GEI Response: It is our opinion that both gross and surficial stability will not be a concern following project completion based on the most current conceptual grading plans. We have performed both gross and surficial stability calculations for the existing site conditions with the anticipation of the proposed development. Please see Appendix A for both gross and surficial stability calculations of the existing site conditions. We will provide a professional opinion that the site will have a factor of safety of 1.5 or greater for both gross and surficial stability following project plan completion once final grading plans have been made available for our review.

DSD-Geology Comment 00068|Page: *Please note, the requested addendum/update letter must be uploaded with the "Geotechnical Investigation Report Addendum" PDF file option only.*

Please note, to avoid additional reviews, do not attempt to submit any additional documents using the "Geotechnical Investigation Report Addendum" PDF file option as this will overwrite the previously submitted record geotechnical document for the project.

Please note, geotechnical documents that are uploaded incorrectly are unacceptable as record documents.

GEI Response: Noted.

DSD-Geology Comment 00068|Page: *Please note, storm water requirements for the proposed conceptual development will be evaluated by DSD-Engineering review. Priority Development Projects may require an investigation of storm water infiltration feasibility in accordance with the City's current Storm Water Standards. Check with your DSD-Engineering reviewer for requirements. If necessary, DSD-Engineering may request DSD-Geology review of the storm water infiltration evaluation.*

GEI Response: Noted.

The findings and opinions presented herein have been made in accordance with generally accepted principles and practice in the field of geotechnical engineering within the City of San Diego. No warranty, either expressed or implied, is made.

If you have any questions regarding this letter, please contact our office. Reference to our **Job No. 10-9977** will help expedite a response to your inquiry.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

Jaime A. Cerros, P.E.
R.C.E. 34422/G.E. 2007
Senior Geotechnical Engineer

Leslie D. Reed, President
P.G. 3391/C.E.G. 999

Steve Osetek, Project Geologist

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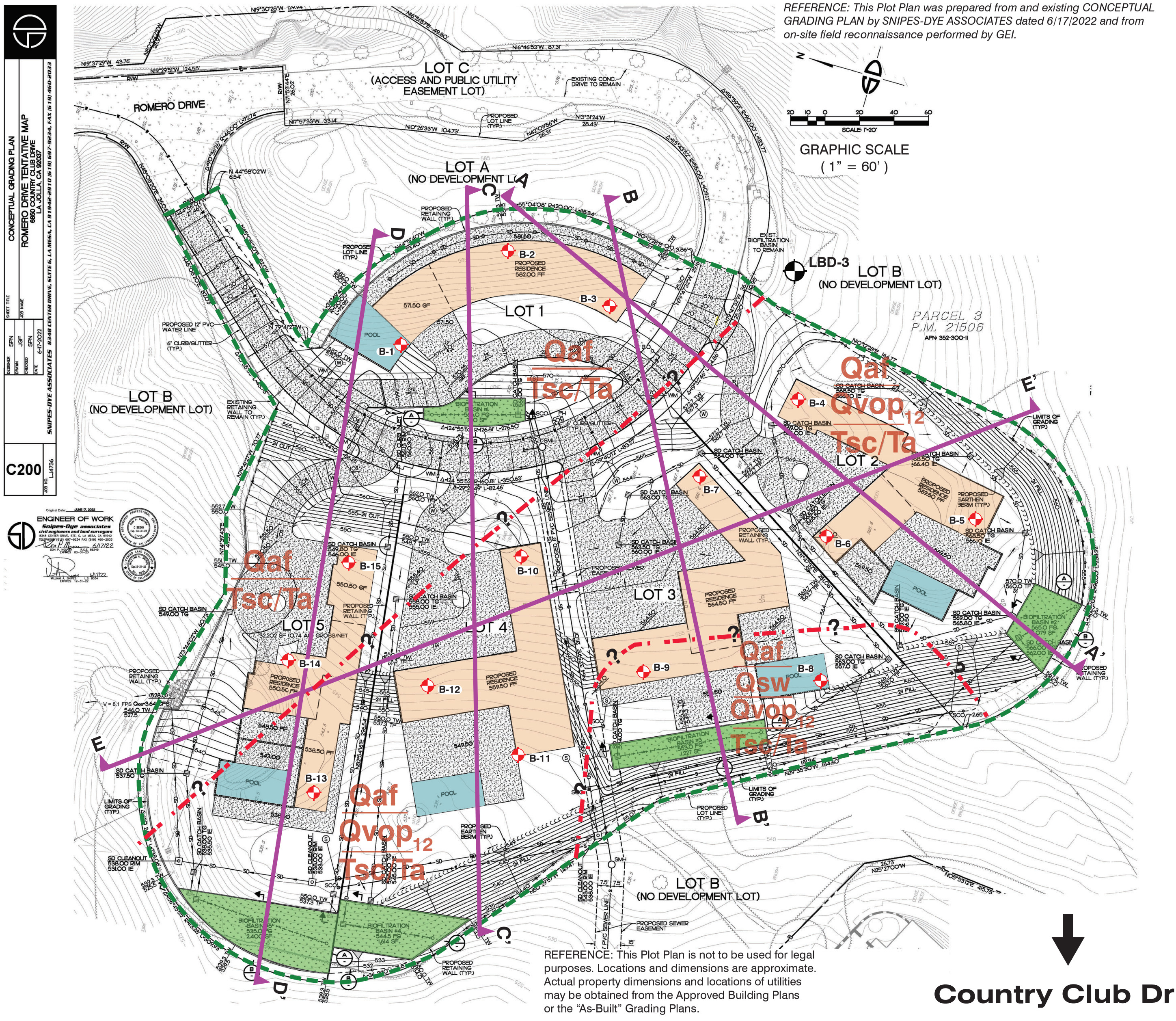
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

REFERENCE: *This Plot Plan was prepared from and existing CONCEPTUAL GRADING PLAN by SNIPES-DYE ASSOCIATES dated 6/17/2022 and from on-site field reconnaissance performed by GEI.*



REFERENCE: This Plot Plan is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

Country Club Drive

LEGEND

▭	Limits of Geotechnical Investigation and Approximate Limits of Remedial Grading
B-14	Approximate Location of Exploratory Boring
LBD-3	Approximate Location of Large Diameter Boring (2011 Investigation)
E E'	Approximate Location of Geologic Cross Section
■	Approximate Location of Proposed Structure
■	Approximate Location of Proposed Swimming Pool
🟩	Approximate Location of Biofiltration Basin

GEOLOGIC UNITS

Qaf	Artificial Fill
Qsw	Slopewash
$Qvop_{12}$	Very Old Paralic Deposits, Unit 12
Tsc/Ta	Scripps Formation/Ardath Shale - undifferentiated
?-·-?-	Approximate Geologic Contact

PLOT PLAN AND SITE SPECIFIC GEOLOGIC MAP

The Reserve - Romero Subdivision
Romero Drive
APN 325-300-11-00
La Jolla, CA.
Figure No. I
Job No. 10-9977

Geotechnical Exploration, Inc.

January 2023

10-9977-P-10-2022.ai

APPENDIX A

Report of Geotechnical Investigation –
Third Update -
October 14, 2022

REPORT OF GEOTECHNICAL INVESTIGATION – THIRD UPDATE

The Reserve – Romero Subdivision
Romero Drive
APN 352-300-11-00
La Jolla, California

JOB NO. 10-9977
14 October 2022

Prepared for:

Mr. Pierre Van Der Nerwe
La Jolla Reserve, LLC

Geotechnical Exploration, Inc.
SOIL AND FOUNDATION ENGINEERING ● GROUNDWATER ● ENGINEERING GEOLOGY

14 October 2022

Mr. Pierre Van Der Nerwe
La Jolla Reserve, LLC
10452 Coyote Hill Glen
Escondido, CA 92026

Job No. 10-9977

Subject: **Report of Geotechnical Investigation – Third Update**
The Reserve – Romero Subdivision
Romero Drive
APN 352-300-11-00
La Jolla, California

Dear Mr. Van Der Nerwe:

Per the request of your project architect, Mr. Kent Coston, ***Geotechnical Exploration, Inc.*** has prepared this report as a third update to our *"Report of Preliminary Geotechnical and Geologic Investigation"* dated November 16, 2011, our "Update *Report of Preliminary Geotechnical and Geologic Investigation"* dated October 23, 2016, and our *"Report of Limited Geotechnical Investigation Proposed Storm Water Infiltration BMPs"* dated August 28, 2017. We recently prepared a *"Limited Geotechnical Update"* report dated July 29, 2022, to acknowledge that the previous grading plans have been updated. This third update report is intended to supersede our *"Limited Geotechnical Update"* report dated July 29, 2022, following our recently performed additional soil investigation in the area of the proposed five (5) lot residential development associated with the new 5-lot subdivision.

This opportunity to be of service is sincerely appreciated. If you have any questions regarding report, please do not hesitate to contact our office. Reference to our **Job No. 10-9977** will help to expedite a response to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

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R.C.E. 34422/G.E. 2007
Senior Geotechnical Engineer

Leslie D. Reed, President
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REPORT OF GEOTECHNICAL INVESTIGATION – THIRD UPDATE

The Reserve – Romero Subdivision
Romero Drive APN 352-300-11-00
La Jolla, California

JOB NO. 10-9977

I. SCOPE OF WORK

As stated, ***Geotechnical Exploration, Inc.*** has prepared this report as a third update to our *"Report of Preliminary Geotechnical and Geologic Investigation"* dated November 16, 2011, our "Update *Report of Preliminary Geotechnical and Geologic Investigation"* dated October 23, 2016, and our *"Report of Limited Geotechnical Investigation Proposed Storm Water Infiltration BMPs"* dated August 28, 2017. We recently prepared a *"Limited Geotechnical Update"* report dated July 29, 2022, to acknowledge that the previous grading plans have been updated. This third update report is intended to supersede our *"Limited Geotechnical Update"* report dated July 29, 2022, following our recently performed additional soil investigation in the area of the proposed five (5) lot residential development associated with the new 5-lot subdivision.

It is our understanding that the most current grading plans have been updated since our 2016 report was prepared. Based on our review of the most current conceptual grading plans prepared by Snipes-Dye Associates dated June 17, 2022, we understand that the property will be subdivided to include 5 residential lots for 5 new single-family residences. Swimming pools and associated new exterior improvements will be constructed on those lots.

The site has been altered in the area of the proposed 5 new residential lots since our 2016 update geotechnical report was prepared for the property. The alterations observed in the area of the new development include ornamental grass landscaping, a concrete path along the outer perimeter of Lots 2 through 5 and a catch basin BMP located in the northwest corner of the proposed 5-lot development. As part of this update, we reviewed the previous reports and current plans, discussed the scope of work for the current project and performed an additional soil investigation in the area of the proposed new residential development on August 18 and 20, 2022. Refer to Figure No. I, the Vicinity Map, for the site location. Refer to Figure No. II, Plot Plan and Site-Specific Geologic Map, for the approximate locations of the proposed 5 new residential lots and the site-specific geologic map.

We previously issued the following documents for this site:

1. *"Report of Preliminary Geotechnical and Geologic Investigation"* dated November 16, 2011.

2. *"Grading Plan Change A Review*" dated October 12, 2016.

3. *"Update Report of Preliminary Geotechnical and Geologic Investigation"* dated October 23, 2016.

4. *"Report of Limited Geotechnical Investigation Proposed Storm Water Infiltration BMPs"* dated August 28, 2017.

5. *"Limited Geotechnical Update"* report dated July 29, 2022.

The architectural plans for the residences, prepared by Coston Architects Inc., and dated October 11, 2016, were provided at the time of our previously issued 2016 report. See Appendix A for our *"Update Report of Preliminary Geotechnical and Geologic Investigation"* dated October 23, 2016. We understand that the grading plans have been updated at this time. Although the geologic and geologic hazards portion of our previously issued 2016 report remain applicable, the grading and foundation related recommendations for the most current development need to be updated. We performed an additional soil investigation in the area of the proposed new development to evaluate the current soil conditions after the most recent site alterations and are providing new geotechnical recommendations in this report where required.

II. <u>FIELD EXPLORATION FINDINGS</u>

Our additional field investigation work was conducted on August 18 and 19, 2022. The field investigation consisted of surface reconnaissance and a subsurface exploration program utilizing a limited access track mounted drill rig to investigate and sample the subsurface soils. Fifteen (15) exploratory borings (B-1 through B-15) were excavated to depths ranging from 2.5 to 15 feet in the areas of the proposed 5 new residences and associated improvements. The borings were continuously logged in the field by our geologist and described in accordance with the Unified Soil Classification System. The approximate locations of the exploratory borings are shown on Figure No. II.

As encountered in our recent soil investigation, the area of the property to receive the new residences is underlain at depth by stiff/medium dense to very stiff/dense formational material, which is overlain by loose to medium dense fill (Qaf) ranging in depth from 1 to 4½ feet in the building pad areas. Fill soil is thickest on the northwestern portion of proposed Lot 5 (encountered in B-13 and B-14) in the area of the existing catch basin BMP. Slopewash (Qsw) soils were also observed underlying the fill soils in relatively limited areas on the lower western portion of lot 3 as encountered in B-8 and B-9. The medium dense slopewash soils were encountered at depths ranging from approximately 2 to 2.5 feet from the existing ground surface and were observed to be approximately 2 feet in thickness. Scripps Formation/Ardath Shale (Tsc/Ta) formational materials were encounter underlying the fill soils on Lot 1 and the eastern half of the proposed residence on Lot 5. Expansion testing of representative samples of the sandy lean clay Scripps Formation/Ardath Shale formational materials resulted in an expansion index of 83, classifying the soils as having a medium expansion potential. Very Old Paralic Deposits, Unit12 (Qvop12), were encountered underlying the fill and slopewash soils on Lots 2 to 4. The silty sand Very Old Paralic Deposits are generally considered to have a low to medium high expansion potential.

Exploratory boring logs have been prepared based on our observations and laboratory test results, and are attached as Figure Nos. IIIa-o. Laboratory tests were performed on retrieved soil samples in order to evaluate their physical and mechanical properties. The test results are presented on Figure Nos. IIIa-o and IVa-c.

The existing fill soils will require removal and recompacted to their full depth of approximately 4½ feet. Based on our review of the current grading plans, the existing fill soils will be completely removed from the eastern portions of the proposed residences during the planned cuts of the grading operation. All existing fill should be completely removed and properly recompacted prior to the addition of any fill material and/or structural foundations, slabs, and improvements. Based on our experience, the density of slopewash soils may vary at other locations from the conditions observed at the locations of our exploratory borings. Additional observations and evaluation of the exposed slopewash soils will be required by a representative from our firm during grading operations, and additional recommendations may be required.

Recommendations from our previous reports remain applicable except as superseded in this report. The seismic soil design parameters provided in the October 23, 2016, report are no longer applicable. The seismic soil design parameters, which were updated in the July 29, 2022, report (and provided again in this report) are in accordance with 2019 CBC and ASCE 7.16, and remain applicable.

We also performed updated slope stability calculations based on the current grading plans using the Bishop method in the *Slide* program by RocScience. The slope stability analyses were performed along cross sections A-A′, B-B′, C-C′, D-D′ and E-E′, (see Figure Nos. Va-e). The location of each cross sections is presented on the Plot Plan and Site-Specific Geologic Map, Figure No. II. Based on our slope stability analyses, a factor of safety less than 1.5 against gross or shallow slope failure does not exist on the property. In our professional opinion, the site will have a factor of safety of 1.5 or greater following the proposed construction. Refer to Appendix C for the results of the analysis.

Based on the current grading plans, five (5) separate biofiltration basins are proposed for the new 5-lot subdivision. That is, one biofiltration basin for each of the new residential lots. As previously mentioned, we performed infiltration testing and prepared our "*Report of Limited Geotechnical Investigation, Proposed Storm Water Infiltration BMPs*" dated June 14, 2017. At the time of our infiltration testing in 2017, possible future subdivision development of the property was discussed and infiltration testing was performed at 10 locations (INF-1 to INF-10) for the possible future subdivision. The recorded infiltration rates at the previously tested locations where site alterations were not performed along the western and southwestern perimeter of Lots 2, 3, and 4 (INF-1 to INF-4) may remain applicable for the use of the project civil engineer after final grading plans have been reviewed and confirmed.

Updated recommendations for the soil seismic parameters are being provided in this report with updated figures associated with the most current conceptual grading plans, as discussed below.

It is our opinion, based on our review of our previous geotechnical reports and the results of our original field investigation, that no significant soil or geologic hazards exist at the subject site and the property is well suited for the proposed residential project.

III. UPDATED RECOMMENDATIONS

1. *General:* Grading should conform to the guidelines presented in the California Building Code (CBC, 2019), as well as the requirements of the City of San Diego.

   During earthwork construction, removal of the undocumented variable density fill soils, as well as general grading procedures of the contractor, should be observed, and the fill placed and selectively tested by representatives of the geotechnical engineer, ***Geotechnical Exploration Inc***. If any unusual or unexpected conditions are exposed in the field, they should be reviewed by the geotechnical engineer and if warranted, modified and/or additional remedial recommendations will be offered. Specific guidelines and comments pertinent to the planned development are provided herein.

   The recommendations presented herein have been completed using the information provided to us regarding site development. If information concerning the proposed development is revised or any changes are made in the design and location of the proposed property, they must be modified or approved in writing by this office.

2. *Clearing and Stripping:* The areas of proposed new residential improvements should be cleared of the existing concrete flatwork not being utilized in the new construction, abandoned utilities and any other obstructions present at

the time of construction. After clearing, the ground surface should be stripped of vegetation within the areas of proposed new construction. This includes any roots from trees and shrubbery. After clearing the ground surface should be stripped of existing vegetation within the areas of proposed new construction. Holes resulting from the removal of root systems or other buried obstructions that extend below the planned grades should be cleared and backfilled with suitable compacted material compacted to the requirements provided under Recommendation Nos. 5, 6 and 7 below. Prior to any filling operations, the cleared and stripped vegetation and debris should be disposed of off-site.

3. *Excavation:* After the entire site has been cleared and stripped, the existing fill soils should be removed and recompacted. The removal should be observed and approved by a representative of ***Geotechnical Exploration Inc***. to verify that all the fill soil has been completely removed. In addition, the condition of any exposed slopewash soils should be observed and approved by a representative of our firm as well. It is anticipated that the depth of the existing loose fill soil removal across the site will be approximately 1 to 4½ feet below existing grade in the areas of the proposed residences, swimming pools and improvements. It should be mentioned that the depths of removal described above are based on the results of our exploratory borings locations. Deeper or shallower removal may be necessary in areas outside our exploratory borings.

Based on our experience with similar materials in the project area, it is our opinion that the existing fill, slopewash and formational materials can be excavated utilizing ordinary medium to heavyweight earthmoving equipment. Contractors should not, however, be relieved of making their own independent evaluation of excavating the on-site materials prior to submitting their bids. Contractors should also review this report and our 2016 report (Appendix A), along with the boring logs to understand the scope and quantity of grading required for this project. Variability in excavating the subsurface materials should be expected across the project area.

The areal extent required to remove the surficial soils should be confirmed by our representatives during the excavation work based on their examination of the soils being exposed. The lateral extent of the excavation and recompaction should be at least 5 feet beyond the edge of the perimeter ground level foundations of the new residential structure and any areas to receive exterior improvements or fill slopes, where feasible, or to the depth of excavation or planned fill at that location, whichever is greater.

4. *Cut-Fill Transition:* New structures should not bear on a cut-fill transition line. If the final plans indicate a cut-fill transition line exists within the proposed residence building envelope (as is proposed for all 5 residences on the current plans), we recommend that the cut portion of the building pad be undercut to a minimum of 3 feet below the bottom of the proposed footing depth. The bottom of the overexcavation should be observed and approved by a representative of ***Geotechnical Exploration Inc***. to verify that all loose and unsuitable soils have been completely removed prior to reprocessing.

   After approval, the bottom of the excavation should be scarified to a minimum depth of 8 inches below removal grade elevations, brought to near-optimum moisture conditions and recompacted to at least 90 percent relative compaction (based on ASTM Test Method D1557). Backfill and compaction of the remaining structural fill should be performed based on the recommendations presented in the following sections. No structures should be supported on a building pad with a structural fill soil thickness differential greater than 5 feet.

5. *Subgrade Preparation:* After the site has been cleared, stripped, and the required excavations made, the exposed subgrade soils in areas to receive new fill and/or slab on-grade building improvements should be scarified to a depth of 6 inches, moisture conditioned, and compacted to the requirements for structural fill. Medium expansive Scripps Formation/Ardath Shale formational materials are expected to be encountered during the grading for the proposed residential pad of Lot 1 and the eastern half of the residential pad on Lot 5. Low to medium expansive Very Old Paralic Deposits are expected to be encountered during the grading for the proposed residential pads on Lots 2 through 4. Where planned cuts expose low to medium expansive formational materials in the building areas, they should be scarified and moisture conditioned to at least 3 percent over optimum moisture. Where needed, undercutting should be performed as explained above.

6. *Material for Fill:* Existing on-site low expansion potential soils (Expansion Index of 50 or less per ASTM D4829-19) with an organic content of less than 3 percent by volume are, in general, suitable for use as fill. Where feasible, medium expansion potential soils should be blended with low expansion potential soils during grading and may be used as structural fill under building areas when properly mixed and moisture conditioned. Imported fill material, where required, should have a low expansion potential. In addition, both imported and existing on-site materials for use as fill should not contain rocks or lumps more than 6 inches in greatest dimension if the fill soils are compacted with heavy compaction equipment (or 3 inches in greatest dimension if compacted with lightweight equipment). All materials for use as fill should be approved by our representative prior to importing to the site.

7. *Structural Fill Compaction:* All structural fill, and areas to receive any associated improvements, should be compacted to a minimum degree of compaction of 90 percent based upon ASTM D1557-12e1. Fill material should be spread and compacted in uniform horizontal lifts not exceeding 8 inches in uncompacted thickness. Before compaction begins, the fill should be brought to a water content that will permit proper compaction by either: (1) aerating and drying the fill if it is too wet, or (2) watering the fill if it is too dry. Each lift should be thoroughly mixed before compaction to ensure a uniform distribution of moisture. For low to medium expansive soils, the moisture content should be at least 3 percent over optimum. Highly expansive soils, if encountered at the site, must be placed outside building and improvement areas and should be compacted with at least 5 percent over optimum moisture content.

   Any rigid improvements founded on the existing surficial soils can be expected to undergo movement and possible damage. ***Geotechnical Exploration, Inc.*** takes no responsibility for the performance of any improvements built on loose natural soils or inadequately compacted fills. Subgrade soils in any exterior area receiving concrete improvements should be verified for compaction and moisture by a representative of our firm within 48 hours prior to concrete placement.

   No uncontrolled fill soils should remain after completion of the site work. In the event that temporary ramps or pads are constructed of uncontrolled fill soils, the loose fill soils should be removed and/or recompacted prior to completion of the grading operation.

8. *Water Soluble Sulfate Testing*: It is recommended that after rough grading is completed representative samples be obtained of the surficial soils to be in contact with the proposed concrete foundations to test for water-soluble sulfate content and chlorides. Test results should be evaluated by an engineer specializing in soil corrosivity and cement type recommendations should be provided by the Structural Engineer based on the soluble sulfate test results. It is noted that ***Geotechnical Exploration, Inc.*** does not practice corrosion engineering and our recommendation here should be construed as an aid to the owner. A corrosion specialist should be consulted for any specific design requirement.

9. *Seismic Data Bases:* The estimation of the peak ground acceleration and the repeatable high ground acceleration (RHGA) likely to occur at the site is based on the known significant local and regional faults within 100 miles of the site.

10. *Updated Seismic Design Criteria:* The proposed structure should be designed in accordance with the 2019 CBC, which incorporates by reference the ASCE 7-16 for seismic design. We have determined the mapped spectral acceleration values for the site based on a latitude of 32.8379 degrees and a longitude of -117.2583 degrees, utilizing a program titled "S*eismic Design Map Tool" and* provided by the USGS through SEAOC, which provides a solution for ASCE 7-16 utilizing digitized files for the Spectral Acceleration maps.

11. *Structure and Foundation Design*: The design of the new structures and foundations should be based on Seismic Design Category D, Risk Category II.

12. *Spectral Acceleration and Design Values*: The structural seismic design, when applicable, should be based on the following values, which are based on the site location, soil characteristics, and seismic maps by USGS, as required by

the 2019 CBC. A response Spectrum Acceleration (SA) vs. Period (T) for the site is included in Appendix B. The Site Class D (Stiff Soils) values for this property are:

TABLE I

Mapped Spectral Acceleration Values and Design Parameters

S_S	S_1	S_{MS}	S_{M1}	S_{DS}	S_{D1}	Fa	Fv	PGA	PGA_M	SDC
1.387	0.485	1.387	0.878	0.925	0.585	1.0	1.81	0.633	0.696	D

13. *Footings*: Footing configuration and reinforcement should be designed by the project Structural Engineer. The following are provided as design minimums.

We recommend that the proposed structures be supported on conventional, individual-spread and/or continuous footing foundations bearing on undisturbed stiff/medium dense to very stiff/dense formational materials or on properly compacted fill soils over formational soils. ***No footings should be underlain by undocumented fill soils.*** All building footings for one- and two-story structures should be built on formational soils or properly compacted fill prepared as recommended in this report. Building pad undercutting due to cut/fill transition will require all building footings to be in properly compacted fill soils. The footings should be founded at least 18 inches below the lowest adjacent finished grade when founded into properly compacted fill as previously described or medium dense to dense formational soils.

Footings located adjacent to utility trenches should have their bearing surfaces situated below an imaginary 1.0:1.0 plane projected upward from the bottom edge of the adjacent utility trench. Otherwise, the utility trenches should be excavated farther from the footing locations. Footings located adjacent to the tops of slopes should be extended sufficiently deep to provide at least 8 feet of horizontal cover between the slope face and outside edge of the footing at the footing bearing level.

14. *Bearing Values*: At the recommended depths, footings on formational or properly compacted fill soils may be designed for allowable bearing pressures of 2,500 psf for combined dead and live loads and 3,325 psf for all loads, including wind or seismic. The footings should, however, have a minimum width of 15 inches. An increase in soil allowable static bearing can be used as follows: 800 psf for each additional foot over 1.5 feet in depth and 400 psf for each additional foot in width to a total not exceeding 4,000 psf. The static soil bearing value may be increased one-third for seismic and wind load analysis. As previously indicated, all of the foundations for the structure should be built on stiff/medium dense to very stiff/dense formational materials or properly compacted fill soils.

15. *Footing Reinforcement*: All footings should be reinforced as specified by the Project Structural Engineer. However, based on our field investigation findings and laboratory testing, we provide the following minimum recommendations. All continuous footings should contain top and bottom reinforcement to provide structural continuity and to permit spanning of local irregularities. We recommend that a minimum of two No. 5 top and two No. 5 bottom reinforcing bars be provided in the footings. All footings should be reinforced as specified by the structural engineer. A minimum clearance of 3 inches should be maintained between steel reinforcement and the bottom or sides of the footing. Isolated square footings should contain, as a minimum, a grid of three No. 4 steel bars on 12-inch centers, both ways. In order for us to offer an opinion as to whether the footings are founded on soils of sufficient load bearing capacity, it is essential that our representative inspect the footing excavations prior to the placement of reinforcing steel or forms.

NOTE: The project Civil/Structural Engineer should review all reinforcing schedules. The reinforcing minimums recommended herein are not to be construed as structural designs, but merely as minimum reinforcement to reduce the potential for cracking and separations.

16. *Lateral Loads:* Lateral load resistance for the structure supported on footing foundations may be developed in friction between the foundation bottoms and the supporting subgrade. An allowable friction coefficient of 0.35 is considered applicable. An additional allowable passive resistance equal to an equivalent fluid weight of 270 pounds per cubic foot (pcf) acting against the foundations may be used in design provided the footings are poured neat against the stiff/medium dense to very stiff/dense formational or properly compacted fill materials. These lateral resistance value assume a level surface in front of the footing for a minimum distance of three times the embedment depth of the footing and any shear keys, but not less than 8 feet from a slope face, measured from effective top of foundation. Retaining walls supporting surcharge loads or affected by upper foundations should consider the effect of those upper loads.

17. *Settlement:* Settlement under structural design loads is expected to be within tolerable limits for the proposed structures. For footings designed in accordance with the recommendations presented in the preceding paragraphs, we anticipate that the total and differential static settlement for the proposed improvements should be on the order of approximately 1 inch and post-construction differential settlement angular rotation should be less than 1/240.

18. *Concrete Slab On-Grade Criteria -- Minimum Floor Slab Thickness and Reinforcement:* Slabs on-grade may only be used on new, properly compacted fill or when bearing on stiff/medium dense to very stiff/dense formational materials. Based on our experience, we have found that, for various reasons, floor slabs occasionally crack. Therefore, we recommend that all slabs on-grade contain at least a sufficient amount of reinforcing steel to reduce the separation of cracks, should they occur. Slab subgrade soil should be verified by a ***Geotechnical Exploration, Inc***. representative to have the proper moisture content within 48 hours prior to placement of the vapor barrier and pouring of concrete.

All slabs should be reinforced as specified by the project Structural Engineer. However, based on our field investigation findings and laboratory testing, we provide the following minimum recommendations: New interior floor slabs should be a minimum of 5 inches actual thickness and be reinforced with No. 4 bars on 18-inch centers, both ways, placed at mid-height in the slab. Soil moisture content should be kept above the optimum prior to waterproofing or vapor barrier placement under the new concrete slab.

Shrinkage control joints should be specified by the project Structural Engineer. We note that shrinkage cracking can result in reflective cracking in brittle flooring surfaces such as stone and tiles. It is imperative that if movement intolerant flooring materials are to be utilized, the flooring contractor and/or architect should provide specifications for the use of high-quality isolation membrane products installed between slab and floor materials.

19. *Slab Moisture Emission:* Although it is not the responsibility of geotechnical engineering firms to provide moisture protection recommendations, as a service to our clients we provide the following discussion and suggested

minimum protection criteria. Actual recommendations should be provided by the project architect and waterproofing consultants or product manufacturer. It is recommended to contact the vapor barrier manufacturer to schedule a pre-construction meeting and to coordinate a review, in-person or digital, of the vapor barrier installation.

Soil moisture vapor can result in damage to moisture-sensitive floors, some floor sealers, or sensitive equipment in direct contact with the floor, in addition to mold and staining on slabs, walls and carpets. The common practice in Southern California is to place vapor retarders made of PVC, or of polyethylene. PVC retarders are made in thickness ranging from 10- to 60-mil. Polyethylene retarders, called visqueen, range from 5- to 10-mil in thickness. These products are no longer considered adequate for moisture protection and can actually deteriorate over time.

Specialty vapor retarding and barrier products possess higher tensile strength and are more specifically designed for and intended to retard moisture transmission into and through concrete slabs. The use of such products is highly recommended for reduction of floor slab moisture emission.

The following American Society for Testing and Materials (ASTM) and American Concrete Institute (ACI) sections address the issue of moisture transmission into and through concrete slabs: ASTM E1745-17 Standard Specification for Plastic Water Vapor Retarders Used in Contact Concrete Slabs; ASTM E1643-18a Standard Practice for Selection, Design, Installation, and Inspection of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs; ACI 302.2R-06 Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials; and ACI 302.1R-15 Guide to Concrete Floor and Slab Construction.

19.1 Based on the above, we recommend that the vapor barrier consist of a minimum 15-mil extruded polyolefin plastic (no recycled content or woven materials permitted). Permeance as tested before and after mandatory conditioning (ASTM E1745 Section 7.1 and subparagraphs 7.1.1-7.1.5) should be less than 0.01-perms (grains/square foot/hour/per inch of Mercury) and comply with the ASTM E1745-17 Class A requirements. Installation of vapor barriers should be in accordance with ASTM E1643-18a. The basis of design is 15-mil Stego Wrap vapor barrier placed per the manufacturer's guidelines. Reef Industries Vapor Guard membrane has also been shown to achieve a permeance of less than 0.01 perms. We recommend that the slab be poured directly on the vapor barrier, which is placed directly on the prepared properly compacted smooth subgrade soil surface.

19.2 Common to all acceptable products, vapor retarder/barrier joints must be lapped at least 6 inches. Seam joints and permanent utility penetrations should be sealed with the manufacturer's recommended tape or mastic. Edges of the vapor retarder should be extended to terminate at a location in accordance with ASTM E1643-18a or to an alternate location that is acceptable to the project's structural engineer. All terminated edges of the vapor retarder should be sealed to the building foundation (grade beam, wall, or slab) using the manufacturer's recommended accessory for sealing the vapor retarder to pre-existing or freshly placed concrete.

Additionally, in actual practice, stakes are often driven through the retarder material, equipment is dragged or rolled across the retarder, overlapping or jointing is not properly implemented, etc. All these construction deficiencies reduce the retarder's effectiveness. In no case should retarder/barrier products be punctured or gaps be allowed to form prior to or during concrete placement. Vapor barrier-safe screeding and forming systems should be used that will not leave puncture holes in the vapor barrier, such as Beast Foot (by Stego Industries) or equivalent.

19.3 Vapor retarders/barriers do not provide full waterproofing for structures constructed below free water surfaces. They are intended to help reduce or prevent vapor transmission and/or capillary migration through the soil and through the concrete slabs. Waterproofing systems must be designed and properly constructed if full waterproofing is desired. The owner and project designers should be consulted to determine the specific level of protection required.

19.4 Following placement of any concrete floor slabs, sufficient drying time must be allowed prior to placement of floor coverings. Premature placement of floor coverings may result in degradation of adhesive materials and loosening of the finish floor materials.

20. *Exterior Slab Thickness and Reinforcement:* As a minimum for protection of on-site improvements, we recommend that all exterior pedestrian concrete slabs be 4 inches thick and be founded on properly compacted and tested low expansive soil fill, with No. 3 bars at 15-inch centers, both ways, at the center of the slab, and contain adequate isolation and control joints. The performance of on-site improvements can be greatly affected by soil base preparation and the quality of construction. It is therefore important that all improvements are properly designed and constructed for the existing soil conditions. The improvements should not be built on loose soils or fills placed without our observation and testing.

For exterior slabs with the minimum shrinkage reinforcement, control joints should be placed at spaces no farther than 15 feet apart or the width of the slab, whichever is less, and also at re-entrant corners. Control joints in exterior slabs should be sealed with elastomeric joint sealant. The sealant should be inspected every 6 months and be properly maintained.

21. *Retaining Wall Design Parameters – Unrestrained:* The active earth pressure to be utilized in the design of any cantilever site retaining walls, utilizing on-site low expansive or imported very low to low expansive soils as backfill should be based on an Equivalent Fluid Weight of 38 pcf (for level backfill only). For 2.0:1.0 sloping backfill, the cantilever site retaining walls should be designed with an equivalent fluid pressure of 52 pcf. Unrestrained retaining walls should be backfilled with properly compacted very low to low expansive soils. Unrestrained retaining walls with level backfill may use a conversion load factor of 0.31 for vertical surcharge loads converted to uniform lateral surcharge loads and 0.42 when supporting a sloping 2:1 backfill. Temporary cantilever shoring walls may use the same values indicated above. For passive resistance in shoring piles, use the value of 687 pcf times the diameter of the soldier pile, times the depth of embedment below the grade excavation in front of the piles.

22. *Retaining Wall Design Parameters – Restrained:* Temporary or permanent site restrained retaining walls or restrained building retaining walls supporting low expansion potential level backfill may utilize a triangular pressure increasing at a rate of 56 pcf for wall design (78 pcf for sloping 2.0:1.0 backfill). The soil pressure produced by any footings, improvements, or any other surcharge placed within a horizontal distance equal to the height of the retaining portion of the wall should be included in the wall design pressure. A conversion factor of 0.47 pcf may be used to convert vertical uniform surcharge loads to lateral uniform pressure behind a restrained retaining wall with level backfill and 0.64

when supporting a 2:1 sloping backfill. The recommended lateral soil pressures are based on the assumption that no loose soils or unstable soil wedges will be retained by the retaining wall. Backfill soils should consist of low expansion potential soils with an EI of less than 50, and should be placed from the heel of the foundation to the ground surface within the wedge formed by a plane at 30° from vertical, and passing by the heel of the foundation and the back face of the retaining wall.

23. *Retaining Wall Seismic Design Pressures:* For seismic design of unrestrained walls over 6 feet in exposed height, we recommend that the seismic pressure increment be taken as a fluid pressure distribution utilizing an equivalent fluid weight of 17 pcf. This seismic increment is waived for restrained basement walls. If the walls are designed as unrestrained walls, then the seismic load should be added to the static soil pressure.

24. *Retaining Wall Drainage:* The preceding design pressures assume that the walls are backfilled with properly compacted low expansion potential materials (Expansion Index less than 50) and that there is sufficient drainage behind the walls to prevent the build-up of hydrostatic pressures from surface water infiltration. We recommend that drainage be provided by a composite drainage material such as J-Drain 200/220 and J-Drain SWD, or equivalent. No perforated pipes or gravel are utilized with the J-Drain system. The drain material should terminate 12 inches below the exterior finish surface where the surface is covered by slabs or 18 inches below the finish surface in landscape areas. Waterproofing should extend from the bottom to the top of the wall.

It is not within the scope of our services to provide quality control oversight for surface or subsurface drainage construction or retaining wall sealing and base of wall drain construction. It is the responsibility of the contractor to verify proper wall sealing, geofabric installation, protection board installation (if needed), drain depth below interior floor or yard surfaces, pipe percent slope to the outlet, etc.

Geotechnical Exploration, Inc. will assume no liability for damage to structures or improvements that is attributable to poor drainage. The architectural plans should clearly indicate that subdrains for any lower-level walls be placed at an elevation at least 1 foot ***below*** the bottom of the lower-level slabs.

25. *OSHA Requirements:* Where not superseded by specific recommendations presented in this report, trenches, excavations and temporary slopes at the subject site should be constructed in accordance with Title 8, Construction Safety Orders, issued by OSHA.

26. *2019 CBC Requirements:* As stated in CBC 2019, Section 1705.6 Soils*:* "*Special inspections and tests of existing site soil conditions, fill placement and load-bearing requirements shall be performed in accordance with this section and Table 1705.6 (see below). The approved geotechnical report and the construction documents prepared by the registered design professionals shall be used to determine compliance. During fill placement, the special inspector shall verify that proper materials and procedures are used in accordance with the provisions of the approved geotechnical report." A summary of* Table 1705.6 "REQUIRED SPECIAL INSPECTIONS AND TESTS OF SOILS" is presented below:

a) *Verify materials below shallow foundations are adequate to achieve the design bearing capacity;*
b) *Verify excavations are extended to proper depth and have reached proper material;*
c) *Perform classification and testing of compacted fill materials;*
d) *Verify use of proper materials, densities and thicknesses during placement and compaction of compacted fill prior to placement of compacted fill, inspect subgrade and verify that site has been prepared properly.*

Section 1705.6 “Soils” statement and Table 1705.6 indicates that it is mandatory that a representative of this firm (responsible engineering firm) perform observations and fill compaction testing during excavation operations to verify that the remedial operations are consistent with the recommendations presented in this report. All grading excavations resulting from the removal of soils should be observed and evaluated by a representative of our firm before they are backfilled.

Quality control grading observation and field density testing for the purpose of documenting that adequate compaction has been achieved and acceptable soils have been utilized to properly support a project applies not only to fill soils supporting primary structures (unless supported by deep foundations or caissons) but all site improvements such as stairways, patios, pools and pool decking, sidewalks, driveways and retaining walls, etc. Observation and testing of utility line trench backfill also reduces the potential for localized settlement of all of the above including all improvements outside of the footprint of primary structures.

Often after primary building pad grading, it is not uncommon for the geotechnical engineer of record to not be notified of grading performed outside the footprint of the project primary structures. As a result, settlement damage of site improvements such as patios, pool and pool decks, exterior landscape walls and walks, and structure access stairways can occur. It is therefore strongly recommended that the project general contractor, grading contractor, and others tasked with completing the project be advised and acknowledge the importance of adequate and comprehensive observation and testing of soils intended to support the project they are working on. The project geotechnical engineer of record must be contacted and requested to provide these services.

The geotechnical engineer of record, in this case ***Geotechnical Exploration, Inc.,*** cannot be held responsible for the costs and time delays associated with the lack of contact and requests for testing services by the client, general contractor, grading contractor or any of the project design team responsible for requesting the required geotechnical services. Requests for services are to be made through our office telephone number (858) 549-7222 and the telephone number of the GEI personnel assigned to the project.

27. *Utility Trench Backfill:* Utility trenches inside the residential buildings may be backfilled in the pipe bedding portion with granular (sand) soils, but they should be capped with on-site properly compacted and moisture conditioned soil. Those trenches should also be backfilled to prevent exterior water infiltration toward the buildings.

28. *Surface Drainage:* The exterior areas outside the buildings and major improvements should be provided with proper surface drainage to prevent runoff accumulation adjacent to their perimeter. For the residential buildings,

a 5 percent positive drainage should be provided within 10 feet of the perimeter as required by the 2019 CBC.

IV. SWIMMING POOL RECOMMENDATIONS

Final swimming pool plans have not been provided to us during the preparation of this report. Final pool plans should be made available for our review.

29. *Pool Design:* The proposed new pools should be founded entirely in cut formational soils or new properly recompacted fill soils compacted to a minimum degree of compaction of 90 percent. Any existing fill soils around the pool shell and in a concrete deck area should be removed and recompacted prior to the placement of new fill soils to support the pool shell. Any imported soils surrounding the swimming pool should be low-expansive. The new pool excavation should be verified by our firm within 48 hours prior to steel and concrete placement.

    The swimming pool shell should be designed for a soil pressure of at least 56 pcf (for on-site low to medium expansive soils). In addition, any above-grade portions of the pool (where applicable) should be designed as a free-standing wall to support 62.4 pcf water pressure. The outer edge of the pool (or spa) should be provided with a foundation setback of at least 7 feet setback from a descending slope face or retaining walls. The portion of the pool within 10 feet of a retaining wall should also be designed to support the water pressure of 62.4 pcf. A seismic soil increment of 17 pcf may be used for the pool shell design as applicable.

We recommend for the pool shell located 10 feet or greater away from a descending slope face to be designed for a soil pressure of 56 pcf. The properly compacted subgrade of the pool deck should be verified by our firm within 48 hours prior to steel and concrete placement. The pool deck should have dowels or continuous steel reinforcement at all joint locations.

30. *Pool Deck:* The pool deck should be reinforced and constructed per the recommendations in this report. The pool deck should have dowels or continuous steel reinforcement at all joint locations to help reduce the potential for vertical differential damage. In addition, the control and isolation joints should be sealed with elastomeric joint sealant. The sealant should be inspected and maintained periodically by the owner. The swimming pool deck and surrounding area should be provided with adequate surface drainage including positive surface drainage and/or functional area drains. Control joints should be provided at least every 15 feet and at reentrant corners.

31. *Pool Deck Subgrade Observations:* The properly compacted subgrade of the pool deck should be verified by our firm within 48 hours prior to steel and concrete placement. Any fill or backfill placed in the pool deck area should be tested during placement at least every 2 feet in vertical thickness.

V. LIMITATIONS

Our conclusions and recommendations are based on available data obtained from our field investigation, background review and laboratory analysis, as well as our experience with similar soils and natural ground materials located in this area of San Diego.

Of necessity, we must assume a certain degree of continuity between exploratory excavations and/or natural exposures. It is, therefore, necessary that all observations, conclusions, and recommendations be verified at the time excavation begins. In the event discrepancies are noted, additional recommendations may be issued, if required.

The work performed and recommendations presented herein are the result of an investigation and analysis that meet the contemporary standard of care in our profession within the City of San Diego. No warranty is provided.

This report should be considered valid for a period of two (2) years, and is subject to review by our firm following that time. If significant modifications are made to the wall plans, especially with respect to the height and location of the proposed wall structure, this report must be presented to us for immediate review and possible revision.

It is not within the scope of our services to provide quality control oversight for surface or subsurface drainage construction or retaining wall sealing and base of wall drain construction. It is the responsibility of the contractor to verify proper wall sealing, geofabric installation, protection board installation (if needed), drain depth below interior floor or yard surfaces, pipe percent slope to the outlet, etc.

It is the responsibility of the owner and/or developer to ensure that the recommendations summarized in this report are carried out in the field operations and that our recommendations for design of this project are incorporated in the project plans. We should be retained to review the final project plans once they are available, to verify that our recommendations are adequately incorporated in the plans. Additional or revised recommendations may be necessary after our review.

This firm does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and we cannot be responsible for the safety of personnel other than our own. The safety of others is the responsibility of the contractor. The contractor should notify the owner if any of the recommended actions presented herein are considered to be unsafe.

The firm of ***Geotechnical Exploration, Inc.*** shall not be held responsible for changes to the physical condition of the property, such as addition of fill soils or changing drainage patterns, which occur subsequent to issuance of this report and the changes are made without our observations, testing, and approval.

Once again, should any questions arise concerning this report, please feel free to contact the undersigned. Reference to our **Job No. 10-9977** will help to expedite a response to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

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R.C.E. 34422/G.E. 2007
Senior Geotechnical Engineer

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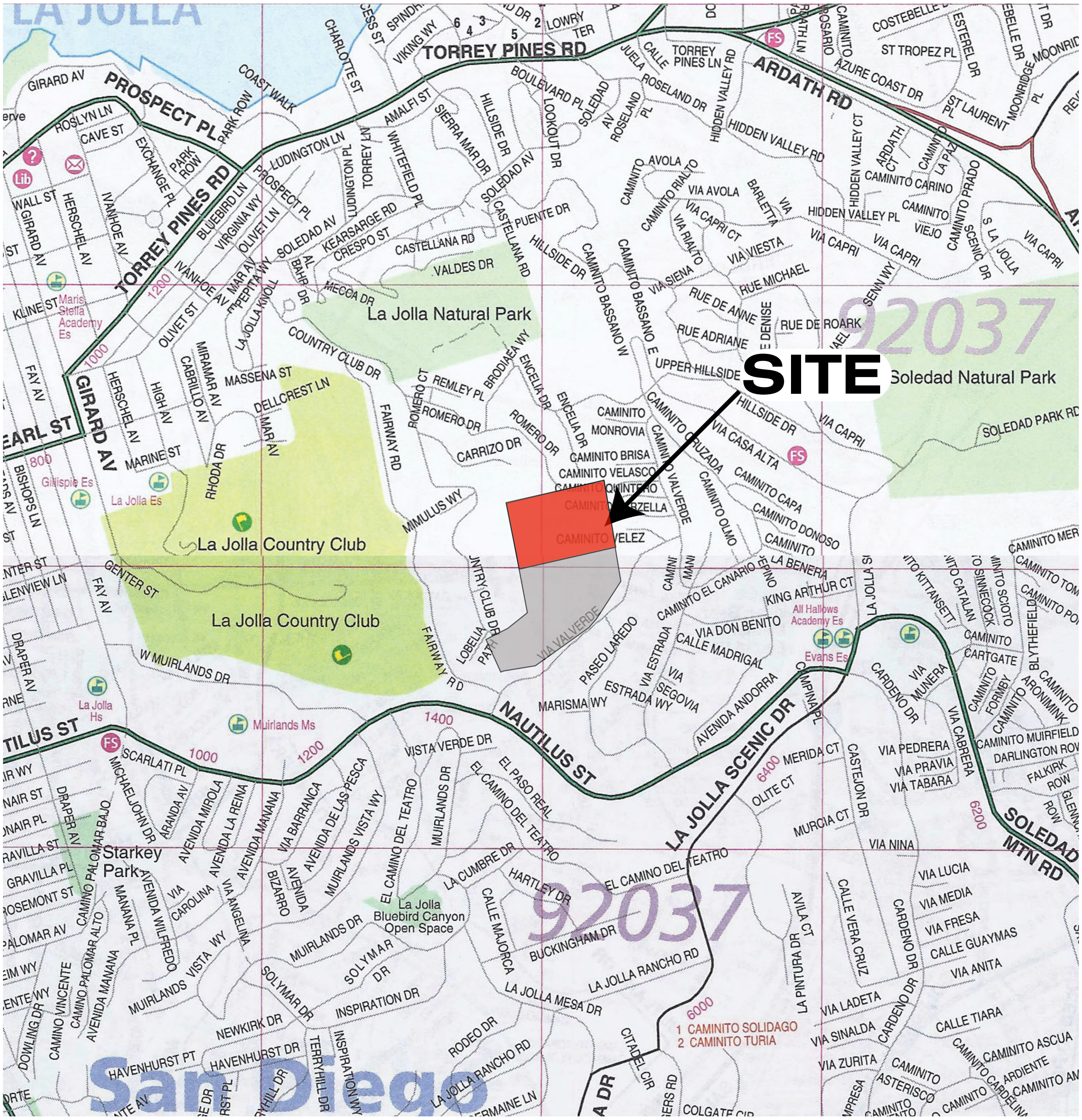
Exp. 3/31/23

VICINITY MAP

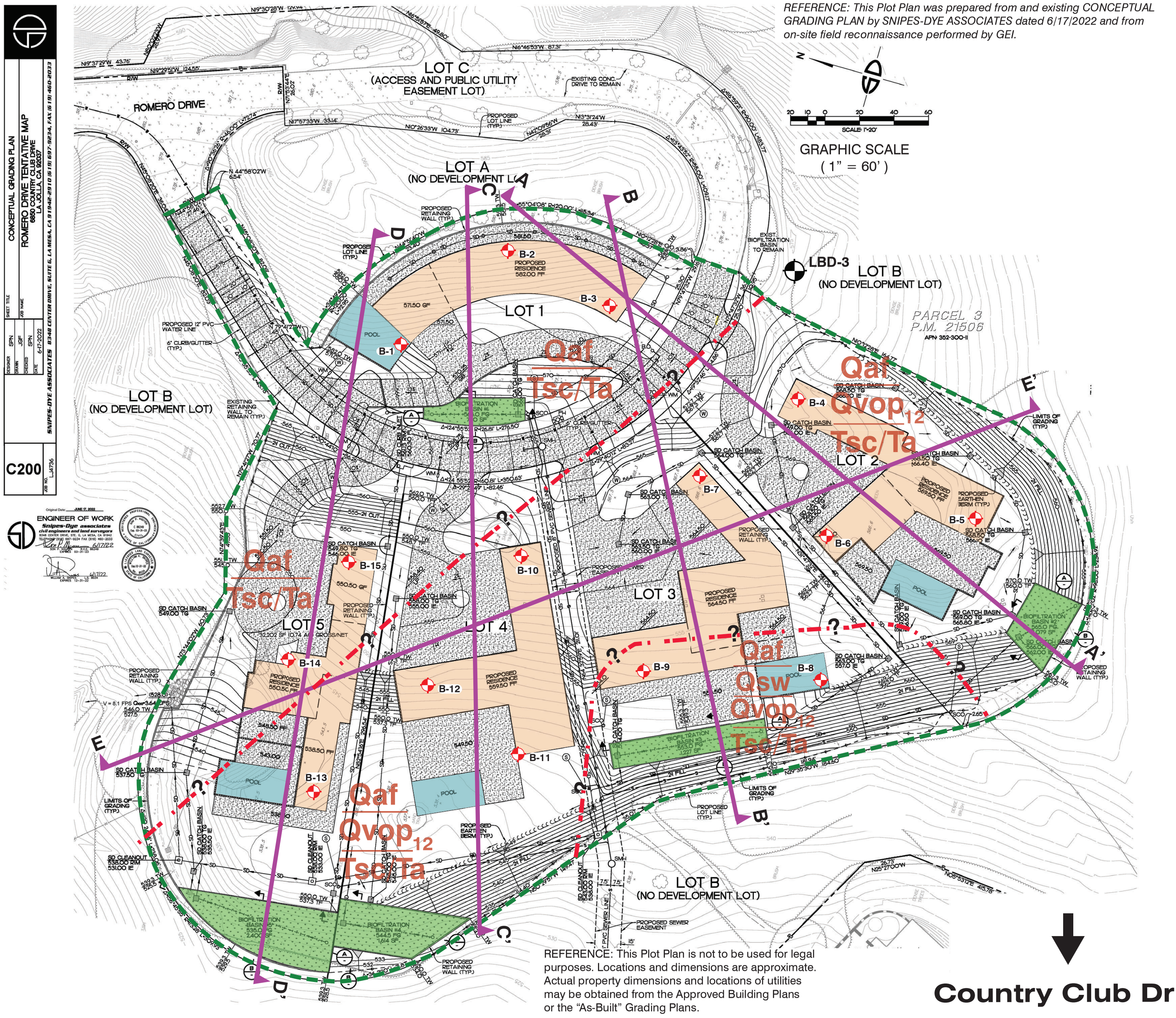


Thomas Bros. Guide San Diego County, pg 1247

The Reserve - Romero Subdivision
Romero Drive
APN 352-300-11-00
La Jolla, CA.

Figure No. I
Job No. 10-9977

Geotechnical Exploration, Inc.



REFERENCE: This Plot Plan was prepared from and existing CONCEPTUAL GRADING PLAN by SNIPES-DYE ASSOCIATES dated 6/17/2022 and from on-site field reconnaissance performed by GEI.

REFERENCE: This Plot Plan is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

LEGEND

⬚	Limits of Geotechnical Investigation and Approximate Limits of Remedial Grading
⊕ B-14	Approximate Location of Exploratory Boring
⊕ LBD-3	Approximate Location of Large Diameter Boring (2011 Investigation)
E ⊔ E'	Approximate Location of Geologic Cross Section
🟧	Approximate Location of Proposed Structure
🟦	Approximate Location of Proposed Swimming Pool
🟩	Approximate Location of Biofiltration Basin

GEOLOGIC UNITS

Qaf	Artificial Fill
Qsw	Slopewash
$Qvop_{12}$	Very Old Paralic Deposits, Unit 12
Tsc/Ta	Scripps Formation/Ardath Shale - undifferentiated
-·?-·?-	Approximate Geologic Contact

PLOT PLAN AND SITE SPECIFIC GEOLOGIC MAP

The Reserve - Romero Subdivision
Romero Drive
APN 325-300-11-00
La Jolla, CA.
Figure No. II
Job No. 10-9977

Geotechnical Exploration, Inc.

October 2022

10-9977-P-10-2022.ai

Geotechnical Exploration, Inc.	**EQUIPMENT:** Track mounted drill rig	**METHOD:** ASTM D1452/1452M-16
	HAMMER: Automatic hammer	**AUGER:** Solid stem, continuous flight
DATE LOGGED: 08/18/2022	**WEIGHT AND DROP HEIGHT:** 140lbs, 30"	**DRILLER:** Native Drilling
WEATHER: Sunny	**DIMENSION & TYPE OF EXCAVATION:** 4.5" diameter boring	
LOGGED BY: SO	**GROUND SURFACE ELEVATION:** ± 572' above mean sea level	
REVIEWED BY: LDR	**GROUNDWATER/SEEPAGE DEPTH:** Not encountered	

DEPTH (feet)	SYMBOL	SAMPLE	**U.S.C.S. CLASSIFICATION, FIELD DESCRIPTION AND GEOLOGIC UNIT** (Grain Size, Relative Density/Consistency, Moisture, Color, Other)	FINES CONTENT (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	RELATIVE COMPACTION (% of MDD)	EXPANSION INDEX	BLOW COUNTS (1.) 6" INCREMENTS	N-VALUE, N_{60}, $(N_1)_{60}$, $(N_1)_{60(CORR)}$ (2.)
0–1	▨	☒	*CLAYEY SAND WITH GRAVEL (SC); fine to medium grained; medium dense; moist; brown to grayish brown; FILL (Qaf).*									
1	▨		*LEAN CLAY (CL); fine grained; stiff to very stiff; moist; yellow; weathered in upper 12"; ARDATH SHALE (Ta).*									
1.5–3	▨	▨									6 6 7	N=13 N_{60}=15
4	▨	■	*@4': very stiff to hard.*								19 50/5" REF	
5			*Bottom of boring at 4'11".* *No groundwater; no caving; backfilled with cuttings.*									
6												
7												
8												
9												
10												
11												
12												
13												

▼	**GROUNDWATER**	**JOB NUMBER:** 10-9977	**EXPLORATORY BORING LOG**
☒	**GRAB (BULK BAG) SAMPLE**	**JOB NAME:** The Reserve - Romero Subdivision	**B-1**
CB	**CARVED BLOCK (CHUNK) SAMPLE**		
■	**MODIFIED CALIFORNIA SAMPLE** (ASTM D3550/D3550-17)	**SITE LOCATION:** Romero Drive, APN 352-300-11-00, La Jolla, CA 92037	**FIGURE NO. IIIa**
▨	**STANDARD PENETRATION TEST** (ASTM D1586/D1586M-18)		
H	**HAND DRIVEN BARREL SAMPLE** (ASTM D4700-15)		

(1.) Blow counts to drive sampler in 6" increments. REF indicates refusal. No Standard Penetration Test correction factors apply when refusal encountered.

(2.) N-value for Standard Penetration Test is the recorded number of blows to drive sampler final 12 inches. N_{60} is the recorded N-value corrected for 60% drill rod energy transfer calculated using Skempton (1986) correction factors, and where applicable, Biringen and Davie (2008) automatic hammer correction factor if energy transfer ratio not known. $(N_1)_{60}$ calculated using Liao and Whitman (1986) overburden correction factor in cohesionless sands. $(N_1)_{60(CORR)}$ calculated using Terzaghi and Peck (1967) dilatancy correction factor for saturated, dense to very dense, silty fine sands and fine sands below the water table.

Geotechnical Exploration, Inc.	**EQUIPMENT:** Track mounted drill rig	**METHOD:** ASTM D1452/1452M-16
	HAMMER: Automatic hammer	**AUGER:** Solid stem, continuous flight
DATE LOGGED: 08/18/2022	**WEIGHT AND DROP HEIGHT:** 140lbs, 30"	**DRILLER:** Native Drilling
WEATHER: Sunny	**DIMENSION & TYPE OF EXCAVATION:** 4.5" diameter boring	
LOGGED BY: SO	**GROUND SURFACE ELEVATION:** ± 585' above mean sea level	
REVIEWED BY: LDR	**GROUNDWATER/SEEPAGE DEPTH:** Not encountered	

DEPTH (feet)	SYMBOL	SAMPLE	**U.S.C.S. CLASSIFICATION, FIELD DESCRIPTION AND GEOLOGIC UNIT** (Grain Size, Relative Density/Consistency, Moisture, Color, Other)	FINES CONTENT (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	RELATIVE COMPACTION (% of MDD)	EXPANSION INDEX	BLOW COUNTS $^{(1.)}$ 6" INCREMENTS	N-VALUE, N_{60}, $(N_1)_{60}$, $(N_1)_{60(CORR)}$ $^{(2.)}$
0–1	▨		*CLAYEY SAND WITH GRAVEL (SC); fine to medium grained; medium dense; moist; brown to grayish brown; FILL (Qaf).*									
1	▨		*LEAN CLAY (CL); fine grained; very stiff to hard; moist; yellow to grayish brown; weathered in upper 6"; ARDATH SHALE (Ta).*									
2	▨	☒										
3	▨	■									13	
4	▨	■		93	17.3	112.6				83	18 50/4" REF	
5	▨	☒										
6			*Bottom of boring at 6'.* *No groundwater; no caving; backfilled with cuttings.*									
7												
8												
9												
10												
11												
12												
13												

▼	**GROUNDWATER**	**JOB NUMBER:** 10-9977	**EXPLORATORY BORING LOG**
☒	**GRAB (BULK BAG) SAMPLE**	**JOB NAME:** The Reserve - Romero Subdivision	**B-2**
CB	**CARVED BLOCK (CHUNK) SAMPLE**		
■	**MODIFIED CALIFORNIA SAMPLE** (ASTM D3550/D3550-17)	**SITE LOCATION:** Romero Drive, APN 352-300-11-00 La Jolla, CA 92037	**FIGURE NO. IIIb**
▨	**STANDARD PENETRATION TEST** (ASTM D1586/D1586M-18)		
H	**HAND DRIVEN BARREL SAMPLE** (ASTM D4700-15)		

(1.) Blow counts to drive sampler in 6" increments. REF indicates refusal. No Standard Penetration Test correction factors apply when refusal encountered.

(2.) N-value for Standard Penetration Test is the recorded number of blows to drive sampler final 12 inches. N_{60} is the recorded N-value corrected for 60% drill rod energy transfer calculated using Skempton (1986) correction factors, and where applicable, Biringen and Davie (2008) automatic hammer correction factor if energy transfer ratio not known. $(N_1)_{60}$ calculated using Liao and Whitman (1986) overburden correction factor in cohesionless sands. $(N_1)_{60(CORR)}$ calculated using Terzaghi and Peck (1967) dilatancy correction factor for saturated, dense to very dense, silty fine sands and fine sands below the water table.

Geotechnical Exploration, Inc.	**EQUIPMENT:** Track mounted drill rig	**METHOD:** ASTM D1452/1452M-16
	HAMMER: Automatic hammer	**AUGER:** Solid stem, continuous flight
DATE LOGGED: 08/18/2022	**WEIGHT AND DROP HEIGHT:** 140lbs, 30"	**DRILLER:** Native Drilling
WEATHER: Sunny	**DIMENSION & TYPE OF EXCAVATION:** 4.5" diameter boring	
LOGGED BY: SO	**GROUND SURFACE ELEVATION:** ± 576' above mean sea level	
REVIEWED BY: LDR	**GROUNDWATER/SEEPAGE DEPTH:** Not encountered	

DEPTH (feet)	SYMBOL	SAMPLE	**U.S.C.S. CLASSIFICATION, FIELD DESCRIPTION AND GEOLOGIC UNIT** (Grain Size, Relative Density/Consistency, Moisture, Color, Other)	FINES CONTENT (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	RELATIVE COMPACTION (% of MDD)	EXPANSION INDEX	BLOW COUNTS (1.) 6" INCREMENTS	N-VALUE, N_{60}, $(N_1)_{60}$, $(N_1)_{60(CORR)}$ (2.)
	▨		*CLAYEY SAND WITH GRAVEL (SC); fine to medium grained; medium dense; moist; brown to grayish brown; FILL (Qaf).*									
1 2	▨	■	*LEAN CLAY (CL); fine grained; very stiff to hard; moist; yellow to grayish brown; weathered in upper 6"; ARDATH SHALE (Ta).*								11 14 45	
3			*Bottom of boring at 2'6". No groundwater; no caving; backfilled with cuttings.*									
4												
5												
6												
7												
8												
9												
10												
11												
12												
13												

▼	**GROUNDWATER**	**JOB NUMBER:** 10-9977	**EXPLORATORY BORING LOG**
☒	**GRAB (BULK BAG) SAMPLE**	**JOB NAME:** The Reserve - Romero Subdivision	**B-3**
CB	**CARVED BLOCK (CHUNK) SAMPLE**		
■	**MODIFIED CALIFORNIA SAMPLE** (ASTM D3550/D3550-17)	**SITE LOCATION:** Romero Drive, APN 352-300-11-00, La Jolla, CA 92037	**FIGURE NO. IIIc**
▨	**STANDARD PENETRATION TEST** (ASTM D1586/D1586M-18)		
H	**HAND DRIVEN BARREL SAMPLE** (ASTM D4700-15)		

(1.) Blow counts to drive sampler in 6" increments. REF indicates refusal. No Standard Penetration Test correction factors apply when refusal encountered.

(2.) N-value for Standard Penetration Test is the recorded number of blows to drive sampler final 12 inches. N_{60} is the recorded N-value corrected for 60% drill rod energy transfer calculated using Skempton (1986) correction factors, and where applicable, Biringen and Davie (2008) automatic hammer correction factor if energy transfer ratio not known. $(N_1)_{60}$ calculated using Liao and Whitman (1986) overburden correction factor in cohesionless sands. $(N_1)_{60(CORR)}$ calculated using Terzaghi and Peck (1967) dilatancy correction factor for saturated, dense to very dense, silty fine sands and fine sands below the water table.

Geotechnical Exploration, Inc.	**EQUIPMENT:** Track mounted drill rig	**METHOD:** ASTM D1452/1452M-16
	HAMMER: Automatic hammer	**AUGER:** Solid stem, continuous flight
DATE LOGGED: 08/18/2022	**WEIGHT AND DROP HEIGHT:** 140lbs, 30"	**DRILLER:** Native Drilling
WEATHER: Sunny	**DIMENSION & TYPE OF EXCAVATION:** 4.5" diameter boring	
LOGGED BY: SO	**GROUND SURFACE ELEVATION:** ± 572' above mean sea level	
REVIEWED BY: LDR	**GROUNDWATER/SEEPAGE DEPTH:** Not encountered	

DEPTH (feet)	SYMBOL	SAMPLE	**U.S.C.S. CLASSIFICATION, FIELD DESCRIPTION AND GEOLOGIC UNIT** (Grain Size, Relative Density/Consistency, Moisture, Color, Other)	FINES CONTENT (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	RELATIVE COMPACTION (% of MDD)	EXPANSION INDEX	BLOW COUNTS (1.) 6" INCREMENTS	N-VALUE, N_{60}, $(N_1)_{60}$, $(N_1)_{60(CORR)}$ (2.)
	▨		*CLAYEY SAND WITH GRAVEL (SC); fine to medium grained; loose to medium dense; moist; dark brown; FILL (Qaf).*									
1	▨	■									3	
	▨	■									3	
2	▨	■									6	
3	▨	☒		38	11.8		9.1	128.5				
4	▨	■									6	
5	▒	■	*SILTY SAND (SM); fine to medium grained; medium dense to dense; moist; brown and reddish brown; moderate cementation; VERY OLD PARALIC DEPOSITS, UNIT 12 (Qvop $_{12}$).*								8	
	▒	■									19	
6	▒	■									19	
	▒	■									34	
	▒	■		19	8.3	120.0					50/4" REF	
7			*Bottom of boring at 6'10".* *No groundwater; no caving; backfilled with cuttings.*									
8												
9												
10												
11												
12												
13												

▼	**GROUNDWATER**	**JOB NUMBER:** 10-9977	**EXPLORATORY BORING LOG**
☒	**GRAB (BULK BAG) SAMPLE**	**JOB NAME:** The Reserve - Romero Subdivision	**B-4**
CB	**CARVED BLOCK (CHUNK) SAMPLE**		
■	**MODIFIED CALIFORNIA SAMPLE** (ASTM D3550/D3550-17)	**SITE LOCATION:** Romero Drive, APN 352-300-11-00, La Jolla, CA 92037	**FIGURE NO. IIId**
▨	**STANDARD PENETRATION TEST** (ASTM D1586/D1586M-18)		
H	**HAND DRIVEN BARREL SAMPLE** (ASTM D4700-15)		

(1.) Blow counts to drive sampler in 6" increments. REF indicates refusal. No Standard Penetration Test correction factors apply when refusal encountered.

(2.) N-value for Standard Penetration Test is the recorded number of blows to drive sampler final 12 inches. N_{60} is the recorded N-value corrected for 60% drill rod energy transfer calculated using Skempton (1986) correction factors, and where applicable, Biringen and Davie (2008) automatic hammer correction factor if energy transfer ratio not known. $(N_1)_{60}$ calculated using Liao and Whitman (1986) overburden correction factor in cohesionless sands. $(N_1)_{60(CORR)}$ calculated using Terzaghi and Peck (1967) dilatancy correction factor for saturated, dense to very dense, silty fine sands and fine sands below the water table.

Geotechnical Exploration, Inc.	**EQUIPMENT:** Track mounted drill rig	**METHOD:** ASTM D1452/1452M-16
	HAMMER: Automatic hammer	**AUGER:** Solid stem, continuous flight
DATE LOGGED: 08/18/2022	**WEIGHT AND DROP HEIGHT:** 140lbs, 30"	**DRILLER:** Native Drilling
WEATHER: Sunny	**DIMENSION & TYPE OF EXCAVATION:** 4.5" diameter boring	
LOGGED BY: SO	**GROUND SURFACE ELEVATION:** ± 567' above mean sea level	
REVIEWED BY: LDR	**GROUNDWATER/SEEPAGE DEPTH:** Not encountered	

DEPTH (feet)	SYMBOL	SAMPLE	**U.S.C.S. CLASSIFICATION, FIELD DESCRIPTION AND GEOLOGIC UNIT** (Grain Size, Relative Density/Consistency, Moisture, Color, Other)	FINES CONTENT (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	RELATIVE COMPACTION (% of MDD)	EXPANSION INDEX	BLOW COUNTS (1.) 6" INCREMENTS	N-VALUE, N_{60}, $(N_1)_{60}$, $(N_1)_{60(CORR)}$ (2.)
1	▨		*CLAYEY SAND WITH GRAVEL (SC); fine to medium grained; loose; moist; dark brown; FILL (Qaf).*									
2 3	░	■	*SILTY SAND (SM); fine to medium grained; dense; moist; reddish brown to gray; moderate cementation; VERY OLD PARALIC DEPOSITS, UNIT 12 (Qvop $_{12}$).*								17 34 50	
4			*Bottom of boring at 3'6". No groundwater; no caving; backfilled with cuttings.*									
5												
6												
7												
8												
9												
10												
11												
12												
13												

▼	**GROUNDWATER**	**JOB NUMBER:** 10-9977	**EXPLORATORY BORING LOG**
☒	**GRAB (BULK BAG) SAMPLE**	**JOB NAME:** The Reserve- Romero Subdivision	**B-5**
CB	**CARVED BLOCK (CHUNK) SAMPLE**		
■	**MODIFIED CALIFORNIA SAMPLE** (ASTM D3550/D3550-17)	**SITE LOCATION:** Romero Drive, APN 352-300-11-00, La Jolla, CA 92037	**FIGURE NO. IIIe**
▨	**STANDARD PENETRATION TEST** (ASTM D1586/D1586M-18)		
H	**HAND DRIVEN BARREL SAMPLE** (ASTM D4700-15)		

(1.) Blow counts to drive sampler in 6" increments. REF indicates refusal. No Standard Penetration Test correction factors apply when refusal encountered.

(2.) N-value for Standard Penetration Test is the recorded number of blows to drive sampler final 12 inches. N_{60} is the recorded N-value corrected for 60% drill rod energy transfer calculated using Skempton (1986) correction factors, and where applicable, Biringen and Davie (2008) automatic hammer correction factor if energy transfer ratio not known. $(N_1)_{60}$ calculated using Liao and Whitman (1986) overburden correction factor in cohesionless sands. $(N_1)_{60(CORR)}$ calculated using Terzaghi and Peck (1967) dilatancy correction factor for saturated, dense to very dense, silty fine sands and fine sands below the water table.

Geotechnical Exploration, Inc.	**EQUIPMENT:** Track mounted drill rig	**METHOD:** ASTM D1452/1452M-16
	HAMMER: Automatic hammer	**AUGER:** Solid stem, continuous flight
DATE LOGGED: 08/18/2022	**WEIGHT AND DROP HEIGHT:** 140lbs, 30"	**DRILLER:** Native Drilling
WEATHER: Sunny	**DIMENSION & TYPE OF EXCAVATION:** 4.5" diameter boring	
LOGGED BY: SO	**GROUND SURFACE ELEVATION:** ± 566' above mean sea level	
REVIEWED BY: LDR	**GROUNDWATER/SEEPAGE DEPTH:** Not encountered	

DEPTH (feet)	SYMBOL	SAMPLE	**U.S.C.S. CLASSIFICATION, FIELD DESCRIPTION AND GEOLOGIC UNIT** (Grain Size, Relative Density/Consistency, Moisture, Color, Other)	FINES CONTENT (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	RELATIVE COMPACTION (% of MDD)	EXPANSION INDEX	BLOW COUNTS (1.) 6" INCREMENTS	N-VALUE, N_{60}, $(N_1)_{60}$, $(N_1)_{60(CORR)}$ (2.)
1	▨	▨	*CLAYEY SAND (SC); fine to medium grained; loose; moist; dark brown; some roots; FILL (Qaf).*								3 3 6	N=9 N_{60}=10
2	▨	▨	*SILTY SAND (SM); fine to medium grained; medium dense; moist; reddish brown to gray; moderate cementation; weathered in upper 6"; VERY OLD PARALIC DEPOSITS, UNIT 12 (Qvop $_{12}$).*									
3											7 9 13	N=22 N_{60}=25
4			*Bottom of boring at 4'.* *No groundwater; no caving; backfilled with cuttings.*									
5												
6												
7												
8												
9												
10												
11												
12												
13												

▼	**GROUNDWATER**	**JOB NUMBER:** 10-9977	**EXPLORATORY BORING LOG**
☒	**GRAB (BULK BAG) SAMPLE**	**JOB NAME:** The Reserve - Romero Subdivision	**B-6**
CB	**CARVED BLOCK (CHUNK) SAMPLE**		
■	**MODIFIED CALIFORNIA SAMPLE** (ASTM D3550/D3550-17)	**SITE LOCATION:** Romero Drive, APN 352-300-11-00, La Jolla, CA 92037	**FIGURE NO. IIIf**
▨	**STANDARD PENETRATION TEST** (ASTM D1586/D1586M-18)		
H	**HAND DRIVEN BARREL SAMPLE** (ASTM D4700-15)		

(1.) Blow counts to drive sampler in 6" increments. REF indicates refusal. No Standard Penetration Test correction factors apply when refusal encountered.

(2.) N-value for Standard Penetration Test is the recorded number of blows to drive sampler final 12 inches. N_{60} is the recorded N-value corrected for 60% drill rod energy transfer calculated using Skempton (1986) correction factors, and where applicable, Biringen and Davie (2008) automatic hammer correction factor if energy transfer ratio not known. $(N_1)_{60}$ calculated using Liao and Whitman (1986) overburden correction factor in cohesionless sands. $(N_1)_{60(CORR)}$ calculated using Terzaghi and Peck (1967) dilatancy correction factor for saturated, dense to very dense, silty fine sands and fine sands below the water table.

Geotechnical Exploration, Inc.	**EQUIPMENT:** Track mounted drill rig	**METHOD:** ASTM D1452/1452M-16
	HAMMER: Automatic hammer	**AUGER:** Solid stem, continuous flight
DATE LOGGED: 08/18/2022	**WEIGHT AND DROP HEIGHT:** 140lbs, 30"	**DRILLER:** Native Drilling
WEATHER: Sunny	**DIMENSION & TYPE OF EXCAVATION:** 4.5" diameter boring	
LOGGED BY: SO	**GROUND SURFACE ELEVATION:** ± 567' above mean sea level	
REVIEWED BY: LDR	**GROUNDWATER/SEEPAGE DEPTH:** Not encountered	

DEPTH (feet)	SYMBOL	SAMPLE	**U.S.C.S. CLASSIFICATION, FIELD DESCRIPTION AND GEOLOGIC UNIT** (Grain Size, Relative Density/Consistency, Moisture, Color, Other)	FINES CONTENT (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	RELATIVE COMPACTION (% of MDD)	EXPANSION INDEX	BLOW COUNTS (1.) 6" INCREMENTS	N-VALUE, N_{60}, $(N_1)_{60}$, $(N_1)_{60(CORR)}$ (2.)
1	▨	▨	*CLAYEY SAND WITH GRAVEL (SC); fine to medium grained; loose; dry to moist; dark brown; some roots; FILL (Qaf).*								5 5 4	N=9 N_{60}=10
2 3 4	▨	▨	*SILTY SAND (SM); fine to medium grained; medium dense to dense; moist; reddish brown to gray; moderate cementation; VERY OLD PARALIC DEPOSITS, UNIT 12 (Qvop $_{12}$).*	21	8.3						11 16 28	N=44 N_{60}=49
5 6 7 8 9 10 11 12 13			*Bottom of boring at 4'6".* *No groundwater; no caving; backfilled with cuttings.*									

▼	**GROUNDWATER**	**JOB NUMBER:** 10-9977	**EXPLORATORY BORING LOG**
☒	**GRAB (BULK BAG) SAMPLE**	**JOB NAME:** The Reserve - Romero Subdivision	**B-7**
CB	**CARVED BLOCK (CHUNK) SAMPLE**		
■	**MODIFIED CALIFORNIA SAMPLE** (ASTM D3550/D3550-17)	**SITE LOCATION:** Romero Drive, APN 352-300-11-00, La Jolla, CA 92037	**FIGURE NO. IIIg**
▨	**STANDARD PENETRATION TEST** (ASTM D1586/D1586M-18)		
H	**HAND DRIVEN BARREL SAMPLE** (ASTM D4700-15)		

(1.) Blow counts to drive sampler in 6" increments. REF indicates refusal. No Standard Penetration Test correction factors apply when refusal encountered.

(2.) N-value for Standard Penetration Test is the recorded number of blows to drive sampler final 12 inches. N_{60} is the recorded N-value corrected for 60% drill rod energy transfer calculated using Skempton (1986) correction factors, and where applicable, Biringen and Davie (2008) automatic hammer correction factor if energy transfer ratio not known. $(N_1)_{60}$ calculated using Liao and Whitman (1986) overburden correction factor in cohesionless sands. $(N_1)_{60(CORR)}$ calculated using Terzaghi and Peck (1967) dilatancy correction factor for saturated, dense to very dense, silty fine sands and fine sands below the water table.

Geotechnical Exploration, Inc.	**EQUIPMENT:** Track mounted drill rig	**METHOD:** ASTM D1452/1452M-16
	HAMMER: Automatic hammer	**AUGER:** Solid stem, continuous flight
DATE LOGGED: 08/18/2022	**WEIGHT AND DROP HEIGHT:** 140lbs, 30"	**DRILLER:** Native Drilling
WEATHER: Sunny	**DIMENSION & TYPE OF EXCAVATION:** 4.5" diameter boring	
LOGGED BY: SO	**GROUND SURFACE ELEVATION:** ± 550' above mean sea level	
REVIEWED BY: LDR	**GROUNDWATER/SEEPAGE DEPTH:** Not encountered	

DEPTH (feet)	SYMBOL	SAMPLE	**U.S.C.S. CLASSIFICATION, FIELD DESCRIPTION AND GEOLOGIC UNIT** (Grain Size, Relative Density/Consistency, Moisture, Color, Other)	FINES CONTENT (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	RELATIVE COMPACTION (% of MDD)	EXPANSION INDEX	BLOW COUNTS (1.) 6" INCREMENTS	N-VALUE, N_{60}, $(N_1)_{60}$, $(N_1)_{60(CORR)}$ (2.)
1		▨	*CLAYEY SAND WITH GRAVEL (SC); fine to medium grained; loose to medium dense; moist; dark brown; some roots; FILL (Qaf).*								3 3 6	N=9 N_{60}=10
2 3		▨	*SILTY SAND (SM); fine to medium grained; medium dense; moist; brown; SLOPEWASH (Qsw).*								6 6 8	N=14 N_{60}=16
4 5 6 7		▨	*SILTY SAND (SM); fine to medium grained; medium dense; moist; reddish brown; VERY OLD PARALIC DEPOSITS, UNIT 12 (Qvop $_{12}$).*								6 9 13	N=22 N_{60}=25
8		▨	*LEAN CLAY (CL); fine grained; stiff to very stiff; moist; yellow to reddish brown to gray; ARDATH SHALE (Ta).*									
			Bottom of boring at 7'6". *No groundwater; no caving; backfilled with cuttings.*									
9												
10												
11												
12												
13												

▼	**GROUNDWATER**	**JOB NUMBER:** 10-9977	**EXPLORATORY BORING LOG**
☒	**GRAB (BULK BAG) SAMPLE**	**JOB NAME:** The Reserve - Romero Subdivision	**B-8**
CB	**CARVED BLOCK (CHUNK) SAMPLE**		
■	**MODIFIED CALIFORNIA SAMPLE** (ASTM D3550/D3550-17)	**SITE LOCATION:** Romero Drive, APN 352-300-11-00, La Jolla, CA 92037	**FIGURE NO. IIIh**
▨	**STANDARD PENETRATION TEST** (ASTM D1586/D1586M-18)		
H	**HAND DRIVEN BARREL SAMPLE** (ASTM D4700-15)		

(1.) Blow counts to drive sampler in 6" increments. REF indicates refusal. No Standard Penetration Test correction factors apply when refusal encountered.

(2.) N-value for Standard Penetration Test is the recorded number of blows to drive sampler final 12 inches. N_{60} is the recorded N-value corrected for 60% drill rod energy transfer calculated using Skempton (1986) correction factors, and where applicable, Biringen and Davie (2008) automatic hammer correction factor if energy transfer ratio not known. $(N_1)_{60}$ calculated using Liao and Whitman (1986) overburden correction factor in cohesionless sands. $(N_1)_{60(CORR)}$ calculated using Terzaghi and Peck (1967) dilatancy correction factor for saturated, dense to very dense, silty fine sands and fine sands below the water table.

Geotechnical Exploration, Inc.	**EQUIPMENT:** Track mounted drill rig	**METHOD:** ASTM D1452/1452M-16
	HAMMER: Automatic hammer	**AUGER:** Solid stem, continuous flight
DATE LOGGED: 08/18/2022	**WEIGHT AND DROP HEIGHT:** 140lbs, 30"	**DRILLER:** Native Drilling
WEATHER: Sunny	**DIMENSION & TYPE OF EXCAVATION:** 4.5" diameter boring	
LOGGED BY: SO	**GROUND SURFACE ELEVATION:** ± 550' above mean sea level	
REVIEWED BY: LDR	**GROUNDWATER/SEEPAGE DEPTH:** Not encountered	

DEPTH (feet)	SYMBOL	SAMPLE	**U.S.C.S. CLASSIFICATION, FIELD DESCRIPTION AND GEOLOGIC UNIT** (Grain Size, Relative Density/Consistency, Moisture, Color, Other)	FINES CONTENT (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	RELATIVE COMPACTION (% of MDD)	EXPANSION INDEX	BLOW COUNTS [1.] 6" INCREMENTS	N-VALUE, N_{60}, $(N_1)_{60}$, $(N_1)_{60(CORR)}$ [2.]
0–0.5			*CLAYEY SAND WITH GRAVEL (SC); fine to medium grained; loose; dry to moist; dark brown; FILL (Qaf).*									
0.5–2		▨									4 3 3	N=6 N_{60}=7
2–2.5		▨									6	
2.5–3.5		▨	*SILTY SAND (SM); fine to medium grained; medium dense; moist; brown; SLOPEWASH (Qsw).*								6 7	N=13 N_{60}=15
3.5–4												
4–4.5		▨									6	
4.5–5.5		▨	*SILTY SAND (SM); fine to medium grained; medium dense; moist; brown to reddish brown; VERY OLD PARALIC DEPOSITS, UNIT 12 (Qvop $_{12}$).*								6 6	N=12 N_{60}=13
5.5–6												
6–7.5		▨									4 8 10	N=18 N_{60}=20
7.5–8			*@7.5': becomes reddish brown to gray, moderate cementation.*									
8–9.5		▨									11 12 16	N=28 N_{60}=31
9.5			*Bottom of boring at 9'6".* *No groundwater; no caving; backfilled with cuttings.*									
10–13												

▼	**GROUNDWATER**	**JOB NUMBER:** 10-9977	**EXPLORATORY BORING LOG**
☒	**GRAB (BULK BAG) SAMPLE**	**JOB NAME:** The Reserve - Romero Subdivision	**B-9**
CB	**CARVED BLOCK (CHUNK) SAMPLE**		
■	**MODIFIED CALIFORNIA SAMPLE** (ASTM D3550/D3550-17)	**SITE LOCATION:** Romero Drive, APN 352-300-11-00, La Jolla, CA 92037	**FIGURE NO. IIIi**
▨	**STANDARD PENETRATION TEST** (ASTM D1586/D1586M-18)		
H	**HAND DRIVEN BARREL SAMPLE** (ASTM D4700-15)		

(1.) Blow counts to drive sampler in 6" increments. REF indicates refusal. No Standard Penetration Test correction factors apply when refusal encountered.

(2.) N-value for Standard Penetration Test is the recorded number of blows to drive sampler final 12 inches. N_{60} is the recorded N-value corrected for 60% drill rod energy transfer calculated using Skempton (1986) correction factors, and where applicable, Biringen and Davie (2008) automatic hammer correction factor if energy transfer ratio not known. $(N_1)_{60}$ calculated using Liao and Whitman (1986) overburden correction factor in cohesionless sands. $(N_1)_{60(CORR)}$ calculated using Terzaghi and Peck (1967) dilatancy correction factor for saturated, dense to very dense, silty fine sands and fine sands below the water table.

Geotechnical Exploration, Inc.	**EQUIPMENT:** Track mounted drill rig	**METHOD:** ASTM D1452/1452M-16
	HAMMER: Automatic hammer	**AUGER:** Solid stem, continuous flight
DATE LOGGED: 08/19/2022	**WEIGHT AND DROP HEIGHT:** 140lbs, 30"	**DRILLER:** Native Drilling
WEATHER: Sunny	**DIMENSION & TYPE OF EXCAVATION:** 4.5" diameter boring	
LOGGED BY: SO	**GROUND SURFACE ELEVATION:** ± 567' above mean sea level	
REVIEWED BY: LDR	**GROUNDWATER/SEEPAGE DEPTH:** Not encountered	

DEPTH (feet)	SYMBOL	SAMPLE	**U.S.C.S. CLASSIFICATION, FIELD DESCRIPTION AND GEOLOGIC UNIT** (Grain Size, Relative Density/Consistency, Moisture, Color, Other)	FINES CONTENT (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	RELATIVE COMPACTION (% of MDD)	EXPANSION INDEX	BLOW COUNTS (1.) 6" INCREMENTS	N-VALUE, N_{60}, $(N_1)_{60}$, $(N_1)_{60(CORR)}$ (2.)
1		▨	*CLAYEY SAND WITH GRAVEL (SC); fine to medium grained; loose; very moist; dark brown; some roots; FILL (Qaf).*								2 3 3	N=6 N_{60}=7
2 3			*SILTY SAND (SM); fine to medium grained; medium dense; moist; brown; SLOPEWASH (Qsw).*									
4 5		▨	*SILTY SAND (SM); fine to medium grained; dense to very dense; moist; reddish brown to gray; VERY OLD PARALIC DEPOSITS, UNIT 12 ($Qvop_{12}$).*								16 18 32	N=50 N_{60}=56
6 7			*@6.5': becomes moderately cemented.*									
8 9		▨		21	8.3						29 37 50	N=87 N_{60}=97
10 11 12 13			*Bottom of boring at 9'6". No groundwater; no caving; backfilled with cuttings.*									

▼	**GROUNDWATER**	**JOB NUMBER:** 10-9977	**EXPLORATORY BORING LOG**
☒	**GRAB (BULK BAG) SAMPLE**	**JOB NAME:** The Reserve - Romero Subdivision	**B-10**
CB	**CARVED BLOCK (CHUNK) SAMPLE**		
■	**MODIFIED CALIFORNIA SAMPLE** (ASTM D3550/D3550-17)	**SITE LOCATION:** Romero Drive, APN 352-300-11-00, La Jolla, CA 92037	**FIGURE NO. IIIj**
▨	**STANDARD PENETRATION TEST** (ASTM D1586/D1586M-18)		
H	**HAND DRIVEN BARREL SAMPLE** (ASTM D4700-15)		

(1.) Blow counts to drive sampler in 6" increments. REF indicates refusal. No Standard Penetration Test correction factors apply when refusal encountered.

(2.) N-value for Standard Penetration Test is the recorded number of blows to drive sampler final 12 inches. N_{60} is the recorded N-value corrected for 60% drill rod energy transfer calculated using Skempton (1986) correction factors, and where applicable, Biringen and Davie (2008) automatic hammer correction factor if energy transfer ratio not known. $(N_1)_{60}$ calculated using Liao and Whitman (1986) overburden correction factor in cohesionless sands. $(N_1)_{60(CORR)}$ calculated using Terzaghi and Peck (1967) dilatancy correction factor for saturated, dense to very dense, silty fine sands and fine sands below the water table.

Geotechnical Exploration, Inc.	**EQUIPMENT:** Track mounted drill rig	**METHOD:** ASTM D1452/1452M-16
	HAMMER: Automatic hammer	**AUGER:** Solid stem, continuous flight
DATE LOGGED: 08/19/2022	**WEIGHT AND DROP HEIGHT:** 140lbs, 30"	**DRILLER:** Native Drilling
WEATHER: Sunny	**DIMENSION & TYPE OF EXCAVATION:** 4.5" diameter boring	
LOGGED BY: SO	**GROUND SURFACE ELEVATION:** ± 544' above mean sea level	
REVIEWED BY: LDR	**GROUNDWATER/SEEPAGE DEPTH:** Not encountered	

DEPTH (feet)	SYMBOL	SAMPLE	**U.S.C.S. CLASSIFICATION, FIELD DESCRIPTION AND GEOLOGIC UNIT** (Grain Size, Relative Density/Consistency, Moisture, Color, Other)	FINES CONTENT (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	RELATIVE COMPACTION (% of MDD)	EXPANSION INDEX	BLOW COUNTS (1.) 6" INCREMENTS	N-VALUE, N_{60}, $(N_1)_{60}$, $(N_1)_{60(CORR)}$ (2.)
1			*CLAYEY SAND WITH GRAVEL (SC); fine to medium grained; medium dense; moist; reddish brown; FILL (Qaf).*									
2 3		▨	*SILTY SAND (SM); fine to medium grained; medium dense; moist; reddish brown; VERY OLD PARALIC DEPOSITS, UNIT 12 (Qvop $_{12}$).*								12 12 12	N=24 N_{60}=27
4 5		▨	*@4.5': becomes weakly cemented.*									
6		▨	*LEAN CLAY (CL); fine grained; very stiff; moist; reddish brown and gray; ARDATH SHALE (Ta).*	97	17.3						10 11 15	N=26 N_{60}=29
7 8 9 10 11 12 13			*Bottom of boring at 6'6".* *No groundwater; no caving; backfilled with cuttings.*									

▼	**GROUNDWATER**
☒	**GRAB (BULK BAG) SAMPLE**
CB	**CARVED BLOCK (CHUNK) SAMPLE**
■	**MODIFIED CALIFORNIA SAMPLE** (ASTM D3550/D3550-17)
▨	**STANDARD PENETRATION TEST** (ASTM D1586/D1586M-18)
H	**HAND DRIVEN BARREL SAMPLE** (ASTM D4700-15)

JOB NUMBER: 10-9977

JOB NAME:
The Reserve - Romero Subdivision

SITE LOCATION:
Romero Drive, APN 352-300-11-00,
La Jolla, CA 92037

EXPLORATORY BORING LOG

B-11

FIGURE NO. IIIk

(1.) Blow counts to drive sampler in 6" increments. REF indicates refusal. No Standard Penetration Test correction factors apply when refusal encountered.

(2.) N-value for Standard Penetration Test is the recorded number of blows to drive sampler final 12 inches. N_{60} is the recorded N-value corrected for 60% drill rod energy transfer calculated using Skempton (1986) correction factors, and where applicable, Biringen and Davie (2008) automatic hammer correction factor if energy transfer ratio not known. $(N_1)_{60}$ calculated using Liao and Whitman (1986) overburden correction factor in cohesionless sands. $(N_1)_{60(CORR)}$ calculated using Terzaghi and Peck (1967) dilatancy correction factor for saturated, dense to very dense, silty fine sands and fine sands below the water table.

Geotechnical Exploration, Inc.	**EQUIPMENT:** Track mounted drill rig	**METHOD:** ASTM D1452/1452M-16
	HAMMER: Automatic hammer	**AUGER:** Solid stem, continuous flight
DATE LOGGED: 08/19/2022	**WEIGHT AND DROP HEIGHT:** 140lbs, 30"	**DRILLER:** Native Drilling
WEATHER: Sunny	**DIMENSION & TYPE OF EXCAVATION:** 4.5" diameter boring	
LOGGED BY: SO	**GROUND SURFACE ELEVATION:** ± 550' above mean sea level	
REVIEWED BY: LDR	**GROUNDWATER/SEEPAGE DEPTH:** Not encountered	

DEPTH (feet)	SYMBOL	SAMPLE	**U.S.C.S. CLASSIFICATION, FIELD DESCRIPTION AND GEOLOGIC UNIT** (Grain Size, Relative Density/Consistency, Moisture, Color, Other)	FINES CONTENT (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	RELATIVE COMPACTION (% of MDD)	EXPANSION INDEX	BLOW COUNTS (1.) 6" INCREMENTS	N-VALUE, N_{60}, $(N_1)_{60}$, $(N_1)_{60(CORR)}$ (2.)
0–1.5	▨	☒	*LEAN CLAY (CL); fine to medium grained; soft to firm; moist; dark brown; FILL (Qaf).*									
1.5–3	▨	■		99	11.1						5 6 10	
3–4	▨	▒									2 6 9	N=15 N_{60}=17
4–4.5	▨	▒	*LEAN CLAY (CL); fine grained; very stiff; moist; yellow to reddish brown to gray; ARDATH SHALE (Ta).*									
4.5–5	▨											
5–6.5	▨	▒									8 10 13	N=23 N_{60}=26
6.5			*Bottom of boring at 6'6".* *No groundwater; no caving; backfilled with cuttings.*									
7												
8												
9												
10												
11												
12												
13												

▼	**GROUNDWATER**	**JOB NUMBER:** 10-9977	**EXPLORATORY BORING LOG**
☒	**GRAB (BULK BAG) SAMPLE**	**JOB NAME:** The Reserve - Romero Subdivision	**B-12**
CB	**CARVED BLOCK (CHUNK) SAMPLE**		
■	**MODIFIED CALIFORNIA SAMPLE** (ASTM D3550/D3550-17)	**SITE LOCATION:** Romero Drive, APN 352-300-11-00, La Jolla, CA 92037	**FIGURE NO. III/**
▒	**STANDARD PENETRATION TEST** (ASTM D1586/D1586M-18)		
H	**HAND DRIVEN BARREL SAMPLE** (ASTM D4700-15)		

(1.) Blow counts to drive sampler in 6" increments. REF indicates refusal. No Standard Penetration Test correction factors apply when refusal encountered.

(2.) N-value for Standard Penetration Test is the recorded number of blows to drive sampler final 12 inches. N_{60} is the recorded N-value corrected for 60% drill rod energy transfer calculated using Skempton (1986) correction factors, and where applicable, Biringen and Davie (2008) automatic hammer correction factor if energy transfer ratio not known. $(N_1)_{60}$ calculated using Liao and Whitman (1986) overburden correction factor in cohesionless sands. $(N_1)_{60(CORR)}$ calculated using Terzaghi and Peck (1967) dilatancy correction factor for saturated, dense to very dense, silty fine sands and fine sands below the water table.

Geotechnical Exploration, Inc.

EQUIPMENT: Track mounted drill rig
METHOD: ASTM D1452/1452M-16
HAMMER: Automatic hammer
AUGER: Solid stem, continuous flight
DATE LOGGED: 08/19/2022
WEIGHT AND DROP HEIGHT: 140lbs, 30"
DRILLER: Native Drilling
WEATHER: Sunny
DIMENSION & TYPE OF EXCAVATION: 4.5" diameter boring
LOGGED BY: SO
GROUND SURFACE ELEVATION: ± 541' above mean sea level
REVIEWED BY: LDR
GROUNDWATER/SEEPAGE DEPTH: Not encountered

DEPTH (feet)	SYMBOL	SAMPLE	**U.S.C.S. CLASSIFICATION, FIELD DESCRIPTION AND GEOLOGIC UNIT** (Grain Size, Relative Density/Consistency, Moisture, Color, Other)	FINES CONTENT (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	RELATIVE COMPACTION (% of MDD)	EXPANSION INDEX	BLOW COUNTS [1.] 6" INCREMENTS	N-VALUE, N_{60}, $(N_1)_{60}$, $(N_1)_{60(CORR)}$ [2.]
0–1	SC		*CLAYEY SAND WITH GRAVEL (SC); fine to medium grained; loose to medium dense; moist; dark brown to reddish brown; FILL (Qaf).*									
1–2	SC	☒										
2.5–4	SC	■									8 9 11	
4–4.5	SC	▨									7	
4.5–5.5	SM	▨	*SILTY SAND (SM); fine to medium grained; medium dense to dense; moist; reddish brown; VERY OLD PARALIC DEPOSITS, UNIT 12 (Qvop $_{12}$).*								8 14	N=22 N_{60}=25
7–8.5	SM	▨									16 19 24	N=43 N_{60}=48
8.5–10	SM	■									26 24 50/ 5.5" REF	
12–13.5	SM	▨									14 19 20	N=39 N_{60}=49
13.5–15	SM	▨									11 10 11	N=21 N_{60}=27
15			*Bottom of boring at 15'. No groundwater; no caving; backfilled with cuttings.*									

▼ **GROUNDWATER**
☒ **GRAB (BULK BAG) SAMPLE**
CB **CARVED BLOCK (CHUNK) SAMPLE**
■ **MODIFIED CALIFORNIA SAMPLE** (ASTM D3550/D3550-17)
▨ **STANDARD PENETRATION TEST** (ASTM D1586/D1586M-18)
H **HAND DRIVEN BARREL SAMPLE** (ASTM D4700-15)

JOB NUMBER: 10-9977
JOB NAME:
The Reserve - Romero Subdivision
SITE LOCATION:
Romero Drive, APN 352-300-11-00,
La Jolla, CA 92037

EXPLORATORY BORING LOG

B-13

FIGURE NO. IIIm

(1.) Blow counts to drive sampler in 6" increments. REF indicates refusal. No Standard Penetration Test correction factors apply when refusal encountered.

(2.) N-value for Standard Penetration Test is the recorded number of blows to drive sampler final 12 inches. N_{60} is the recorded N-value corrected for 60% drill rod energy transfer calculated using Skempton (1986) correction factors, and where applicable, Biringen and Davie (2008) automatic hammer correction factor if energy transfer ratio not known. $(N_1)_{60}$ calculated using Liao and Whitman (1986) overburden correction factor in cohesionless sands. $(N_1)_{60(CORR)}$ calculated using Terzaghi and Peck (1967) dilatancy correction factor for saturated, dense to very dense, silty fine sands and fine sands below the water table.

Geotechnical Exploration, Inc.	**EQUIPMENT:** Track mounted drill rig	**METHOD:** ASTM D1452/1452M-16
	HAMMER: Automatic hammer	**AUGER:** Solid stem, continuous flight
DATE LOGGED: 08/19/2022	**WEIGHT AND DROP HEIGHT:** 140lbs, 30"	**DRILLER:** Native Drilling
WEATHER: Sunny	**DIMENSION & TYPE OF EXCAVATION:** 4.5" diameter boring	
LOGGED BY: SO	**GROUND SURFACE ELEVATION:** ± 545' above mean sea level	
REVIEWED BY: LDR	**GROUNDWATER/SEEPAGE DEPTH:** Not encountered	

DEPTH (feet)	SYMBOL	SAMPLE	**U.S.C.S. CLASSIFICATION, FIELD DESCRIPTION AND GEOLOGIC UNIT** (Grain Size, Relative Density/Consistency, Moisture, Color, Other)	FINES CONTENT (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	RELATIVE COMPACTION (% of MDD)	EXPANSION INDEX	BLOW COUNTS (1.) 6" INCREMENTS	N-VALUE, N_{60}, $(N_1)_{60}$, $(N_1)_{60(CORR)}$ (2.)
1	▨	☒	*LEAN CLAY WITH SAND (CL); fine to medium grained; soft; moist; dark brown to grayish brown; FILL (Qaf).*	84	8.3		10.0	124.7		55		
2	▨	■			14.5	103.1			83		7 7 8	
3 4	▨	▨									3 4 4	N=8 N_{60}=9
	▨		*@4.5': becomes moist to very moist.*									
5 6	▨	■	*LEAN CLAY (CL); fine grained; very stiff; moist; yellow to reddish brown to gray; ARDATH SHALE (Ta).*								8 9 14	
7	▨	▨									6 8 12	N=20 N_{60}=22
8 9 10 11 12 13			*Bottom of boring at 8'.* *No groundwater; no caving; backfilled with cuttings.*									

▽	**GROUNDWATER**	**JOB NUMBER:** 10-9977	**EXPLORATORY BORING LOG**
☒	**GRAB (BULK BAG) SAMPLE**	**JOB NAME:** The Reserve - Romero Subdivision	**B-14**
CB	**CARVED BLOCK (CHUNK) SAMPLE**		
■	**MODIFIED CALIFORNIA SAMPLE** (ASTM D3550/D3550-17)	**SITE LOCATION:** Romero Drive, APN 352-300-11-00, La Jolla, CA 92037	**FIGURE NO. IIIn**
▨	**STANDARD PENETRATION TEST** (ASTM D1586/D1586M-18)		
H	**HAND DRIVEN BARREL SAMPLE** (ASTM D4700-15)		

(1.) Blow counts to drive sampler in 6" increments. REF indicates refusal. No Standard Penetration Test correction factors apply when refusal encountered.

(2.) N-value for Standard Penetration Test is the recorded number of blows to drive sampler final 12 inches. N_{60} is the recorded N-value corrected for 60% drill rod energy transfer calculated using Skempton (1986) correction factors, and where applicable, Biringen and Davie (2008) automatic hammer correction factor if energy transfer ratio not known. $(N_1)_{60}$ calculated using Liao and Whitman (1986) overburden correction factor in cohesionless sands. $(N_1)_{60(CORR)}$ calculated using Terzaghi and Peck (1967) dilatancy correction factor for saturated, dense to very dense, silty fine sands and fine sands below the water table.

Geotechnical Exploration, Inc.	**EQUIPMENT:** Track mounted drill rig	**METHOD:** ASTM D1452/1452M-16
	HAMMER: Automatic hammer	**AUGER:** Solid stem, continuous flight
DATE LOGGED: 08/19/2022	**WEIGHT AND DROP HEIGHT:** 140lbs, 30"	**DRILLER:** Native Drilling
WEATHER: Sunny	**DIMENSION & TYPE OF EXCAVATION:** 4.5" diameter boring	
LOGGED BY: SO	**GROUND SURFACE ELEVATION:** ± 555' above mean sea level	
REVIEWED BY: LDR	**GROUNDWATER/SEEPAGE DEPTH:** Not encountered	

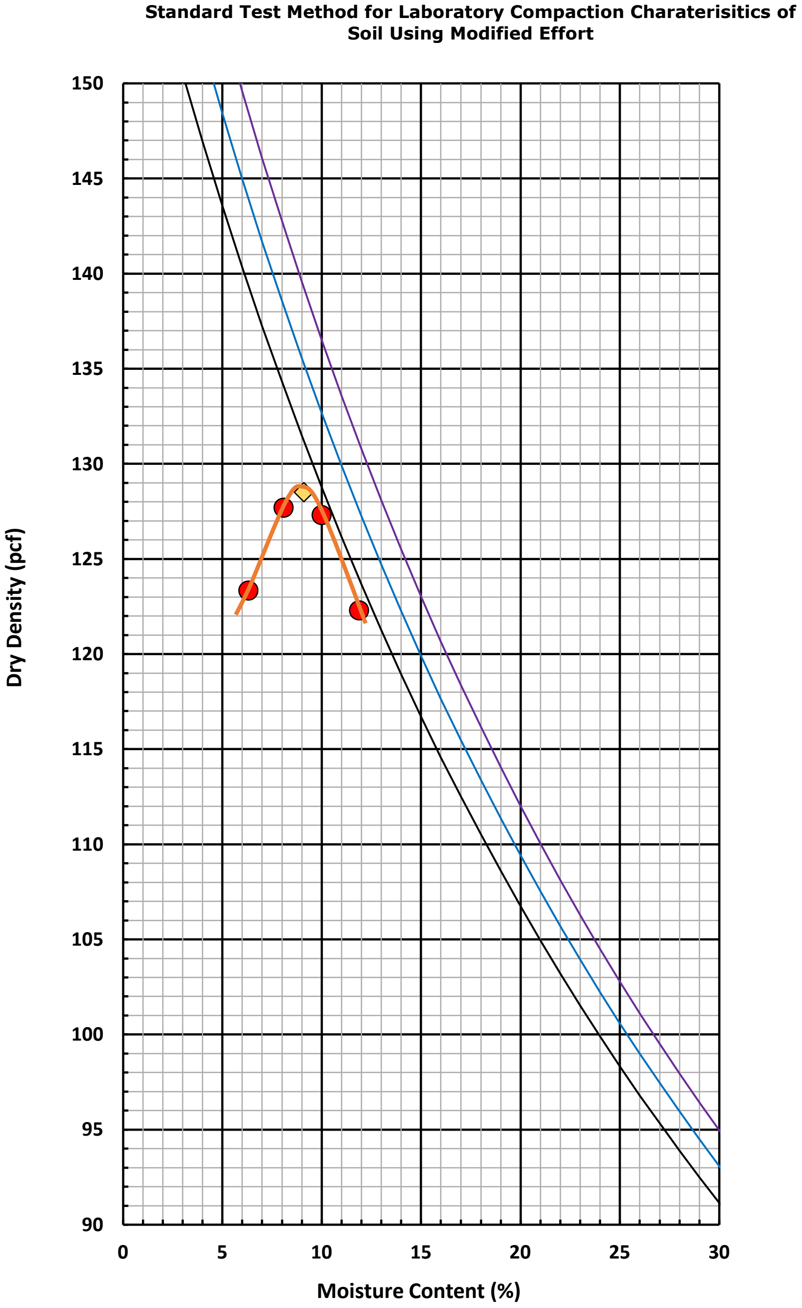
DEPTH (feet)	SYMBOL	SAMPLE	**U.S.C.S. CLASSIFICATION, FIELD DESCRIPTION AND GEOLOGIC UNIT** (Grain Size, Relative Density/Consistency, Moisture, Color, Other)	FINES CONTENT (%)	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	RELATIVE COMPACTION (% of MDD)	EXPANSION INDEX	BLOW COUNTS [1] 6" INCREMENTS	N-VALUE, N_{60}, $(N_1)_{60}$, $(N_1)_{60(CORR)}$ [2]
	▨		*LEAN CLAY WITH SAND (CL); fine to medium grained; soft; moist; dark brown; FILL (Qaf).*									
1	▨											
2	▨	☒	*LEAN CLAY (CL); fine grained; very stiff to hard; moist; yellow to reddish brown to gray; ARDATH SHALE (Ta).*									
3	▨											
4	▨											
5	▨	▨									12	
6	▨	▨		96	8.9						15 19	N=34 N_{60}=38
7			*Bottom of boring at 6'6".* *No groundwater; no caving; backfilled with cuttings.*									
8												
9												
10												
11												
12												
13												

▼	**GROUNDWATER**	**JOB NUMBER:** 10-9977	**EXPLORATORY BORING LOG**
☒	**GRAB (BULK BAG) SAMPLE**	**JOB NAME:** The Reserve - Romero Subdivision	**B-15**
CB	**CARVED BLOCK (CHUNK) SAMPLE**		
■	**MODIFIED CALIFORNIA SAMPLE** (ASTM D3550/D3550-17)	**SITE LOCATION:** Romero Drive, APN 352-300-11-00, La Jolla, CA 92037	**FIGURE NO. IIIo**
▨	**STANDARD PENETRATION TEST** (ASTM D1586/D1586M-18)		
H	**HAND DRIVEN BARREL SAMPLE** (ASTM D4700-15)		

(1.) Blow counts to drive sampler in 6" increments. REF indicates refusal. No Standard Penetration Test correction factors apply when refusal encountered.

(2.) N-value for Standard Penetration Test is the recorded number of blows to drive sampler final 12 inches. N_{60} is the recorded N-value corrected for 60% drill rod energy transfer calculated using Skempton (1986) correction factors, and where applicable, Biringen and Davie (2008) automatic hammer correction factor if energy transfer ratio not known. $(N_1)_{60}$ calculated using Liao and Whitman (1986) overburden correction factor in cohesionless sands. $(N_1)_{60(CORR)}$ calculated using Terzaghi and Peck (1967) dilatancy correction factor for saturated, dense to very dense, silty fine sands and fine sands below the water table.

LABORATORY TEST RESULTS



Source of Material	Boring B-4
Depth	2.5-4 ft
U.S.C.S.	CLAYEY SAND (SC)
Test Method	ASTM D1557-12(2021) Method A

TEST RESULTS	
Maximum Dry Density (PCF)	128.5
Optimum Moisture Content (%)	9.1
Coarse Material +No. 4. Sieve (%)	
Corrected Maximum Dry Density (pcf)	
Corrected Optimum Moisture Content (%)	

Curves of 100% Saturation for

Specific Gravity equal to:

2.80

2.70

2.60

Geotechnical Exploration, Inc.

JOB NUMBER: 10-9977

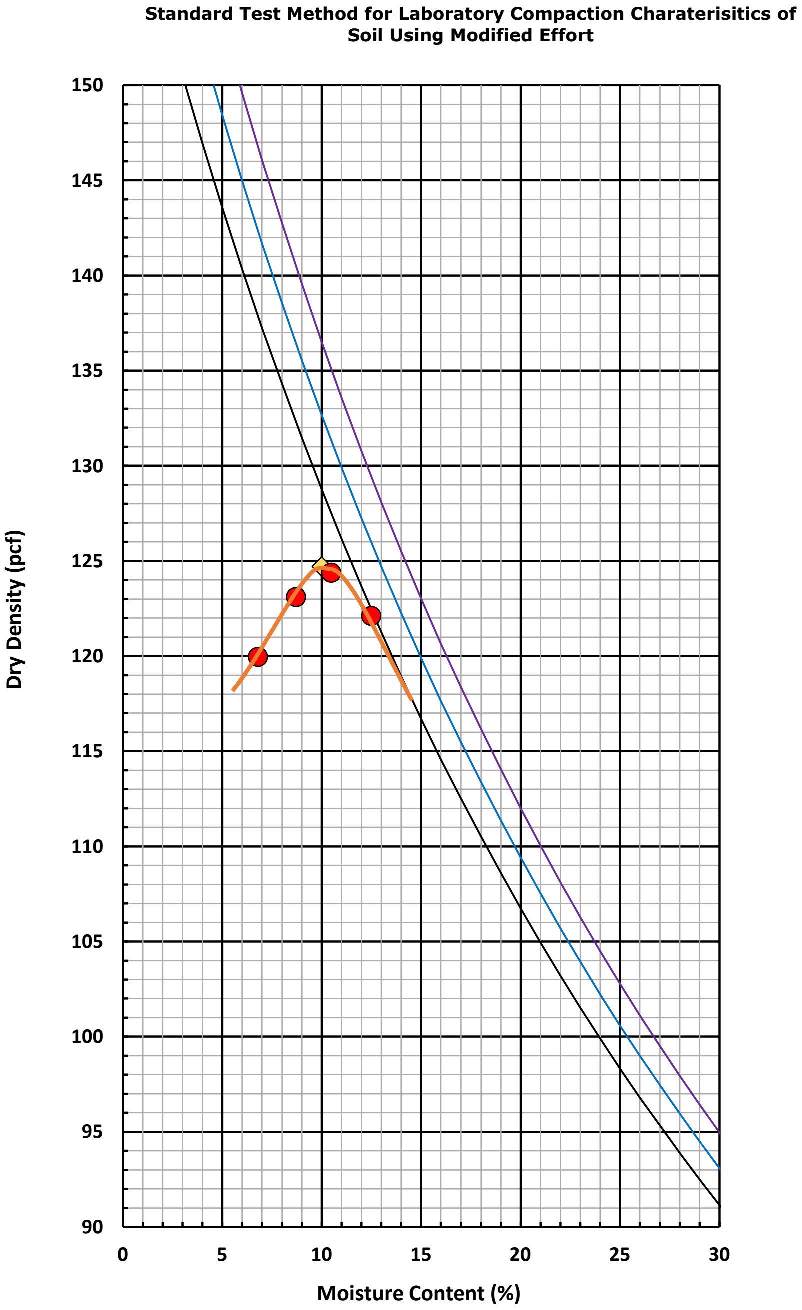
JOB NAME: The Reserve - Romero Subdivision

SITE LOCATION:
Romero Drive, APN 352-300-11,
La Jolla, CA

MOISTURE-DENSITY RELATIONSHIP

FIGURE NO. IVa

LABORATORY TEST RESULTS



Source of Material	Boring B-14
Depth	0-1.5 ft
U.S.C.S.	LEAN CLAY WITH SAND (CL)
Test Method	ASTM D1557-12(2021) Method A

TEST RESULTS	
Maximum Dry Density (PCF)	124.7
Optimum Moisture Content (%)	10.0
Coarse Material +No. 4. Sieve (%)	
Corrected Maximum Dry Density (pcf)	
Corrected Optimum Moisture Content (%)	

Curves of 100% Saturation for

Specific Gravity equal to:

2.80

2.70

2.60

Geotechnical Exploration, Inc.

JOB NUMBER: 10-9977

JOB NAME: The Reserve - Romero Subdivision

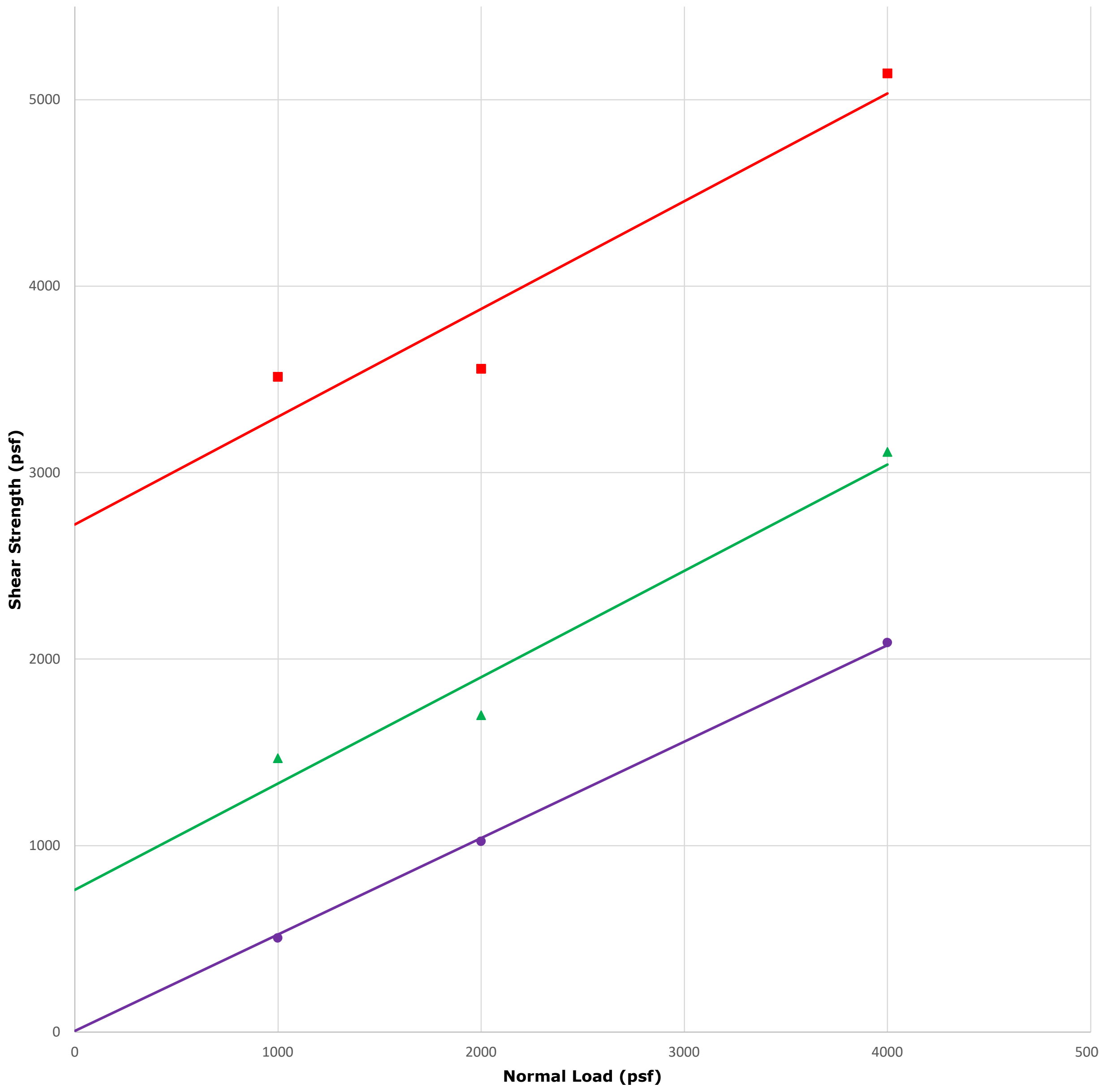
SITE LOCATION:
Romero Drive, APN 352-300-11
La Jolla, CA

MOISTURE-DENSITY RELATIONSHIP

FIGURE NO. IVb

LABORATORY TEST RESULTS

Standard Test Method for Direct Shear Tests of Soils



Symbol	—■—	—▲—	—●—		
Source of Material	Boring B-2	Boring B-2	Boring B-4		
Depth	3-4.5ft	3-4.5ft	2.5-4ft		
U.S.C.S.	LEAN CLAY (CL)	LEAN CLAY (CL)	CLAYEY SAND (SC)		
Test Method	ASTM D3080, Unsaturated, Peak, Undisturbed In-Place Density and Moisture	ASTM D3080, Unsaturated, Residual, Undisturbed In-Place Density and Moisture	ASTM D3080, Saturated, Peak, Remolded to 90% density		
Friction Angle φ (degrees)	30	29.7	27.9		
Cohesion (psf)	2721.6	763.2	0		

Geotechnical Exploration, Inc.

JOB NUMBER: 10-9977

JOB NAME: The Reserve - Romero Subdivision

SITE LOCATION:
Romero Drive, APN 352-300-11,
La Jolla, CA

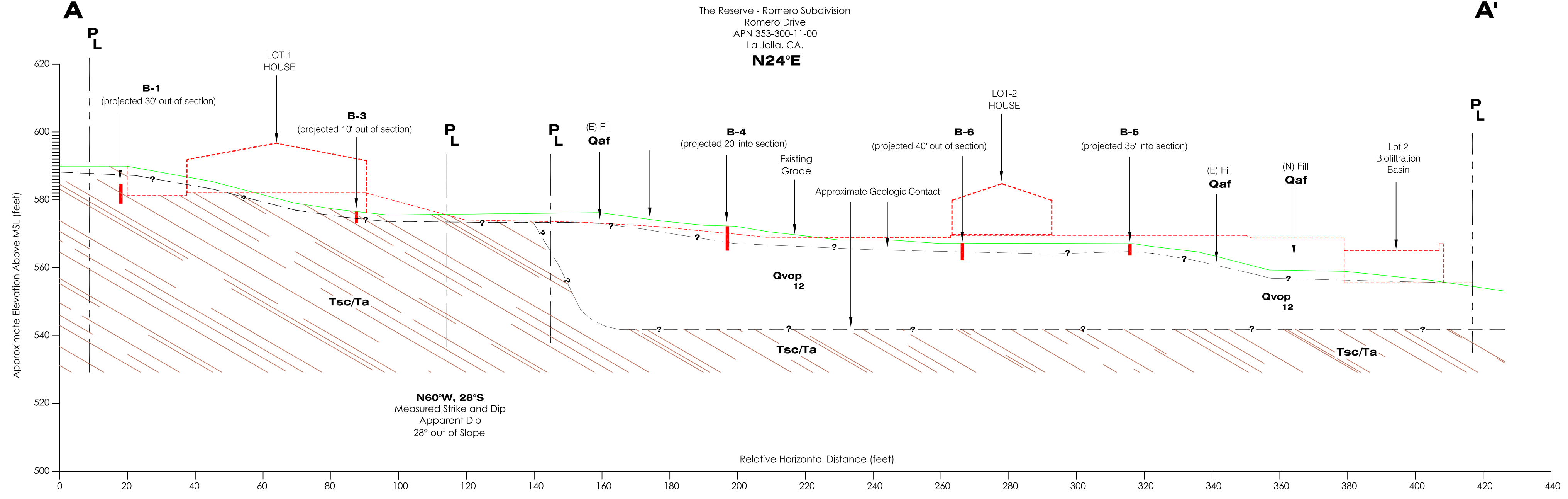
DIRECT SHEAR

FIGURE NO. IVc

GEOLOGIC CROSS SECTION A-A'

The Reserve - Romero Subdivision
Romero Drive
APN 353-300-11-00
La Jolla, CA.

N24°E



Scale: 1" = 20'
(Horizontal and Vertical)

(N) Qaf	New Artificial Fill	$Qvop_{12}$	Very Old Paralic Deposits, Unit 12	_?_ ?_	Approximate Geologic Contact
(E) Qaf	Existing Artificial Fill	Tsc	Scripps Formation	🟥	Proposed Grade and Structures
Qsw	Slopewash	Ta	Ardath Shale		

NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

Figure No.Va
Job No. 10-9977

Geotechnical Exploration, Inc.

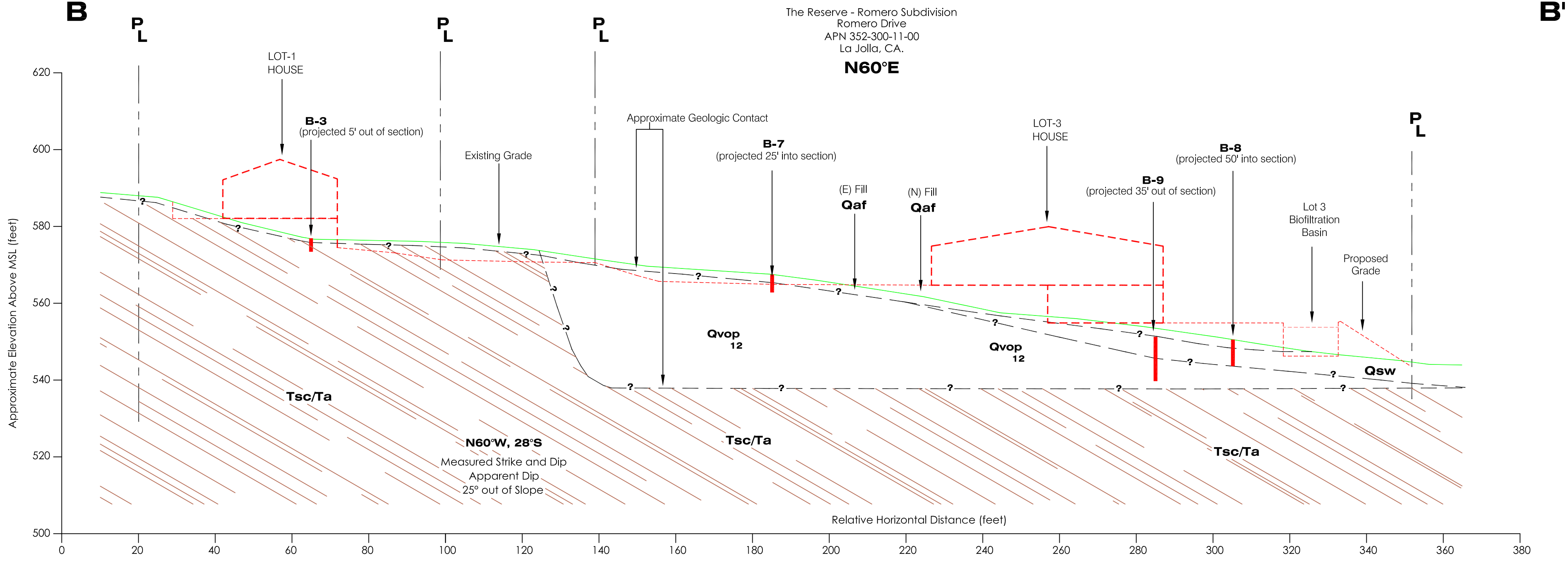
October 2022

10-9977-AA-9-2022

GEOLOGIC CROSS SECTION B-B'

The Reserve - Romero Subdivision
Romero Drive
APN 352-300-11-00
La Jolla, CA.

N60°E



Scale: 1" = 20'
(Horizontal and Vertical)

(N) Qaf	New Artificial Fill	$Qvop_{12}$	Very Old Paralic Deposits, Unit 12	—?—?—	Approximate Geologic Contact
(E) Qaf	Existing Artificial Fill	Tsc	Scripps Formation	⬚	Proposed Grade and Structures
Qsw	Slopewash	Ta	Ardath Shale		

NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

Figure No.Vb
Job No. 10-9977

Geotechnical Exploration, Inc.

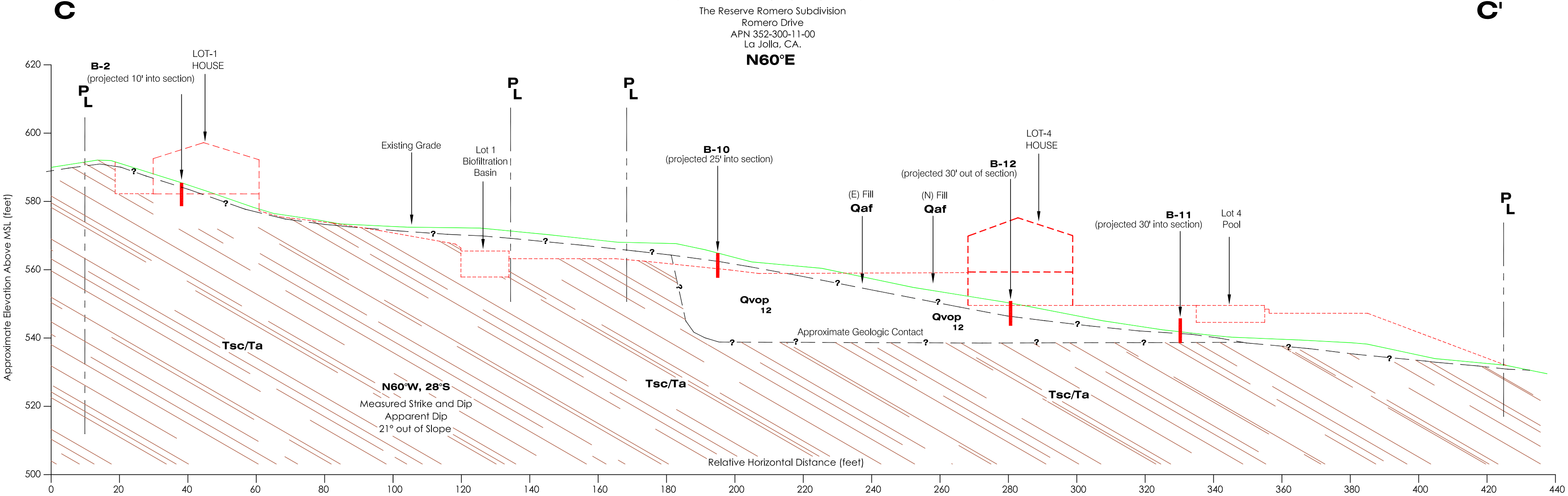
October 2022

10-9977-BB-9-2022

GEOLOGIC CROSS SECTION C-C'

The Reserve Romero Subdivision
Romero Drive
APN 352-300-11-00
La Jolla, CA.

N60°E



(N) Qaf	New Artificial Fill	**Qvop$_{12}$**	Very Old Paralic Deposits, Unit 12	–?–?–	Approximate Geologic Contact
(E) Qaf	Existing Artificial Fill	**Tsc**	Scripps Formation	▭	New Grade and Structures
Qsw	Slopewash	**Ta**	Ardath Shale		

Scale: 1" = 20'
(Horizontal and Vertical)

NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

Figure No.Vc
Job No. 10-9977

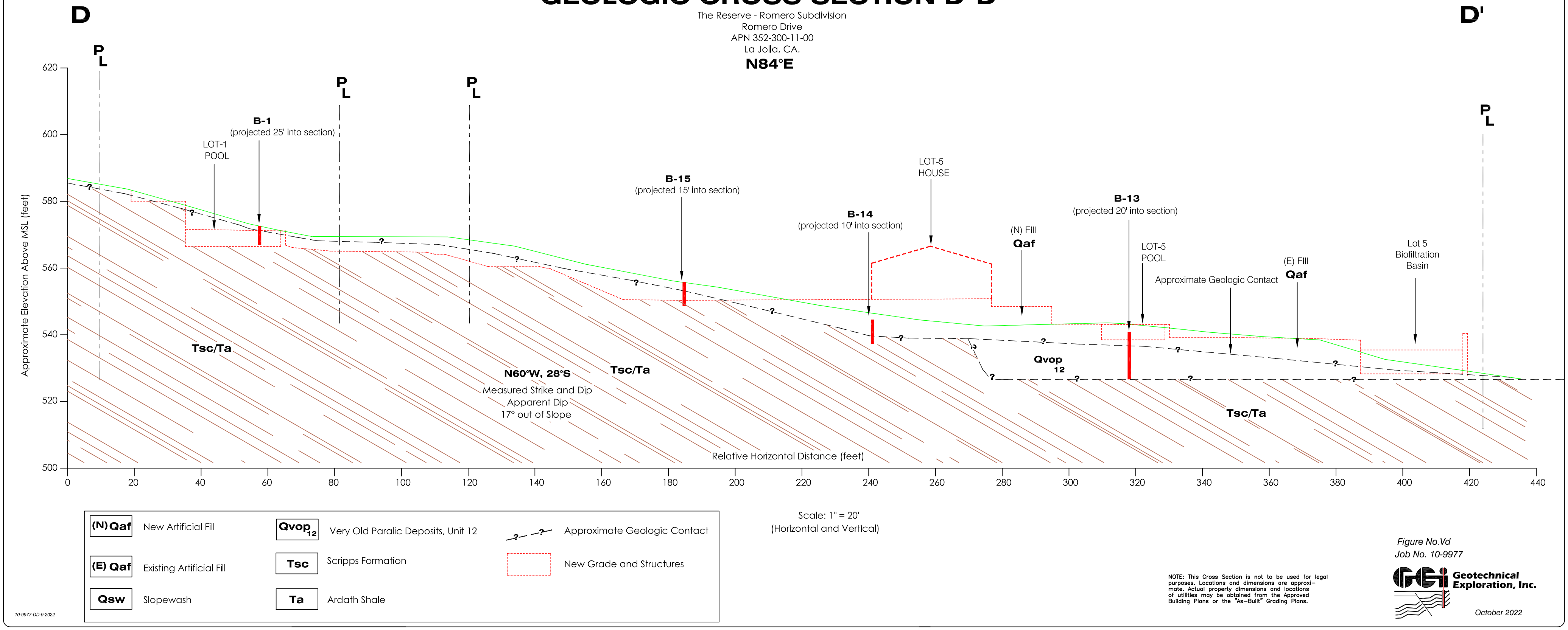
Geotechnical Exploration, Inc.

October 2022

10-9977-CC-9-2022

GEOLOGIC CROSS SECTION D-D'

The Reserve - Romero Subdivision
Romero Drive
APN 352-300-11-00
La Jolla, CA.
N84°E



Scale: 1" = 20'
(Horizontal and Vertical)

NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

Figure No.Vd
Job No. 10-9977

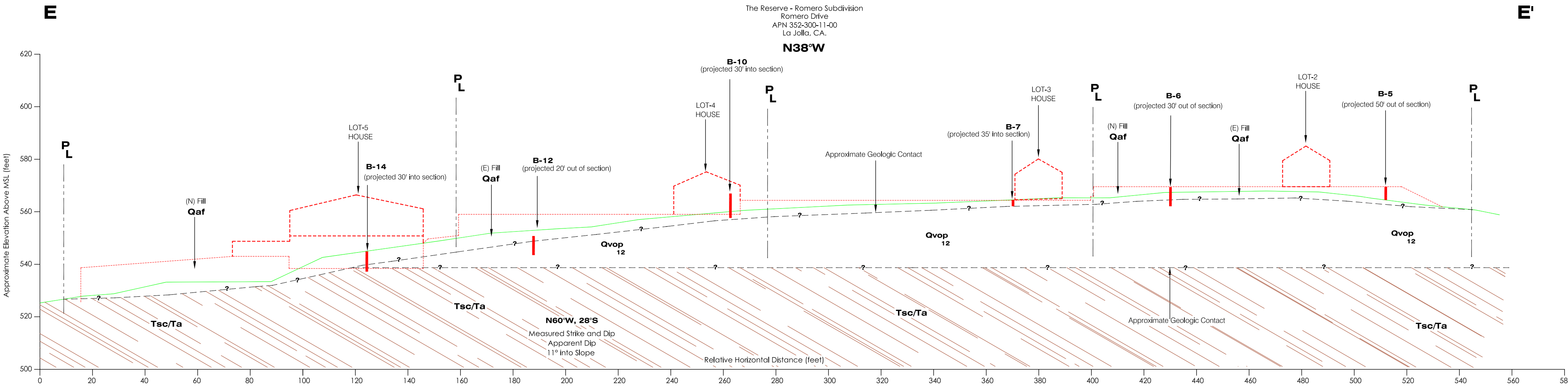
Geotechnical Exploration, Inc.

October 2022

GEOLOGIC CROSS SECTION E-E'

The Reserve - Romero Subdivision
Romero Drive
APN 352-300-11-00
La Jolla, CA.

N38°W



Scale: 1" = 20'
(Horizontal and Vertical)

(N) Qaf	New Artificial Fill	**$Qvop_{12}$**	Very Old Paralic Deposits, Unit 12	—?—?—	Approximate Geologic Contact
(E) Qaf	Existing Artificial Fill	**Tsc**	Scripps Formation	▭	New Grade and Structures
Qsw	Slopewash	**Ta**	Ardath Shale		

NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

10-9977-EE-9-2022

Figure No.Ve
Job No. 10-9977

Geotechnical Exploration, Inc.

October 2022

APPENDIX A

Update Report of Preliminary Geotechnical and Geologic Investigation -
October 23, 2016

UPDATE REPORT OF PRELIMINARY GEOTECHNICAL AND GEOLOGIC INVESTIGATION

The Reserve LLC Residential Project
Romero Drive
APN 352-300-07-00
La Jolla, California

JOB NO. 10-9977.1
23 October 2016

Prepared for:

Mr. Robert Aguilar
La Jolla Reserve, LLC

Geotechnical Exploration, Inc.

SOIL AND FOUNDATION ENGINEERING ● GROUNDWATER ● ENGINEERING GEOLOGY

23 October 2016

La Jolla Reserve, LLC **Job No. 10-9977.1**
c/o Manchester Financial Group
101 Ash Street, Suite 1900
San Diego CA 92101
Attn: Mr. Robert Aguilar

Subject: **Update Report of Preliminary Geotechnical and Geologic Investigation**
The Reserve LLC Residential Project
Romero Drive
APN 352-300-07-00
La Jolla, California

Dear Mr. Aguilar:

In accordance with your request, ***Geotechnical Exploration, Inc.*** has prepared this update report of geotechnical and geologic investigation at the subject property in La Jolla. Our original fieldwork was performed between August 11 and September 22, 2011.

In our opinion, if the conclusions and recommendations presented in this report are implemented during site preparation, the site will be suited for the proposed residential project.

This opportunity to be of service is sincerely appreciated. Should you have any questions concerning the following report, please do not hesitate to contact us. Reference to our **Job No. 10-9977.1** will expedite a response to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

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UPDATED REPORT OF PRELIMINARY GEOTECHNICAL AND GEOLOGIC INVESTIGATION

The Reserve LLC Residential Project
Romero Drive
La Jolla, California

Job No. 10-9977.1

I. PROJECT DESCRIPTION

It is our understanding, based on discussions with the client, Mr. Robert Aguilar of The Reserve LLC, and Mr. Kent Coston of Coston Architects Inc., that it is planned to construct a new one-story, single-family residence on the northern portion of Parcel 3 of The Reserve LLC property. We understand the proposed approximately 2,500-square-foot construction will be of conventional materials. The site has never been developed and is currently vacant. As such, a geotechnical investigation was performed.

The City of San Diego geologic hazard map for the area shows a fault crossing the northeast corner of the property described as a "*…potentially Active Fault. Inactive, presumed inactive or activity unknown,"* identified as Geologic Hazard Category (GHC) Zone 12. The northern half of the property is also mapped within a zone underlain by a *"slide-prone formation."* As such, to address these geologic concerns, a geologic investigation was performed in addition to the geotechnical investigation.

We performed a geologic and geotechnical investigation of the entire Reserve property in 2011. In preparation of our original report and during the field phase of exploration, which required temporary access road construction, we utilized a topographic survey map and grading plan prepared by the project Civil Engineering consultant, The Paul Design Group, City of San Diego approval dated July 13, 2011.

We note that the original field investigation was performed in 2011 with a grading permit issued by the City of San Diego to build temporary access roads. In addition, the grading and field exploration of the site, classified as Environmentally Sensitive Land (ESL) by the City of San Diego, required full time monitoring by biological, paleontological and archaeological consultants with Native American monitors.

In preparation of this update report, we utilized plans prepared by Coston Architects Incorporated entitled *"The Reserve: Single Family Residence; Romero Drive; La Jolla, CA 92037"* dated July 26, 2016. The project and associated improvements will be located within Parcel 3 of The Reserve property within Subarea C and portions of Subareas A and B (as shown on the referenced 2016 plans).

The objectives of the geotechnical and geologic investigation were as follows:

1. To evaluate the geotechnical and geologic aspects of the site with regard to the feasibility of the proposed residential project.

2. To evaluate the existing subsurface soil conditions at the site.

3. To evaluate representative samples of the soils for their engineering properties.

4. To address the general geology at the site, including an evaluation of the mapped and encountered (if any) geologic hazards.

5. To provide conclusions and recommendations pertinent to site preparation, mitigation of encountered geologic hazards (if any) and any required grading operations.

6. To provide preliminary foundation and design criteria suitable to the proposed residences.

These objectives are the same with respect to the current 2016 residential project.

II. SCOPE OF WORK

With the above in mind, the Scope of Work that was performed is outlined as follows:

1. ***Geotechnical Exploration Inc***. served as the designated Construction Manager (CM) during the implementation of the 2011 grading required at the site to allow access for exploratory drilling equipment. In addition environmental concerns as defined on the approved 2011 grading plans required implementation of biological, archaeological and Native American monitoring programs during the site work.

2. Three temporary access roads were constructed in 2011 using conventional grading equipment. These work areas required manual installation of *"limits-of-work fencing"* and silt fencing (BMPs) prior to and after grading; removal of existing vegetation; segregation of removed vegetation (brush); export of non-native brush; mulching of native vegetation removed from the road areas; stripping, stockpiling and segregation of individual road topsoils; re-contour grading of the roads following completion of exploration; replacement of the topsoils on the individual roads, etc.

3. In addition to the 2011 temporary access roads, seven other areas were delineated by "*limits-of-work*" environmental rope fencing where it was

planned to access proposed exploratory trenches with an all-terrain backhoe for exploration.

4. Review of available geologic reports and maps pertinent to the site and the general vicinity.

5. Six large-diameter exploratory borings were advanced in 2011 across the entire 25.25-acre Reserve property. With respect to the current project area within Parcel 3, one of the large-diameter borings, B-3, is applicable to characterization of the subsurface conditions below the currently planned project. The large-diameter boring, placed at the end of a temporary access road was downhole-logged by our Principal Certified Engineering Geologist.

   Both bulk and chunk samples of the encountered natural ground/formational materials were retrieved from the boring for laboratory testing for geotechnical/soil physical parameters with respect to required foundation/bearing soil evaluations, hillside stability analyses, soil strength, classification, etc.

6. Ten exploratory trenches were advanced in 2011 in selected areas to explore shallow fill and native soil conditions across the entire Reserve property in areas being considered for development. Two of these trenches were placed in and near the current project area, trenches T-4 and T-7. One trench, T-6, was extended across the mapped "*potentially active fault*" zone in order to evaluate the subsurface for the presence of the conjectured fault hazard. Both bulk and chunk samples of the encountered natural ground/formational materials were retrieved from the trenches for subsequent soil laboratory testing.

7. Laboratory testing on selected soil samples to aid in assessing their classification per applicable portions of the Unified Soil Classification System (see Appendix A), as well as their field moisture content and density and other soil physical parameters. Selected test results applicable to the currently planned project have been utilized here.

8. Geotechnical engineering analysis of the results of our research, and field and laboratory soil testing with respect to the currently planned project.

9. Slope stability analyses of cross sections drawn through the entire Reserve property utilizing the soil strength laboratory data, as well as proprietary information concerning soil strength properties of the formational materials as encountered on other nearby properties in La Jolla. One of these cross sections, Cross Section B-B', and our analyses include the current project area.

10. Preparation of this update report for the current project per City of San Diego guidelines including the pertinent results of field and laboratory soil testing, along with the updated findings from our geologic investigation and conclusions and recommendations (with the pertinent cross section, pertinent excavation logs and other graphics). This report also addresses the seismic risk potential of the site with respect to local and regional faulting per the current California Building Code. Our report includes:

    - A geologic map of the property prepared from the measurements made on our site geologic traverse and measurements taken on encountered sedimentary bedding (layers) and fractures within the large-diameter borings and trenches and on outcrops.
    - The pertinent cross section prepared through the property that includes the current project using the referenced topographic survey and our measurements.

- The pertinent results of our soil laboratory testing.
- The pertinent results of our research from available geologic reports and maps.
- Opinions regarding the mapped and encountered geologic hazards at the property with respect to the current project.
- Preliminary conclusions and geotechnical recommendations for development of the planned current clubhouse project.

III. EXECUTIVE SUMMARY

In summary, our subsurface investigation revealed that the entire property is underlain by very competent, high-strength formational materials of the Tertiary Ardath Shale, undifferentiated Tertiary Scripps/Ardath Shale Formations and the Quaternary Lindavista Formation, currently referred to as Quaternary Very Old Paralic Deposits (Qvop). The Quaternary Very Old Paralic Deposits (Qvop) underlie the current 2016 project area. The formational units are covered in the most part with a shallow thickness of sandy slopewash soils, topsoils and locally varying thicknesses of fill soils. Portions of the site have historically been used as unpaved roadways going back at least 8 decades. Other areas of the Reserve property have been filled, notably upper canyon and canyon margin areas on the northeast portion of the property east of the current project area.

A potentially active fault does not exist on the site. Trenching excavation across the mapped fault zone revealed no offset in uniformly dipping interbeds of claystones and sandstone of the Scripps Formation. Nearby surficial outcrops also do not display significant faulting offset of the layered formational materials and reveal generally consistent attitudes between boring, trench and outcrop exposures.

The undifferentiated Tertiary Scripps/Ardath Shale Formation bedding is parallel to or dips out of slope across the northern portion of the site with measured attitudes of up to 32 degrees out of slope to the south and southwest. The current 2016 project area is not planned for this area and will not be constructed over significant thicknesses of these materials.

Shallow surficial slopewash and topsoil materials and the existing old fill soils are not currently suitable for support of the planned improvements. The slopewash and fill will have to be removed and recompacted if required to achieve planned design grades. The clay topsoils are to be removed from planned project construction areas and relocated to non-construction areas or be exported from the site. Old fill soils adjacent to a canyon and an existing unpaved road on the northeastern portion of the property, northeast of the current project but affecting site access, will have to be dressed to improve their erosion resistance, if planned to be left in place.

Portions of the encountered undifferentiated Scripps/Ardath Formations materials consist of hard/dense silt and clay, silty sand (sandstone) and clayey silt with minor amounts of gravel. They are weathered in their upper portion beneath the surficial fill and slopewash, consisting of slightly fractured clayey sand. The contact between the units is characterized as intertounging. These deposits were explored to practical depths of 86 feet below the ground surface and are assumed to be over 150 feet thick. They are underlain conformably by the Mount Soledad Formation (not exposed at the site) comprised of conglomerate and sandstone. The currently planned 2016 project will not be founded on these materials.

Measurements of the bedding attitudes within the large-diameter borings through the Quaternary Very Old Paralic Deposits/Lindavista Formation, undifferentiated Ardath/Scripps Formations and Ardath Shale in our exploratory trenches and on our geologic traverse indicate that the sedimentary layering is part of a broad syncline or monocline with steeper southward dips on the northern portion of the property, becoming shallower and horizontal on the central and southern portion of the property. No significant fracturing indicative of landsliding or faulting was observed within the borings, trenches or in outcrop. No remolded clay gouge or bedding seams characteristic of bedding plane (parallel) landslide slip surfaces were observed within the borings, trenches or on outcrop.

Slope stability evaluations indicate the hillsides across the property, including the current 2016 project, have a factor of safety against deep-seated failure of 1.5 or greater and are suitable for development as a residential project per guidelines of the City of San Diego.

We have also provided herein recommendations for preparation of the site for the currently planned new conventional residential improvements as well as preliminary foundation and other soil design recommendations. All excavations should be monitored for newly exposed geologic conditions during the construction phase. Further, as project planning proceeds and the actual locations of house pads, roads and other improvements are determined additional shallow exploration may be required to confirm local soil conditions. Additional recommendations may be issued.

IV. SITE DESCRIPTION AND BACKGROUND

The entire 25.25-acre undeveloped Reserve property is known as Assessor's Parcel No. 352-300-04-00, a portion of Pueblo Lot 1263, according to Miscellaneous Map No. MM36, in the La Jolla area of the City and County of San Diego, State of California. The site is an irregularly shaped property that wraps around the southeast side of a ridgeline extending to the southwest from the southwestern flank of Mount Soledad.

It is located northeast of the southern terminus of Country Club Drive, south of the cul-de-sacs of Romero Drive and Encelia Drive and is bounded to the east and southeast by residential tract properties along Via Valverde in the La Jolla area of the City of San Diego. The property is bordered on the west by Foxhill Estate, a residential property. The current project is located within Parcel 3 of The Reserve property. Refer to the Vicinity Map, Figure No. Ia, for the location of the property.

The property is accessed from the southern end of Romero Drive. An unpaved driveway trends south toward the planned location of the current project, though no roadway currently extends from this unpaved road to access the project site. There are no habitable improvements on the property. The property in general is undeveloped and vacant. Figure No. Ib, an aerial photograph of the property from 1927, is included here.

In the approximate area of the currently planned project the property consists of a relatively uniform, moderately sloping, southerly descending hillside with elevations ranging from approximately 550 feet above Mean Sea Level (MSL) to approximately 565 feet above MSL.

As part of our Scope of Services for our 2011/2012 project, we researched available map and aerial photograph records of the site. We have reviewed the USGS La Jolla, California 7.5-minute quadrangle maps dated 1975, City of San Diego topographic and orthophotographic maps of the area including the site, Lambert coordinates 246-1689 dated 1953, 1963 and 1979; 2011 Google Earth imagery, a 1927 historic aerial site photo and USDA stereo-pair, high-angle photographs of the site (AXN-8M-90 and 91) taken in 1953. Refer to the Site Plan, Figure No. IIa for the currently planned project and the Site Plan and Geologic Map, Figure No. IIb, for the general configuration of the entire Reserve property. The location of the currently planned project is depicted on this plan.

V. FIELD INVESTIGATION

A. Exploratory Excavations

Exploration of the entire Reserve property included 6 large-diameter borings, 10 backhoe trenches and mapping of formational outcrop. Three large-diameter borings were advanced for purposes of geologic evaluation. Three shallower borings were advanced after geologic exploration to further define the depths and lateral extent of old fill soils encountered earlier in our investigation. Large diameter borings B-2 and B-3 were placed just south and just north of the currently planned project. The borings were advanced to depths of 80 and 86 feet, respectively.

The exploratory trenches were placed primarily in order to obtain representative soil samples and to define local soil and geologic profiles across the property. One trench, T-6, was advanced across a mapped City of San Diego potentially active fault zone, north of the planned project location, to confirm the presence or lack of faulting. Trench T-4 was advanced just south of the planned project and trench T-7 was advanced just north. Both trenches were advanced to depths of approximately 6 feet. Trench T-6 was advanced to a maximum depth of approximately 10 feet. Refer to the Site Plan and Geologic Map, Figure No. IIb, for the locations of the exploratory excavations.

The soils and geologic conditions encountered in the shallow excavations were logged by our field representatives and samples were taken of the predominant soils throughout the field operation. Our Principal Certified Engineering Geologist downhole logged geologic conditions encountered in boring B-3, and the referenced exploratory trenches. Exploratory logs were prepared on the basis of our observations and laboratory testing. The logs for the referenced borings and trenches pertinent to the current project have been included here. The results have been summarized on Figure Nos. IIIa-l. The predominant soils have been visually classified in general conformance with applicable portions of the Unified Soil Classification System (refer to Appendix A).

VI. FIELD AND LABORATORY TESTS & SOIL INFORMATION

A. Field Tests

The trenches were logged by our representatives. A pointed steel bar and other tools were used to qualitatively assess the penetration resistance and in situ density of the encountered soil types. Trench soil samples were also examined under hand lens and moistened with a spray bottle. Bulk (disturbed) samples of the soils were retrieved for subsequent laboratory testing from the trenches and borings. Relatively undisturbed chunk samples of native ground soils were also retrieved from the excavations for laboratory density testing.

B. Laboratory Tests

Laboratory tests were performed on disturbed and relatively undisturbed soil samples in order to evaluate their physical and mechanical properties and their ability to support the proposed residential and commercial improvements. Test results are presented on Figure Nos. III and IV. The following tests were conducted on the sampled soils pertinent to the current project:

1. *Determination of Percentage of Particles Smaller than #200 Sieve (ASTM D1140-06)*
2. *Moisture Content (ASTM D2216-05)*
3. *Expansion Index (ASTM D4829-07)*
4. *Density Measurements (ASTM D1188-07)*
5. *Atterberg Limits (D4318-05)*
6. *Direct Shear Test (ASTM D3080-04)*

The *Determination of Percentage of Particles Smaller than -200 Sieve* test (ASTM D1140-06) aids in classification of the tested soils based on their fine material content and provides qualitative information related to engineering characteristics such as expansion potential, permeability, and shear strength.

The *Moisture Content* of a soil sample is a measure of the water content, expressed as a percentage of the dry weight of the sample (ASTM D2216-05).

The expansion potential of soils is determined, when necessary, utilizing the *Standard Test Method for Expansion Index of Soils* (ASTM D4829-07). In accordance with the Standard (Table 5.3), potentially expansive soils are classified as follows:

EXPANSION INDEX	***EXPANSION POTENTIAL***
0 to 20	Very low
21 to 50	Low
51 to 90	Medium
91 to 130	High
Above 130	Very high

Based on our particle-size test results, our visual classification, EI test results (Expansion Indices of 8 and 70 on the encountered shallow Quaternary Very Old Deposits/Lindavista Formation materials and undifferentiated Scripps/Ardath Formation materials, respectively) our experience with similar soils, it is our opinion that the shallow tested materials in and below the currently planned project area have a very low to medium expansion potential.

Density measurements on selected samples of the retrieved formational materials were performed using the *Bulk Specific Gravity Utilizing Paraffin-Coated Specimens* method (ASTM D1188-07). This helps to establish the unit weight of the formational exposures/outcrops.

The *Atterberg Limits* (ASTM D4318-05) are a basic measure of the nature of a fine-grained soil. Depending on the water content of the soil, it may appear in four states: solid, semi-solid, plastic and liquid. In each state the consistency and behavior of a soil is different and thus so are its engineering properties. Therefore, the boundary between each state can be defined based on a change in the soil's behavior. The test method measures the *Liquid Limit* (LL), the boundary between the liquid and the plastic states; the *Plastic Limit* (PL), the boundary between the plastic and the semi-solid states; and the *Shrinkage Limit* (SL), the boundary between the semi-solid and the solid states. Correlations of Atterberg Limits and percentage of fine soil content have been used to assign shear strength values to the encountered soils.

The *Direct Shear Tests* (ASTM D3080-04) were performed on relatively undisturbed soil samples in order to evaluate their strength characteristics. The shear tests were performed with a constant strain rate direct shear machine. The specimens tested were saturated and then sheared under various normal loads under drained conditions.

Based on the laboratory test data, our observations of the primary soil types, and our previous experience with laboratory testing of similar soils, our Geotechnical Engineer has assigned values for the angle of internal friction and cohesion to those soils that will provide significant lateral support or load bearing on the project. These values have been utilized in assigning the recommended bearing value as well as active and passive earth pressure design criteria for foundations and retaining walls.

VII. REGIONAL GEOLOGIC DESCRIPTION

San Diego County has been divided into three major geomorphic provinces: the Coastal Plain, the Peninsular Ranges and the Salton Trough. The Coastal Plain exists west of the Peninsular Ranges. The Salton Trough is east of the Peninsular Ranges. These divisions are the result of the basic geologic distinctions between the areas. Mesozoic metavolcanic, metasedimetary and plutonic rocks predominate in the Peninsular Ranges with primarily Cenozoic sedimentary rocks to the west and east of this central mountain range (Demere, 1997).

In the Coastal Plain region, where the subject property is located, the "basement" consists of Mesozoic crystalline rocks. Basement rocks are also exposed as high relief areas (e.g., Black Mountain northeast of the subject property and Cowles Mountain near the San Carlos area of San Diego). Younger Cretaceous and Tertiary sediments lap up against these older features. The Cretaceous sediments form the local basement rocks on the Point Loma area. These sediments form a "layer cake" sequence of marine and non-marine sedimentary rock units, with some formations up to 140 million years old. Faulting related to the La Nacion and Rose Canyon Fault zones has broken up this sequence into a number of distinct fault blocks in the southwestern part of the county. Northwestern portions of the county are relatively undeformed by faulting (Demere, 1997).

The Peninsular Ranges form the granitic spine of San Diego County. These rocks are primarily plutonic, forming at depth beneath the earth's crust 140 to 90 million years ago as the result of the subduction of an oceanic crustal plate beneath the North American continent. These rocks formed the much larger southern California batholith. Metamorphism associated with the intrusion of these great granitic masses affected the much older sediments that existed near the surface over that period of time. These metasedimentary rocks remain as roof pendants of marble, schist, slate, quartzite and gneiss throughout the Peninsular Ranges. Locally, Miocene-age volcanic rocks and flows have also accumulated within these mountains (e.g., Jacumba Valley). Regional tectonic forces and erosion over time have uplifted and unroofed these granitic rocks to expose them at the surface (Demere, 1997).

The Salton Trough is the northerly extension of the Gulf of California. This zone is undergoing active deformation related to faulting along the Elsinore and San Jacinto Fault Zones, which are part of the major regional tectonic feature in the southwestern portion of California, the San Andreas Fault Zone. Translational movement along these fault zones has resulted in crustal rifting and subsidence. The Salton Trough, also referred to as the Colorado Desert, has been filled with sediments to depth of approximately 5 miles since the movement began in the early Miocene, 24 million years ago. The source of these sediments has been the local mountains as well as the ancestral and modern Colorado River (Demere, 1997).

As indicated previously, the San Diego area is part of a seismically active region of California. It is on the eastern boundary of the Southern California Continental Borderland, part of the Peninsular Ranges Geomorphic Province. This region is part of a broad tectonic boundary between the North American and Pacific Plates. The actual plate boundary is characterized by a complex system of active, major, right-lateral strike-slip faults, trending northwest/southeast. This fault system extends eastward to the San Andreas Fault (approximately 70 miles from San Diego) and westward to the San Clemente Fault (approximately 50 miles off-shore from San Diego) (Berger and Schug, 1991).

In California, major earthquakes can generally be correlated with movement on active faults. As defined by the California Division of Mines and Geology (Hart, E.W., 1980), an "*active*" fault is one that has had ground surface displacement within Holocene time (about the last 11,000 years). Additionally, faults along which major historic earthquakes have occurred (about the last 210 years in California) are also considered to be active (Association of Engineering Geologist, 1973). The California Division of Mines and Geology defines a "potentially active" fault as one that has had ground surface displacement during Quaternary time, that is, between 11,000 and 1.6 million years (Hart, E.W., 1980).

VIII. SITE-SPECIFIC PROJECT GEOLOGY

Excerpts from Regional Geologic Maps (with legends) including the site are included herein as Figures Nos. Va-b. Figure No Va is an excerpt from a geologic map prepared by Michael Kennedy, *Geology of the La Jolla Quadrangle (1975)* included within Bulletin 200 of the California Division of Mines and Geology (now the California Geologic Survey), *Geology of the San Diego Metropolitan Area, California.* This map indicates the site is underlain by the Tertiary Ardath Shale (Ta). This is inaccurate.

Figure No. Vb is an excerpt from the California Geologic Survey and United States Geological Survey *Geologic Map of the San Diego 30’x60’ Quadrangle, California* by Michael P. Kennedy and Siang S. Tan (2008). On this 2008 map, the native ground materials underlying the project portion of the site are also shown to be the Ardath Shale Formation (Ta).

We note that our attached Figure No. IIb, the Site Plan and Geologic Map, differs from the referenced 1975 and 2008 regional geologic maps. The referenced geologic maps indicate that the Tertiary Ardath Shale extends to the ground surface and underlies topsoils over the entire site, including the currently planned project area. However, our exploration revealed that Quaternary Very Old Paralic Deposits (Qvop), referred to as the Lindavista Formation (Qln) in our 2011 report, underlie a significant portion of the site including the currently planned (2016) project. Recent exploration on the adjacent Foxhill Estate property to the west indicates that these same materials also underlie that property. A geologic cross section that includes the currently planned project, Cross Section B-B’ (Figure No. VI), has been included here and illustrates the subsurface conditions underlying the currently planned project.

Based on our exploratory drilling with downhole geologic observations, our shallow exploratory trench excavations, our geologic traverse and our review of site photos and geologic maps, we consider the geologic conditions below and in the immediate vicinity of the property, including the currently planned project, to be relatively well defined. Deep foundation systems such as caissons and grade-beams should not be required for structures or exterior improvements due to geologic hazards. Based on our findings, it is our opinion that landslide stabilization or landslide mitigation procedures are not required.

A. ***Stratigraphy***

Quaternary Artificial Fill (Qaf): Artificial fill soils were encountered on the site. These are believed to be up to 8 decades old or more. Artificial fill up to 1½ feet thick was encountered in trench T-4, and was originally placed on an unpaved road constructed decades ago. The encountered fill soils appear to be associated with this roadway. They consist of clayey sand and silty sand with varying amounts of gravel, cobble and some debris (e.g., concrete, wood). No significant thicknesses of artificial fill are believed to exist in the area of the currently planned project, which appears to be undisturbed land. Refer to Figure Nos. II, III and VI for details.

Quaternary Slopewash (Qsw): A veneer of slopewash covers most of the site, especially the southern and central portions where the Qvop materials/Lindavista Formation exists. This slopewash unit consists of silty sand and ranges from 2 to 3 feet thick. It was encountered in boring B-3 and trench T-4, nearest the current project site. Where encountered on or near the surface it is in a dry and loose condition. This material is not suitable for support of structures or other improvements in situ. It is suitable for use as fill material if properly removed and recompacted. It is of very low expansivity. Refer to Figure Nos. III and VI for details.

Quaternary Very Old Paralic Deposits (Qvop)/Lindavista Formation (Qln): Quaternary Very Old Paralic Deposits (Qvop), referred to as the Quaternary Lindavista Formation (Qln) in our 2011 report, overlie the Tertiary Ardath Shale and Tertiary undifferentiated Scripps/Ardath Shale Formations. This contact is unconformable and is distinctive due to prominent basal lag gravel observed across the site that is part of the Qvop materials. The lag gravel rests on the underlying Tertiary units. This unit was encountered in all exploratory borings and in trenches T-1, T-2, T-3, T-4, and T-5. This unit underlies the currently planned 2016 project location.

The encountered materials consist of silty sand, clayey sand and sandy clay interbeds. They are in a dense to very dense/stiff condition. They are generally massive and sub-horizontal to horizontal. Clayey sand portions of the unit have a very low Expansion Index. The unit appears to dip up to 6 to 8 degrees to the west where encountered on the northern portion of the entire Reserve property. We note that the Quaternary Very Old Paralic Deposits (Qvop)/Lindavista Formation (Qln) is not mapped in this area on various publically available geologic references though other geotechnical investigators (e.g., Hart 2010) have recognized this unit in the area in proprietary reports. Refer to Figure Nos. II, III and VI for details.

Undifferentiated Tertiary Scripps/Ardath Shale Formations (Tsc/Ta): The Ardath Shale and Scripps Formation are believed to be intertongued on the northern portion of the site and are characterized as "*undifferentiated*." The basis for this distinction is the sandier nature of the sedimentary layers encountered in our boring B-3 and trenches T-6, T-7, T-8, T-9 and T-10 and our experience with encountered sedimentary structures within the Scripps Formation. We note that the Scripps and Ardath Shale Formations are known to be intergradational (Kennedy, 1975).

The encountered undifferentiated Scripps/Ardath Shale Formations materials consist of firm to hard silty clay (mudstone), clay (shale) and sandy silt (siltstone), and dense silty sand. Clay portions of the unit have a medium Expansion Index. These deposits were explored to practical depths of 86 feet below the ground surface. They were also explored in the referenced trenches and they are exposed in outcrops on the northern portion of the site.

These materials unconformably underlie the Quaternary Very Old Paralic Deposits (Qvop). It is unlikely they will be encountered on the currently planned project site where they are believed to exist at depths of 10 feet or greater below the ground surface. Refer to Figure Nos. II and III for details.

B. Geologic Structure

Bedding: Bedding is generally massive and subhorizontal to horizontal within the encountered Quaternary Very Old Paralic Deposits (Qvop)/Quaternary Lindavista

Formation (Qln). The basal lag gravel was measured to dip approximately 6 to 8 degrees to the west on the northern portion of the property. The westerly dip of the contact and lag gravel is believed to be due to the uplift of Mount Soledad. Refer to cross section B-B′, included here as Figure No. VI. This section has also been utilized to assess slope stability for the current project. Refer to the Geologic Hazards Section VII of this report for more details. In summary, the bedding attitudes as observed across most of the site, including the location of the currently planned project, as measured in borings and trenches applicable to the current project, are considered neutral to favorable.

Faults: As shown on City of San Diego Geologic Hazards Map Sheet No. 29, a Zone 12 fault *("potentially active fault")* is mapped crossing the northeastern portion of the property. The location of the fault is inferred and is dashed. A *"buffer zone"* 200 feet wide, 100 feet to either side of the inferred fault trace, is mapped parallel to the inferred fault trace. Because the fault trace is inferred the buffer zone is included as suggested area to be explored for the presence of faulting. This area was explored by trenching for faulting and no faults were discovered. Refer to the Geologic Hazards Section VII of this report.

The mapped Zone 12 fault is referred to as the Country Club Fault on various geologic references, including Kennedy (1975), Kennedy, Tan, Chapman and Chase (1975) and Treiman (1993). It is mapped crossing the subject property on these references, though it is dashed (approximately located) on Kennedy (1975) as on the City of San Diego's Geologic hazards map Sheet 29. It has been described as a reverse fault and a dip slip fault. Where well exposed in outcrops along Romero Drive it juxtaposes the Cretaceous Cabrillo Formation (Kc) and the Tertiary Mount Soledad Formation (Tms). At other localities on the southeast flank of Mount Soledad it juxtaposes the Quaternary Very Old Paralic Deposits (Qvop)/Lindavista Formation (Qln) and the

Tertiary Ardath Shale formation. The Quaternary Old Paralic Deposits (Qop), formerly described as the Bay Point Formation, overlie the fault without offset in this area.

As part of our investigation we placed an exploratory trench, T-6, across the Zone 12 shown on the City of San Diego's geologic hazard map. Refer to Figure No. IIIg. An additional exploratory trench, T-10, was placed within the Zone 12 area to the south. See Figure No. IIIh. In addition to the subsurface trenching, we were also able to directly observe outcrop southeast of our trench within the Zone 12 feature and outcrop exposed within a drainage channel to the southwest of our trench within Zone 12. The trench exposures and outcrops display sedimentary beds of the undifferentiated Scripps/Ardath Shale Formations. These beds consist of alternating layers of sandy and silty clay and silty sand. They strike generally east-west or east northeast-west southwest. Dips range from 23 to 26 degrees. No offset of the encountered and observed beds was observed.

Landsliding: Landslides (and slope creep) are both gravity driven soil and earth-movement phenomena. Movement occurs, therefore, primarily in a directly downslope direction. A conjectured landslide is mapped on the southern portion of the entire Reserve property per City of San Diego Geologic Hazards Map sheet 29. It does not include the currently planned project area. This feature is referred to as Zone 22, a *"possible or conjectured"* landslide. This map feature was explored for landsliding by advancing two exploratory borings within the margins of the mapped Zone 22 feature. No landslides were encountered. The hillside areas of the property have not been significantly affected by these earth movement phenomena. Refer to the Geologic Hazards Section VII of this report for more detail.

Cross sections generated during our original 2011 investigation, including the cross section included here B-B′, have been utilized, along with our laboratory analyses, to evaluate slope stability. Factors of safety of 1.5 or higher exist across the hillsides on the entire Reserve property, including the currently planned 2016 project. Slope stability calculations pertinent to cross section B-B′ are included herein, refer to Appendix B.

IX. GEOLOGIC HAZARDS

The entire Reserve property is mapped within Geologic Hazard Categories (GHC) Zones 12, 22, 26 and 27 on Sheet 29 of the City of San Diego Geologic Hazard Zone maps. The currently planned 2016 project is located entirely within GHC Zone 26. GHC Zone 12 is also mapped crossing the northeasterly portion of the currently planned project area. Refer to an excerpted portion of this map and legend, included here as Figure No. VII.

Zone 12 is a mapped geologic fault referred to as the Country Club Fault and described as a "…*Potentially Active Fault. Inactive, presumed inactive or activity unknown.*" We note that the fault is mapped on the Geologic Hazard Sheet 29 as crossing the northeast corner of the property. We explored for the fault as part of our 2011 investigation. No evidence of faulting was encountered.

The northern half of the property, including the currently planned project, is mapped within Zone 26. Zone 26 includes areas of *"potential slope instability"* underlain by a *"slide-prone formation"* (e.g., the Tertiary-age Ardath Shale, Ta) and *"unfavorable geologic structure…"* Our investigation indicates that the Ardath Shale (Ta) does not exist at the ground surface across the entire site. Additionally, undifferentiated Ardath Shale/Scripps Formation (Ta/Tsc) exists on the northern portion of the property and Quaternary Very Old Paralic Deposits (Qvop)/Quaternary Lindavista Formation materials (Qln) also overlie the Tertiary deposits over a significant portion of the property, including the currently planned project area. The Quaternary Very Old Paralic Deposits (Qvop)/Lindavista Formation (Qln) overlying the Ardath Shale and undifferentiated Ardath Shale/Scripps Formation has favorable geologic structure.

A geologic investigation was conducted to evaluate the on-site geology and nature of the noted geologic hazards that might affect the site, including the currently planned 2016 project. Our investigation drew upon information gathered from published and unpublished geologic maps and reports, as well as results of our exploratory trenches, borings and geologic traverse.

A. <u>Seismicity</u>

In California, major earthquakes can generally be correlated with movement on active faults. As defined by the California Geological Survey (Bryant and Hart, 2007), an *"active"* fault is one that has had ground surface displacement within Holocene time (about the last 11,000 years). Additionally, faults along which major historical earthquakes have occurred (about the last 210 years in California) are also considered to be active (Association of Engineering Geologist, 1973). The California Geologic Survey defines a *"potentially active"* fault as one that has had ground surface displacement during Quaternary time, that is, between 11,000 and 1.6 million years (Bryant and Hart, 2007).

The San Diego area is part of a seismically active region of California. It is on the eastern boundary of the Southern California Continental Borderland, part of the Peninsular Ranges Geomorphic Province. This region is part of a broad tectonic boundary between the North American and Pacific Plates. The actual plate boundary is characterized by a complex system of active, major, right-lateral strike-slip faults, trending northwest/southeast. This fault system extends eastward to the San Andreas Fault (approximately 70 miles from San Diego) and westward to the San Clemente Fault (approximately 50 miles off-shore from San Diego) (Berger and Schug, 1991).

During recent history, prior to April 2010, the San Diego County area has been relatively quiet seismically. No fault ruptures or major earthquakes had been experienced in historic time within the greater San Diego area. Since earthquakes have been recorded by instruments (since the 1930s), the San Diego area has experienced scattered seismic events with Richter magnitudes (M) generally less than M4.0. During June 1985, a series of small earthquakes occurred beneath San Diego Bay, three of which were M4.0 to M4.2. In addition, the Oceanside earthquake of July 13, 1986, located approximately 26 miles offshore of the City of Oceanside, was a M5.3 (Hauksson and Jones, 1988).

On June 15, 2004, a M5.3 earthquake occurred approximately 45 miles southwest of downtown San Diego (26 miles west of Rosarito, Mexico). Although this earthquake was widely felt, no significant damage was reported. Another widely felt earthquake on a distant southern California fault was a M5.4 event that took place on July 29, 2008, west southwest of the Chino Hills area of Riverside County. Several earthquakes ranging from M5.0 to M6.0 occurred in northern Baja California, centered in the Gulf of California on August 3, 2009. These were felt in San Diego but no injuries or damage was reported. A M5.8 earthquake followed by a M4.9 aftershock occurred on December 30, 2009, centered about 20 miles south of the Mexican border city of Mexicali. These were also felt in San Diego, swaying high-rise buildings, but again no significant damage or injuries were reported.

On April 4, 2010, a large earthquake occurred in Baja California, Mexico. It was widely felt throughout the southwest including Phoenix, Arizona and San Diego in California. This M7.2 event, the Sierra El Mayor earthquake, occurred in northern Baja California approximately 40 miles south of the Mexico-USA border at shallow depth along the principal plate boundary between the North American and Pacific plates. According to the U. S. Geological Survey, this is an area with a high level of historical seismicity, and it has recently also been seismically active, though this is the largest event to strike in this area since 1892. The April 4, 2010, earthquake appears to have been larger than the M6.9 earthquake in 1940 or any of the early 20th century events (e.g., 1915 and 1934) in this region of northern Baja California. The event caused widespread damage to structures, closure of businesses, government offices and schools, power outages, displacement of people from their homes and injuries in the nearby major metropolitan areas of Mexicali in Mexico and Calexico in southern California. Estimates of the cost of the damage range to $100 million.

This event's aftershock zone extended significantly to the northwest, overlapping with the portion of the fault system that is thought to have ruptured in 1892. Some structures in the San Diego area experienced minor damage and there were some injuries. Ground motions for the April 4, 2010, main event, recorded at stations in San Diego and reported by the California Strong Motion Instrumentation Program (CSMIP), ranged up to 0.058g. Aftershocks from this event have continued along the trend northwest and southeast of the original event, including within San Diego County, closer to the San Diego metropolitan area. There have been hundreds of these earthquakes including events up to M5.7.

B. Local and Regional Faults

For the location of faults discussed herein refer to the Regional Fault Map (Figure No. VIIIa) and the Local Fault Map (Figure No. VIIIb).

Country Club Fault: The Country Club Fault is shown to be *"approximately located"* on the northeastern portion of the property on the *"Geology of the La Jolla Quadrangle"*, a map included within Bulletin 200 of the California Division of Mines and Geology (now the California Geological Survey) *Geology of the San Diego Metropolitan Area, California*. No other faults are shown to cross the site on this map prepared by Kennedy (1975). The Country Club Fault is part of the Rose Canyon Fault Zone, per Kennedy, Tan, Chapman and Chase (1975):

> The western side of the fault zone is formed by the Country Club Fault, which lies adjacent to the Pacific Beach syncline between La Jolla and Mission Bay. The Country Club Fault is exposed on north facing slopes of Mount Soledad from near Romero Drive to the sea cliffs at La Jolla Cove. Rocks of Eocene age are downdropped to the west and juxtaposed with Upper Cretaceous strata along this segment of the fault zone. The late Pleistocene Bay Point Formation appears to overlap the fault without offset at both La Jolla and Pacific Beach although sediments of the early Pleistocene Lindavista Formation are faulted….along the southern part of the Country Club fault, strata of the Lindavista Formation are juxtaposed with Eocene rocks. The dip-slip component of faulting at this locality is approximately 30 m, with the younger rocks downdropped along its western side.

The Country Club Fault is identified on City of San Diego Geologic Hazards Map Sheet 29 as a Zone 12 geologic hazard feature, a "…*Potentially Active Fault. Inactive, presumed inactive or activity unknown."* This zone is 200 feet wide with the fault *"approximately located"* (i.e., dashed) within the center of this zone. As part of our geologic investigation we advanced an exploratory trench, T-6, across most of the width of this zone from southwest to northeast, and explored formational outcrop exposures across the northeast portion of this zone to (slightly southeast of our exploratory trench). The combination of trenching exposure and outcrop exposure allowed us to explore the width of the mapped Zone 12 for faulting.

Layered formational soils of the undifferentiated Ardath Shale/Scripps Formation were encountered within our trench, T-6. These same materials comprise the outcrop exposures. The layered materials generally strike from east-west to northwest-southeast and dip 20 to 30 degrees to the south and southwest. No significant faulting was observed within our exploratory trench nor on the outcrop exposures. Based on these findings, it is our opinion the Country Club Fault does not exist on the property and does not affect the currently planned project.

Other proprietary/public record reports also document a nearby investigation of this fault. These include:

1. *"Geotechnical Investigation and Geologic Reconnaissance; Berno Marie Anderson Residence; 7231 Romero Drive; La Jolla, California"* by Geocon Inc., prepared for Signature Architecture & Planning dated June 22, 1999.

2. *"Report of Updated Geotechnical Investigation; Romero Drive Residential; 7231 Romero Drive; La Jolla, California"* prepared by Southern California Soil & Testing, Inc. for Mr. Bill McCulley dated March 27, 2003.

3. *"Response to City of San Diego Geotechnical Review Letter; Romero Drive Residential; 7231 Romero Drive; San Diego, California"* prepared by Southern California Soil & Testing, Inc. for Mr. Bill McCulley dated June 16, 2003.

4. *"Response to City of San Diego First Geotechnical Review of Documents, Grading Plans for 7231 Romero Drive, Lot 11, Block E of La Jolla Country Club Heights, Work Order 422528, Drawing No. 330J35, PTS No. 30251; San Diego, California"* prepared by Southern California Soil & Testing, Inc. for Mr. Ken Cornell dated October 6, 2004.

5. *"Response to City of San Diego Geotechnical Review Letter; Romero Drive Residential; 7231 Romero Drive; San Diego, California"* prepared by Southern California Soil & Testing, Inc. for Mr. Bill McCulley dated March 1, 2005.

6. *"Response to City of San Diego Geotechnical Review Letter; Romero Drive Residential; 7231 Romero Drive; San Diego, California;"* prepared by Southern California Soil & Testing, Inc. for Mr. Ken Cornell dated July 15, 2005.

Reference No. 1 includes descriptions of the Country Club Fault from outcrop exposures on a property approximately 0.1-mile north of the subject property on the east side of Romero Drive. The authors, Geocon, Inc., note that the measured fault trend at the exposure is north-south and not N50 W as indicated on geologic maps by Kennedy (1975) and Kennedy and Tan (1975). They further describe the fault as being parallel to Romero Drive at their location, dipping approximately 60 degrees to the east and characterized by an approximately 4-foot-thick zone of highly sheared and brecciated sandstone and siltstone. No similar features were encountered during our exploration on the Reserve site.

References No. 2 through 6 are reports of updated geotechnical investigation of the same property in Reference No. 1 by a different geotechnical investigator, Southern California Soil & Testing. These reports describe the fault exposure on the 7231 Romero Drive site as juxtaposing the Cretaceous Cabrillo Formation on the east side of the fault against the younger Tertiary Mount Soledad Formation west of the fault. This differs from referenced geologic maps that identify the Country Club Fault as juxtaposing the Cabrillo and Mount Soledad Formation on the east side of the fault against the Ardath Shale west of the fault. They note that referenced geologic maps show the Country Club Fault west of the 7231 Romero Drive site. Further, *"…The strike of the main rupture of the fault in the general vicinity has been mapped as N30 to 50W. The attitude of the fault on site was measured as N10W/90 and N10W/80E."* It is the opinion of the report authors that the fault on that site is not the Country Club Fault but is most likely associated with it, and that it has a significant, although undetermined, amount of throw.

Rose Canyon Fault Zone: Other faults within the Rose Canyon Fault Zone include the Mount Soledad Fault and the Rose Canyon Fault. These are located less than 1 mile northwest of the subject site. The Rose Canyon Fault Zone is mapped trending north-south from Oceanside to downtown San Diego, from where it appears to head southward into San Diego Bay, through Coronado and offshore. Refer to Figure Nos. VIIIa-b. The Rose Canyon Fault Zone is considered to be a complex zone of onshore and offshore, en echelon strike slip, oblique reverse, and oblique normal faults. The Rose Canyon Fault is considered to be capable of causing a M7.2 earthquake and considered microseismically active, although no significant recent earthquake is known to have occurred on the fault. Investigations in the Rose Canyon Fault Zone at the Police Administration and Technical Center in downtown San Diego, in San Diego Bay, at the SDG&E facility in Rose Canyon, and elsewhere, have encountered offsets in Holocene (geologically recent) sediments. These findings confirm Holocene displacement on the Rose Canyon Fault and this fault was upgraded to an *"active"* fault in November 1991 (California Geological Survey – Fault-Rupture Hazard Zones in California, Special Publication No. 42, Interim Revision 2007).

Coronado Bank Fault: The Coronado Bank Fault is located approximately 12 miles southwest of the site (see Figure Nos. VIIIa-b). Evidence for this fault is based upon geophysical data (acoustic profiles) and the general alignment of epicenters of recorded seismic activity (Greene, 1979). A M5.3 earthquake recorded July 13, 1986, is known to have been centered on the fault or within the Coronado Bank Fault Zone. Although this fault is considered active, due to the seismicity within the fault zone, it is significantly less active seismically than the Elsinore Fault (Hileman, 1973). It is postulated that the Coronado Bank Fault is capable of generating a M7.6 earthquake and is of great interest due to its close proximity to the greater San Diego metropolitan area.

Elsinore Fault: The Elsinore Fault is located approximately 38 to 56 miles east and northeast of the site (see Figure Nos. VIIIa-b). The Elsinore Fault extends approximately 200 km (125 miles) from the Mexican border to the northern end of the Santa Ana Mountains. The Elsinore Fault zone is a 1- to 4-mile-wide, northwest-southeast-trending zone of discontinuous and en echelon faults extending through portions of Orange, Riverside, San Diego, and Imperial Counties. Individual faults within the Elsinore Fault Zone range from less than 1 mile to 16 miles in length. The trend, length and geomorphic expression of the Elsinore Fault Zone identified it as being a part of the highly active San Andreas Fault system.

Like the other faults in the San Andreas system, the Elsinore Fault is a transverse fault showing predominantly right-lateral movement. According to Hart, et al. (1979), this movement averages less than 1 centimeter per year. Along most of its length, the Elsinore Fault Zone is marked by a bold topographic expression consisting of linearly aligned ridges, swales and hallows. Faulted Holocene alluvial deposits (believed to be less than 11,000 years old) found along several segments of the fault zone suggest that at least part of the zone is currently active.

Although the Elsinore Fault Zone belongs to the San Andreas set of active, northwest-trending, right-slip faults in the southern California area (Crowell, 1962), it has not been the site of a major earthquake in historic time, other than a M6.0 earthquake near the town of Elsinore in 1910 (Richter, 1958; Toppozada and Parke, 1982). However, based on length and evidence of Late-Pleistocene or Holocene displacement, Greensfelder (1974) has estimated that the Elsinore Fault Zone is reasonably capable of generating an earthquake as large as M7.5. Recent study and logging of exposures in trenches in Glen Ivy Marsh across the Glen Ivy North Fault (a strand of the Elsinore Fault Zone between Corona and Lake Elsinore), suggest a maximum earthquake recurrence interval of 300 years, and when combined with previous estimates of the long-term horizontal slip rate of 0.8 to 7.0 mm/year, suggest typical earthquakes of M6.0 to M7.0 (Rockwell, 1985).

San Jacinto Fault: The San Jacinto Fault is located 60 to 82 miles to the northeast of the site. Refer to Figure Nos. VIIIa-b. The San Jacinto Fault Zone consists of a series of closely spaced faults, including the Coyote Creek Fault, that form the western margin of the San Jacinto Mountains. The fault zone extends from its junction with the San Andreas Fault in San Bernardino, southeasterly toward the Brawley area, where it continues south of the international border as the Imperial Transform Fault. (Earth Consultants International [ECI], 2009)

The San Jacinto Fault Zone has a high level of historical seismic activity, with at least 10 damaging earthquakes (M6.0 to M7.0) having occurred on this fault zone between 1890 and 1986. Earthquakes on the San Jacinto Fault in 1899 and 1918 caused fatalities in the Riverside County area. Offset across this fault is predominantly right-lateral, similar to the San Andreas Fault, although some investigators have suggested that dip-slip motion contributes up to 10% of the net slip. (ECI, 2009)

The segments of the San Jacinto Fault that are of most concern to major metropolitan areas are the San Bernardino, San Jacinto Valley and Anza segments. Fault slip rates on the various segments of the San Jacinto are less well constrained than for the San Andreas Fault, but the available data suggest slip rates of 12 ±6 mm/yr for the northern segments of the fault, and slip rates of 4 ±2 mm/yr for the southern segments. For large ground-rupturing earthquakes on the San Jacinto Fault, various investigators have suggested a recurrence interval of 150 to 300 years. The Working Group on California Earthquake Probabilities (WGCEP, 2008) has estimated that there is a 31 percent probability that an earthquake of M6.7 or greater will occur within 30 years on this fault. Maximum credible earthquakes of M6.7, M6.9 and M7.2 are expected on the San Bernardino, San Jacinto Valley and Anza segments, respectively, capable of generating peak horizontal ground accelerations of 0.48 to 0.53g in the County of Riverside, (ECI, 2009). A M5.4 earthquake occurred on the San Jacinto Fault on July 7, 2010. The United States Geological Survey has issued the following statements with respect to the recent seismic activity on southern California faults:

> The San Jacinto fault, along with the Elsinore, San Andreas, and other faults, is part of the plate boundary that accommodates about 2 inches/year of motion as the Pacific plate moves northwest relative to the North American plate. The largest recent earthquake on the San Jacinto fault, near this location, the M6.5 1968 Borrego Mountain earthquake April 8, 1968, occurred about 25 miles southeast of the July 7, 2010 M5.4 earthquake
>
> This M5.4 earthquake follows the 4th of April 2010, Easter Sunday, M7.2 earthquake, located about 125 miles to the south, well south of the US Mexico international border. A M4.9 earthquake occurred in the same area on June 12th at 8:08 pm (Pacific Time). Thus, this section of the San Jacinto fault remains active.
>
> Seismologists are watching two major earthquake faults in southern California. The San Jacinto fault, the most active earthquake fault in southern California, extends for more than 100 miles from the international border into San Bernardino and Riverside, a major

metropolitan area often called the Inland Empire. The Elsinore fault is more than 110 miles long, and extends into the Orange County and Los Angeles area as the Whittier fault. The Elsinore fault is capable of a major earthquake that would significantly affect the large metropolitan areas of southern California. The Elsinore fault has not hosted a major earthquake in more than 100 years. The occurrence of these earthquakes along the San Jacinto fault and continued aftershocks demonstrates that the earthquake activity in the region remains at an elevated level. The San Jacinto fault is known as the most active earthquake fault in southern California. Caltech and USGS seismologist continue to monitor the ongoing earthquake activity using the Caltech/USGS Southern California Seismic Network and a GPS network of more than 100 stations.

B. Slope Stability

We have performed slope stability analysis based on our downhole stratigraphy observations in our exploratory borings, the laboratory test results from retrieved soil samples collected during the drilling, our field review of site conditions, our review of aerial photos, review of pertinent documents and geologic maps, and our experience with similar formational units in the La Jolla area of San Diego. The slope stability analyses were performed along three sections, A-A′, B-B′ C-C′ (see Figure Nos. VIa-c). Section A-A′ and Section C-C′ do not include the currently planned project. Section B-B′ extends across the eastern side of the property where encountered bedding in the Quaternary Very Old Paralic Deposits (Qvop)/Lindavista Formation (Qln) materials is massive and flat lying. Section B-B′ includes the currently planned project. The locations of this (and other) cross sections are presented on the Site Plan and Geologic Map, Figure No. IIb.

Downhole geologic observations elsewhere on the Reserve site, in borings B-1 and B-2, revealed the upper 80 feet of the encountered formational materials to consist of well consolidated, high-strength, fine-grained sandy clays and silts, silty clays, clayey silts and minor silty sand. No landslide deposits, out-of-slope bedding, remolded bedding planes or adverse joint sets were observed.

Direct shear testing on undisturbed soil samples revealed the soil materials to have high strength characteristics. Angles of internal friction averaged 38.5 degrees and cohesions averaged 4,700 psf. In order to be conservative and provide an added factor of safety, we have utilized 24 degrees angle of internal friction and 450 psf cohesion in our slope stability analyses.

Areas with existing loose fill soils that are not removed and properly recompacted (or re-sloped and protected from surface erosion) may undergo either sliding, shallow slump failures, or mud-sliding after heavy rainstorm events. These areas would not adversely affect the current project location.

We performed the slope stability calculations by using the *GSTABL7* with STEDWIN version 2004 program. The program utilizes the Bishop Simplified method of limit equilibrium slope stability conditions. The program calculates the factor of safety against shear soil failure on potential circular slide surfaces. The sliding surfaces start on points chosen on the left side of the slope and exit between two points chosen on the right side of the slope. As a minimum, 40 potential slide surfaces are drawn from each point of the left side of the slope, and the factor of safety against shear soil failure is calculated for each sliding block on each circular surface exiting between the two points. The program output figure shows the lowest safety factors for all the calculated surfaces and the calculated factor of safety for each. Soil strength values, geometry, water conditions, have been input in the program calculations based on geological observations at the site.

Based on our slope stability analysis, a factor of safety (FS) less than 1.5 against slope face failure does not exist at any location across the property, including the currently planned 2016 project. Refer to our Slope Stability results in Appendix B.

C. Other Geologic Hazards

Ground Rupture: Ground rupture is characterized by bedrock slippage along an established fault and may result in displacement of the ground surface. For ground rupture to occur along a fault, an earthquake usually exceeds M5.0. If a M5.0 earthquake were to take place on a local fault, an estimated surface-rupture length 1 mile long could be expected (Greensfelder, 1974). The currently planned 2016 project site is not directly on a known active fault trace and, therefore, the risk of ground rupture affecting planned building pad portions of the property is considered remote.

Ground Shaking: Structural damage caused by seismically induced ground shaking is a detrimental effect directly related to faulting and earthquake activity. Ground shaking is considered to be the greatest seismic hazard in San Diego County. The intensity of ground shaking is dependent on the magnitude of the earthquake, the distance from the earthquake, and the seismic response characteristics of underlying soils and geologic units. Earthquakes of M5.0 or greater are generally associated with notable to significant damage. It is our opinion that the most serious damage to the site would be caused by a large earthquake originating on active strands within the Rose Canyon Fault Zone. Although the chance of such an event is remote, it could occur within the useful life of the structure. Ground shaking will be experienced at the site from earthquakes on active Southern California faults and active faults in northwestern Mexico.

Landslides: Based upon our exploration of the entire Reserve site, our downhole logging, our geologic traverse, review of photographs and the referenced geologic maps (Kennedy, 1975; Kennedy and Tan, 2008), and other geologic references, it is our opinion that there are no deep-seated ancient landslides located on the currently planned 2016 project site. Refer to Section VIII of this report.

Liquefaction: The liquefaction of saturated sands during earthquakes can be a major cause of damage to buildings. Liquefaction is the process by which soils are transformed into a viscous fluid that will flow as a liquid when unconfined. It occurs primarily in loose, saturated sands and silts when they are sufficiently shaken by an earthquake. On this site, the risk of liquefaction of foundation materials due to seismic shaking is considered to be negligible due to the very stiff/dense nature of the natural-ground material and the lack of a shallow static groundwater surface under the site. The currently planned 2016 project site does not have a potential for soil strength loss to occur due to a seismic event.

Flooding and Tsunami: The elevation and location of the property precludes direct risk from these hazards.

D. Geologic Hazards Summary

As indicated on City of San Diego geologic hazard maps, the entire Reserve property is located in an area mapped as having destabilizing geologic conditions. These include a conjectured landslide and an inferred fault as well as concerns for the orientation of formational bedding. These concerns have been investigated via our research and explored by direct observation in our large-diameter borings, shallower trench excavations, geologic traverse, and laboratory soil testing of natural ground formational samples retrieved from the property. Based on the results of our investigation it is our opinion that an ancient landslide does not exist at the Reserve site; and further, a fault does not exist at the Reserve site. Slope stability analyses performed using up-to-date topographic information and the results of the soil strength/shear testing also indicate that slopes across the site have factors of safety in excess of 1.5. As previously described, existing uncontrolled fill soils may become unstable if they are not removed and recompacted or re-sloped and stabilized.

In our opinion, there are no geologic hazards on the currently planned 2016 project site, part of the Reserve property, that would preclude the residential development as currently planned.

X. __GROUNDWATER__

Groundwater was not encountered during the course of our field investigation. We do not expect significant groundwater problems to develop in the future if the property is developed as proposed and proper drainage and subdrainage are maintained.

It should be kept in mind that grading operations will change surface drainage patterns and reduce permeabilities due to the densification of compacted soils. Such changes of surface and subsurface hydrologic conditions, plus irrigation of landscaping or significant increases in rainfall, may result in the appearance of surface or near-surface water at locations where none existed previously. The damage from such water is expected to be localized and cosmetic in nature, if good positive drainage is implemented, as recommended in this report, during and at the completion of construction.

It must be understood that unless discovered during initial site exploration or encountered during site grading operations, it is extremely difficult to predict if or where perched or true groundwater conditions may appear in the future. When site fill or formational soils are fine-grained and of low permeability, water problems may not become apparent for extended periods of time.

Water conditions, where suspected or encountered during grading operations, should be evaluated and remedied by the project civil and geotechnical consultants. The project developer and the property owner, however, must realize that post-construction appearances of groundwater may have to be dealt with on a site-specific basis.

On properties such as the subject site where formational materials exist at relatively shallow depths, even normal landscape irrigation practices or periods of extended rainfall can result in shallow “perched” water conditions. The perching (shallow depth) accumulation of water on a low permeability surface can result in areas of persistent wetting and drowning of lawns, plants and trees. Resolution of such conditions, should they occur, may require site-specific design and construction of subdrain and shallow “wick” drain dewatering systems.

Project site formational deposits are dense to very dense; therefore they are not considered suitable for on-site storm water infiltration.

Subsurface drainage with a properly designed and constructed subdrain system will be required along with continuous back drainage behind any proposed lower-level basement walls, property line retaining walls, or any perimeter stem walls for raised-wood floors where the outside grades are higher than the crawl space grades.

Furthermore, crawl spaces (if constructed) should be provided with the proper cross-ventilation to help reduce the potential for moisture-related problems.

XI. SUMMARY OF FINDINGS

Our subsurface investigation revealed that the currently planned 2016 area of the Reserve property is underlain by very competent, high-strength formational materials of the Quaternary Lindavista Formation (Qln), currently referred to as Quaternary Very Old Paralic Deposits (Qvop). The formational units are covered in the most part with a shallow thickness of sandy slopewash soils, topsoils and locally varying thicknesses of fill soils.

The mapped (GHC Zone 12) potentially active fault does not exist on the site and therefore will not affect the currently planned 2016 project. Trenching excavation across the mapped fault zone revealed no breakage or offset in uniformly dipping interbeds of claystones and sandstone of the Scripps Formation. Nearby surficial outcrops also do not display faulting offset of the layered formational materials and reveal generally consistent attitudes between boring, trench and outcrop exposures.

The undifferentiated Tertiary Scripps/Ardath Shale Formation bedding is parallel to or dips out of or parallel to a slope across the northeastern portion of the site with measured attitudes of up to 32 degrees to the south and southwest. The current 2016 project area is not planned for this area and these materials are not at the ground surface in the project area.

Shallow surficial slopewash and topsoil materials and the existing old fill soils are not currently suitable for support of the planned improvements. The slopewash and fill will have to be removed and recompacted if required to achieve planned design grades. Clay topsoils, if encountered, are to be removed and exported to offsite or approved non-project areas on the site. Old fill soils adjacent to a canyon and an existing unpaved road on the northeastern portion of the property, northeast of the current project but possibly affecting project site access roads, will have to be dressed to improve their erosion resistance, if planned to be left in place.

Measurements of the bedding attitudes within the large-diameter borings through the Quaternary Very Old Paralic Deposits/Lindavista Formation, undifferentiated Ardath/Scripps Formations and Ardath Shale in our exploratory trenches and on our geologic traverse revealed no significant fracturing indicative of landsliding or faulting. No remolded clay gouge or bedding seams characteristic of bedding plane (parallel) landslide slip surfaces were observed within the borings, trenches or on outcrops.

Slope stability evaluations indicate the hillsides across the property, including the current 2016 project, have a factor of safety against deep-seated failure of 1.5 or greater and are suitable for development as a residential project per guidelines of the City of San Diego.

We have also provided herein recommendations for preparation of the site for the currently planned new conventional residential improvements as well as preliminary foundation and other soil design recommendations. All excavations should be monitored for newly exposed geologic conditions during the construction phase. Further, as project planning proceeds and the actual locations of the planned house pad, roads and other improvements are determined, additional shallow exploration may be required to confirm local soil conditions. Additional recommendations may be issued.

XII. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based upon the practical field investigation conducted by our firm and the resulting laboratory tests, in conjunction with our knowledge and experience with similar soils in the La Jolla area of the City of San Diego.

In our opinion, the site is suited for the proposed currently planned 2016 residential development provided the following recommendations are implemented during site development. Conventional construction techniques and materials can be utilized. In addition, in our opinion, development of the site as a residential project would not destabilize adjacent and nearby structures and property improvements or right-of-ways.

The opinions, conclusions, and recommendations presented in this report are contingent upon ***Geotechnical Exploration, Inc.*** being retained to review the final plans and specifications as they are developed and to observe and test the site earthwork and installation of foundations. Accordingly, we recommend that the following paragraph be included on the grading and foundation plans for the project:

> If the geotechnical consultant of record is changed for the project, the work shall be stopped until the replacement has agreed in writing to accept the responsibility within their area of technical competence for approval upon completion of the work. It shall be the responsibility of the permittee to notify the governing agency in writing of such change prior to the commencement or recommencement of grading and/or foundation installation work.

At the time plans for the project become available they should be provided for our review to establish they are in accordance with our recommendations.

A. *Seismic Design Criteria*

1. *Seismic Data Bases:* The estimation of the peak ground acceleration and the repeatable high ground acceleration (RHGA) likely to occur at the site is based on the known significant local and regional faults within 100 miles of the site. The Modified Mercalli Index, a table of ground shaking intensity, is provided as Appendix B.

2. *Seismic Design Criteria:* The proposed structure should be designed in accordance with the 2013 CBC, which incorporates by reference the ASCE 7-10 for seismic design. We have determined the mapped spectral acceleration values for the site based on latitude 32.8370 degrees north and longitude 117.2581 degrees west, utilizing a program titled "*U.S. Seismic Design Maps and Tools*" provided by the USGS, which provides a solution for ASCE 7-10 utilizing digitized files for the Spectral Acceleration maps. See Appendix C.

3. *Structure and Foundation Design*: The design of the new addition structures and foundations should be based on Seismic Design Category D.

4. *Spectral Acceleration and Design Values*: The structural seismic design, when applicable, should be based on the following values, which are based on the site location, soil characteristics, and seismic maps by USGS, as required by the 2013 CBC. A response Spectrum Acceleration (SA) vs. Period (T) for the site is also included in Appendix C. The Site D values for this property are:

TABLE I

Mapped Spectral Acceleration *Values and Design Parameters*

S_s	S_1	F_a	F_v	S_{ms}	S_{m1}	S_{ds}	S_{d1}

1.272	0.491	1.0	1.509	1.272	0.741	0.848	0.494

B. Preparation of Soils for Site Development

3. *Clearing and Stripping:* The planned building pad, roadways and other improvements will require grading excavation. Vegetation will require removal prior to the preparation of building pad and areas of associated improvements. This includes any roots from existing trees and shrubbery. Holes resulting from the removal of root systems or other buried obstructions that extend below the planned grades should be cleared and backfilled with properly compacted fill.

4. *Treatment of Existing Fill and Slopewash:* It is anticipated that 2 to 3 feet of slopewash soils overlie Quaternary Very Old Paralic Deposits (Qvop)/ Lindavista Formation (Qln) formational materials in the currently planned 2016 project area. In order to provide suitable foundation support for improvements planned to be located in areas of existing slopewash soils, these soils should be removed to expose the underlying competent formational soils. New structures and improvements can be constructed on the good-bearing underlying formational soils or the existing slopewash and/or fill soils may be replaced as properly recompacted fill.

   The areal extent and depth required to remove the slopewash soils should be determined by our representatives during the excavation work based on examination of the soils being exposed, but should be either 8 feet beyond the edge of the improvements or perimeter foundations, or to a distance at least equal to the depth of excavations, whichever is larger.

Any rigid improvements founded on the loose surface soils can be expected to undergo movement and possible damage. ***Geotechnical Exploration, Inc.*** takes no responsibility for the performance of any improvements built on loose natural soils or inadequately compacted fills. Subgrade soils in any exterior area receiving concrete improvements should be verified for compaction and moisture within 48 hours prior to concrete placement. Placed and compacted fill soils should be tested at least every 2 feet in vertical depth.

5. *Subgrade Preparation:* After the site has been cleared, stripped, and the required excavations made, the exposed subgrade soils in areas to receive fill and/or building improvements should be scarified to a depth of 6 inches, moisture conditioned, and compacted to the requirements for structural fill.

6. *Expansive Soil Conditions:* If the medium expansive soils are to be used as fill, they should be scarified, moisture conditioned to 5 percent above Optimum Moisture content and compacted to 90 percent. Soils of medium or greater expansion potential should not be used as retaining wall backfill soils. If expansive soils with high or greater Expansion Indices are encountered near the surface in pad or improvement excavations, they should preferably be removed and replaced with very low to low expansion soils, or the planned improvements should be designed to withstand the expansive soil pressures.

7. *Material for Fill:* All existing on-site soils with an organic content of less than 3 percent by volume are, in general, suitable for use as fill. Any required imported fill material should be a low-expansion potential (Expansion Index of 50 or less per ASTM D4829-08). In addition, both imported and existing on-site materials for use as fill should not contain rocks or lumps more than 6 inches in greatest dimension. All materials for use as fill should be approved

by our firm prior to filling. Backfill material to be placed behind retaining walls should be of low expansion potential (EI less than 50) and with particles no larger than 3 inches in diameter. Low expansive material should extend to a distance behind the wall equal to half the height of soil being retained by the wall.

8. *Fill Compaction:* All structural fill should be compacted to a minimum degree of compaction of 90 percent based upon ASTM D1557-09. Fill material should be spread and compacted in uniform horizontal lifts not exceeding 8 inches in uncompacted thickness. Before compaction begins, the fill should be brought to a water content that will permit proper compaction by either: (1) aerating and drying the fill if it is too wet, or (2) moistening the fill with water if it is too dry. Each lift should be thoroughly mixed before compaction to ensure a uniform distribution of moisture. As previously indicated, clayey soils – where allowed – should include a moisture content of at least 5 percent over optimum.

   No uncontrolled fill soils should remain on the project site after completion of the site work. In the event that temporary ramps or pads are constructed of uncontrolled fill soils, the loose fill soils should be removed and/or recompacted prior to completion of the grading operation.

9. *Trench and Retaining Wall Backfill:* All backfill soils placed in utility trenches or behind retaining walls should be compacted to at least 90 percent of Maximum Dry Density. Our experience has shown that even shallow, narrow trenches (such as for irrigation and electrical lines) that are not properly compacted, can result in problems, particularly with respect to shallow groundwater accumulation and migration. Backfill soils placed behind

retaining walls and/or crawl space retaining walls should be installed as early as the retaining walls are capable of supporting lateral loads.

C. Design Parameters Continuous Footings

In order to support the proposed new residential structure on conventional continuous concrete foundations the following recommendations should be followed.

10. *Footings:* We recommend that both one- and two-story structures be supported on conventional, individual-spread and/or continuous footing foundations bearing on undisturbed formational materials and/or properly compacted fill material. Footings should be founded at least 18 inches below the lowest adjacent finished grade. Footings located adjacent to utility trenches should have their bearing surfaces situated below an imaginary 1.5:1.0 plane projected upward from the bottom edge of the adjacent utility trench.

    At the recommended depths, footings on compacted fill main floor may be designed for allowable bearing pressures of 2,000 pounds per square foot (psf) for combined dead and live loads and 2,650 psf for all loads, including wind or seismic. The footings should, however, have a minimum width of 12 inches. If footings are to be extended through the properly compacted fill soils to bear on the formational materials the footings may be designed for 3,000 psf for dead and live loads, and for 4,000 psf when including wind or seismic loads. Foundations close to slopes should be provided with a setback of 8 feet measured from the top of the foundation (see Figure No. IX).

11. *Foundation Reinforcement*: All continuous footings should contain top and bottom reinforcement to provide structural continuity and to permit spanning of local irregularities. We recommend that a minimum of two No. 5 top and two No. 5 bottom reinforcing bars be provided in the footings. A minimum clearance of 3 inches should be maintained between steel reinforcement and the bottom or sides of the footing. Isolated square footings should contain, as a minimum, a grid of three No. 4 steel bars on 12-inch centers, both ways. In order for us to offer an opinion as to whether the footings are founded on soils of sufficient load bearing capacity, it is essential that our representative observe the footing excavations prior to the placement of reinforcing steel or concrete.

    NOTE: The project Civil/Structural Engineer should review all reinforcing schedules. The reinforcing minimums recommended herein are not to be construed as structural designs, but merely as minimum reinforcement to reduce the potential for cracking and separations.

12. *Lateral Loads:* Lateral load resistance for structures supported on footing foundations may be developed in friction between the foundation bottom and the supporting subgrade. An allowable friction coefficient of 0.35 is considered applicable. An additional allowable passive resistance equal to an equivalent fluid weight of 300 pounds per cubic foot acting against the foundations may be used in design provided the footings are poured neat against the adjacent undisturbed formational materials and/or properly compacted fill materials. These lateral resistance values assume a level surface in front of the footing for a minimum distance of four times the embedment depth of the footing.

13. *Settlement:* Settlements under building loads are expected to be within tolerable limits for the proposed improvements. For footings designed and built in accordance with the recommendations presented in the preceding paragraphs, we anticipate that total settlements should not exceed 1 inch and that post-construction differential settlements should be less than ½-inch with a maximum angular rotation of 1/300 provided that the difference in fill thickness across the building area is less than 12 feet.

D. Concrete Slab-on-grade Criteria

14. *Minimum Floor Slab Reinforcement:* Based on our experience, we have found that, for various reasons, floor slabs occasionally crack, causing brittle surfaces such as ceramic tiles to become damaged. Therefore, we recommend that all slabs on-grade contain at least a minimum amount of reinforcing steel to reduce the separation of cracks, should they occur. Interior slabs on-grade should be a minimum of 4 inches actual thickness and be reinforced with No. 3 bars on 15-inch centers, both ways, placed at midheight in the slab. Slab subgrade soil should be verified by a ***Geotechnical Exploration, Inc***. representative to have the proper moisture content within 48 hours prior to placement of the vapor barrier and pouring of concrete.

15. *Slab Moisture Protection and Vapor Barrier Membrane:* Although it is not the responsibility of geotechnical engineering firms to provide moisture protection recommendations, as a service to our clients we provide the following discussion and suggested minimum protection criteria. Actual recommendations should be provided by the architect and waterproofing consultants or product manufacturer.

Soil moisture vapor can result in damage to moisture-sensitive floors, some floor sealers, or sensitive equipment in direct contact with the floor, in addition to mold and staining on slabs, walls, and carpets. The common practice in Southern California is to place vapor retarders made of PVC, or of polyethylene. PVC retarders are made in thickness ranging from 10- to 60-mil. Polyethylene retarders, called visqueen, range from 5- to 10-mil in thickness. These products are no longer considered adequate for moisture protection and can actually deteriorate over time.

Specialty vapor retarding products possess higher tensile strength and are more specifically designed for and intended to retard moisture transmission into and through concrete slabs. The use of such products is highly recommended for reduction of floor slab moisture emission.

The following American Society for Testing and Materials (ASTM) and American Concrete Institute (ACI) sections address the issue of moisture transmission into and through concrete slabs: ASTM E1745-97 (2009) Standard Specification for Plastic Water Vapor Retarders Used in Contact Concrete Slabs; ASTM E154-88 (2005) Standard Test Methods for Water Vapor Retarders Used in Contact with Earth; ASTM E96-95 Standard Test Methods for Water Vapor Transmission of Materials; ASTM E1643-98 (2009) Standard Practice for Installation of Water Vapor Retarders Used in Contact Under Concrete Slabs; and ACI 302.2R-06 Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials.

15.1 Based on the above, we recommend that the vapor barrier consist of a minimum 15-mil extruded polyolefin plastic (no recycled content or woven materials permitted). Permeance as tested before and after mandatory conditioning (ASTM E1745 Section 7.1 and sub-paragraphs 7.1.1-7.1.5) should be less than 0.01 U.S. perms (grains/square foot/hour/inch of mercury [Hg]) and comply with the ASTM E1745 Class A requirements. Installation of vapor barriers should be in accordance with ASTM E1643. The basis of design is 15-mil StegoWrap vapor barrier placed per the manufacturer's guidelines. Reef Industries Vapor Guard membrane has also been shown to achieve a permeance of less than 0.01 perms.

Our suggested acceptable moisture retardant membranes are based on a report entitled "*Report of Water Vapor Permeation Testing of Construction Vapor Barrier Materials*" by Dr. Kay Cooksey, Ph.D., Clemson University, Dept. of Packaging Science, 2009-10. The membrane may be placed directly on properly compacted subgrade soils and directly underneath the slab. Proper slab curing is required to help prevent slab curling.

15.2 Common to all acceptable products, vapor retarder/barrier joints must be lapped and sealed with mastic or the manufacturer's recommended tape or sealing products. In actual practice, stakes are often driven through the retarder material, equipment is dragged or rolled across the retarder, overlapping or jointing is not properly implemented, etc. All these construction deficiencies reduce the retarder's effectiveness. In no case should retarder/barrier products be punctured or gaps be allowed to form prior to or during concrete placement.

15.3 As previously stated, following placement of concrete floor slabs, sufficient drying time must be allowed prior to placement of any floor coverings. Premature placement of floor coverings may result in degradation of adhesive materials and loosening of the finish floor materials.

16. *Concrete Isolation Joints:* We recommend the project Civil/Structural Engineer incorporate isolation joints and sawcuts to at least one-fourth the thickness of the slab in any floor designs. The joints and cuts, if properly placed, should reduce the potential for and help control floor slab cracking. We recommend that concrete shrinkage joints be spaced no farther than approximately 20 feet apart, and also at re-entrant corners. However, due to a number of reasons (such as base preparation, construction techniques, curing procedures, and normal shrinkage of concrete), some cracking of slabs can be expected.

17. *Exterior Slab Reinforcement:* As a minimum for protection of on-site improvements, we recommend that all nonstructural concrete slabs (such as patios, sidewalks, etc.), be at least 4 inches in actual thickness, founded on properly compacted and tested fill or dense native formation and underlain by no more than 3 inches of clean leveling sand, with No. 3 bars at 18-inch centers, both ways, at the center of the slab, and contain adequate isolation and control joints.

    The performance of on-site improvements can be greatly affected by soil base preparation and the quality of construction. It is therefore important that all improvements are properly designed and constructed for the existing soil conditions. The improvements should not be built on loose soils or fills placed without our observation and testing. The subgrade of exterior improvements should be verified as properly prepared within 48 hours prior to concrete placement. A minimum thickness of 2 feet of properly recompacted soils should underlie the exterior slabs on-grade. Fill soils shall be placed on firm natural soils.

For exterior slabs with the minimum shrinkage reinforcement, control joints should be placed at spaces no farther than 15 feet apart or the width of the slab, whichever is less, and also at re-entrant corners. Control and isolation joints in exterior slabs should be sealed with elastomeric joint sealant. The sealant should be inspected every 6 months and be properly maintained.

18. *Concrete Pavement:* For preliminary estimating purposes assume that new driveway slabs should be at least 5½ inches thick and rest on properly prepared and compacted subgrade soils. Subgrade soil for the driveway should be dense/hard or, if fill, be compacted to at least 95 percent of Maximum Dry Density. The concrete should be at least 3,500 psi compressive strength, with control joints no farther than 15 feet apart and also at re-entrant corners. Pavement joints should be properly sealed with permanent joint sealant, as required in sections 201.3.6 through 201.3.8 of the Standard Specifications for Public Work Construction, 2015 Edition. All slab joints shall be placed within 12 hours of concrete placement or as soon as the concrete sets, whichever occurs sooner. The final pavement cross section shall be determined based on R-value soil tests and the anticipated traffic index. R-value tests shall be performed on soil samples obtained after completion of driveway rough grading.

19. *Cal-OSHA Guidelines*: All excavations should follow Cal-OSHA guidelines for safety purposes.

E. Alternative Design Parameters for Pier Foundations

If it is desired to reduce the impact of grading preparation of planned building pads, an alternative for support of the structures would be to use deepened pier or drilled caisson foundation systems for support of the proposed new residential structures. Specific deepened pier foundation recommendations will be provided for once the building type and location are defined. Since slopewash thickness and geologic conditions vary across the site, no specific pier or caisson recommendations are provided at this time.

F. Slopes

It is not anticipated that significant new slopes will be created as part of the development of the currently planned project. The existing natural slope in the project area is considered to be stable. The following recommendations are provided should significant slope be created as an alternative grading option.

20. *Slope Stability*: The existing Reserve site slopes have been evaluated for slope stability as discussed previously, and are stable as described under static conditions and should not be affected negatively by the construction of the structures and associated improvements. Based on slope stability calculations, the calculated factor of safety for gross and shallow slope stability of the project site soils is at least 1.5. Refer to Appendix E for results of slope stability analysis for the proposed project.

21. *Temporary Slopes*: A representative of ***Geotechnical Exploration, Inc.*** must observe any steep temporary slopes ***during construction***. In the event that soils and formational material comprising a slope are not as anticipated (i.e.,

with favorable geology), any required slope design changes would be presented at that time. In general, temporary cut slopes in firm natural soils or properly compacted fill can be made at slope ratios of 0.5:1.0 (horizontal to vertical) but they may not be surcharged within 10 feet of the slope top. Another option would consist of making a vertical cut in dense formational soils no higher than 6 feet in the lower part of the excavation and at a 0.75:1.0 slope ratio in compacted fills for the remaining portion of the cut. If the temporary cuts cannot be fully developed, temporary shoring should be implemented.

Where not superseded by specific recommendations presented in this report, trenches, excavations and temporary slopes at the subject site should be constructed in accordance with Title 8, Construction Safety Orders, issued by Cal-OSHA.

22. *Slope Top/Face Performance:* The soils that occur in close proximity to the top or face of even properly compacted fill or dense/stiff natural ground cut slopes often possess poor lateral stability. The degree of lateral and vertical deformation depends on the inherent expansion and strength characteristics of the soil types comprising the slope, slope steepness and height, loosening of slope face soils by burrowing rodents, and irrigation and vegetation maintenance practices, as well as the quality of compaction of fill soils. Structures and other improvements could suffer damage due to these soil movement factors if not properly designed to accommodate or withstand such movement.

Fill or cut slopes more than 10 feet in height should be constructed at a 2.0:1.0 slope gradient. Slopes less than 10 feet in height (with favorable geology) may be constructed at a 1.5:1.0 slope ratio

23. *Slope Top Structure Performance:* Rigid improvements such as top-of-slope walls, columns, decorative planters, concrete flatwork, swimming pools and other similar types of improvements can be expected to display varying degrees of separation typical of improvements constructed at the top of a slope. The separations result primarily from slope top lateral and vertical soil deformation processes. These separations often occur regardless of being underlain by cut or fill slope material. Proximity to a slope top is often the primary factor affecting the degree of separations occurring.

Typical and to-be-expected separations can range from minimal to up to 1 inch or greater in width. In order to minimize the effect of slope-top lateral soil deformation, we recommend that the top-of-slope improvements be designed with flexible connections and joints in rigid structures so that the separations do not result in visually apparent cracking damage and/or can be cosmetically dressed as part of the ongoing property maintenance. These flexible connections may include “slip joints” in wrought iron fencing, evenly spaced vertical joints in block walls or fences, control joints with flexible caulking in exterior flatwork improvements, etc.

In addition, use of planters to provide separation between top-of-slope hardscape such as patio slabs and pool decking from top-of-slope walls can aid greatly in reducing cosmetic cracking and separations in exterior improvements. Actual materials and techniques would need to be determined by the project architect or the landscape architect for individual properties.

Steel dowels placed in flatwork may prevent noticeable vertical differentials, but if provided with a slip-end they may still allow some lateral displacement.

G. Retaining Wall Design Criteria

24. *Design Parameters – Unrestrained:* The active earth pressure (to be utilized in the design of any cantilever retaining walls, utilizing ***imported*** very low- to low-expansive soils [EI less than 50] as backfill) should be based on an *Equivalent Fluid Weight* of 38 pounds per cubic foot (for level backfill only). In the event that a retaining wall is surcharged by sloping backfill, the design active earth pressure should be based on the appropriate Equivalent Fluid Weight presented in the following table. If the retaining wall will retain medium expansive soils (such as on-site soils), the design soil pressure should be 52 pcf for level backfill and 72 pcf for sloping backfill. Swimming pool walls may be designed for medium expansive soils producing a pressure of 52 pcf for static conditions.

	Height of Slope/Height of Wall*			
Slope Ratio	**0.25**	**0.50**	**0.75**	**1.00(+)**
2.0:1.0	42	48	50	52
1.5:1.0 (where allowed)	52	62	68	70

*To determine design active earth pressures for ratios intermediate to those presented, interpolate between the stated values.

25. *Design Parameters – Restrained:* Retaining walls designed for a restrained condition should utilize a uniform pressure equal to 9xH (nine times the total height of retained soil, considered in pounds per square foot) considered as acting everywhere on the back of the wall ***in addition to the design active Equivalent Fluid Weight***. The soil pressure produced by any footings, improvements, or any other surcharge placed within a horizontal distance

equal to the height of the retaining portion of the wall should be included in the wall design pressure. The recommended lateral soil pressures are based on the assumption that no loose soils or unstable soil wedges will be retained by the retaining wall. Backfill soils should consist of low-expansive soils (EI less than 50) and should be placed from the heel of the foundation to the ground surface within the wedge formed by a plane 30 degrees from vertical passing by the heel of the foundation, and the back face of the retaining wall. If suing on site soils, restrained walls with level backfill may be designed for 75 psf; and 100 pcf for 2.0 to 1.0, horizontal to vertical, sloping backfill.

26. *Surcharge Loads:* Any loads placed on the active wedge behind a cantilever wall should be included in the design by multiplying the load weight by a factor of 0.31. For a restrained wall, the lateral factor should be 0.47. For medium expansive soils, the factors will be 0.42 and 0.59, respectively.

When retaining walls exceed 6 feet in retained height or swimming pools are deeper than 6 feet, seismic soil pressures are required in their design. Swimming pools and structures constructed near slope tops are also required to include seismic soil pressures. For cantilever, unrestrained retaining walls, the recommended seismic pressure should be 10 pcf applied in a triangular distribution. For restrained walls, the seismic pressure may be waived. The seismic pressure distribution should be added to the static pressure distribution.

27. *Wall Drainage:* Proper subdrains and free-draining backwall material or board drains (such as J-drain or Miradrain) should be installed behind all retaining walls (in addition to proper waterproofing) on the subject project. ***Geotechnical Exploration, Inc.*** will assume no liability for damage to

structures or improvements that is attributable to poor drainage. The architectural plans should clearly indicate that subdrains for any lower-level walls be placed at an elevation at least 1 foot below the bottom of the lower-level slabs. At least 0.5-percent gradient should be provided to the subdrain. The subdrain should be placed in an envelope of crushed rock gravel up to 1 inch in maximum diameter, and be wrapped with Mirafi 140N filter or equivalent (see Figure No. X, the Retaining Wall Backdrain and Waterproofing Schematic).

28. *Drainage Quality Control*: It must be understood that it is not within the scope of our services to provide quality control oversight for surface or subsurface drainage construction or retaining wall sealing and base of wall drain construction. It is the responsibility of the contractor and/or their retained construction inspection service provider to verify proper wall sealing, geofabric installation, protection board (if needed), drain depth below interior floor or yard surface, pipe percent slope to the outlet, etc.

H. Site Drainage Considerations

30. *Surface Drainage:* Adequate measures should be taken to properly finish-grade the property after the structure and other improvements are in place. Drainage waters from this site and adjacent properties should be directed away from the footings, floor slabs, and slopes, onto the natural drainage direction for this area or into properly designed and approved drainage facilities provided by the project civil engineer. Roof gutters and downspouts should be installed on the structures, with the runoff directed away from the foundations via closed drainage lines. Proper subsurface and surface drainage will help

minimize the potential for waters to seek the level of the bearing soils under the footings and floor slabs.

Failure to observe this recommendation could result in undermining and possible differential settlement of the structure or other improvements on the site or cause other moisture-related problems. Currently, the California Building Code (CBC) requires a minimum 1-percent surface gradient for proper drainage of building pads unless waived by the building official. Concrete pavement should have a minimum gradient of 0.5-percent. Swimming pool decks should be provided with a minimum 1 percent gradient directed toward area drains.

31. *Erosion Control*: In addition, appropriate erosion control measures should be taken at all times during and after construction to prevent surface runoff waters from entering footing excavations or ponding on finished building pad areas.

32. *Planter Drainage:* Planter areas, flower beds and planter boxes should be sloped to drain away from the footings and floor slabs at a gradient of at least 5 percent within 5 feet from the perimeter walls. Any planter areas adjacent to the residence or surrounded by concrete improvements should be provided with sufficient area drains to help with rapid runoff disposal. No water should be allowed to pond adjacent to the residence or other improvements or anywhere on the site.

I. General Recommendations

33. *Project Start Up Notification:* In order to reduce work delays during site development, this firm should be contacted 24 hours prior to any need for observation of footing excavations or field density testing of compacted fill soils. If possible, placement of formwork and steel reinforcement in footing excavations should not occur prior to observing the excavations; in the event that our observations reveal the need for deepening or redesigning foundation structures at any locations, any formwork or steel reinforcement in the affected footing excavation areas would have to be removed prior to correction of the observed problem (i.e., deepening the footing excavation, recompacting soil in the bottom of the excavation, etc.).

34. *Construction Best Management Practices (BMPs):* Sufficient BMPs must be installed to prevent silt, mud or other construction debris from being tracked into the adjacent street(s) or storm water conveyance systems due to construction vehicles or any other construction activity. The contractor is responsible for cleaning any such debris that may be in the street at the end of each workday or after a storm event that causes breach in the installed construction BMPs. All stockpiles of uncompacted soil and/or building materials that are intended to be left unprotected for any length of time during the rainy season are to be provided with erosion and sediment controls. Such soil must be protected each day when the probability of rain is 40% or greater.

    A concrete washout should be provided on all projects that propose the construction of any concrete improvements that are to be poured in place. All erosion/sediment control devices should be maintained in working order at all times. All slopes that are created or disturbed by construction activity must be protected against erosion and sediment transport at all times. The storage of all construction materials and equipment must be protected against any potential release of pollutants into the environment.

XIII. GRADING NOTES

Geotechnical Exploration, Inc. recommends that we be asked to verify the actual soil conditions revealed during site grading work and footing excavation to be as anticipated in the *"Update Report of Geotechnical and Geologic Investigation* " for the project. In addition, the compaction of any fill soils placed during site grading work must be observed and tested by the soil engineer. It is the responsibility of the grading contractor to comply with the requirements on the grading plans and the local grading ordinance. All retaining wall and trench backfill should be properly compacted. ***Geotechnical Exploration, Inc.*** will assume no liability for damage occurring due to improperly or uncompacted backfill placed without our observations and testing.

We recommend our firm review the project plans prior to submittal to verify that our recommendations have been properly incorporated into them. Additional or modified recommendations may be issued if warranted.

XIV. LIMITATIONS

Our conclusions and recommendations have been based on available data obtained from our preliminary field investigation and laboratory analysis, as well as our experience with similar soils and formational materials located in this area of La Jolla. Of necessity, we must assume a certain degree of continuity between exploratory excavations and/or natural exposures. In the event discrepancies are noted, additional recommendations may be issued, if required.

The work performed and recommendations presented herein are the result of an investigation and analysis that meet the contemporary standard of care in our profession within the County of San Diego. No warranty is provided.

This report should be considered valid for a period of two (2) years, and is subject to review by our firm following that time. The firm of ***Geotechnical Exploration, Inc.*** shall not be held responsible for changes to the physical condition of the property, such as addition of fill soils or changing drainage patterns, which occur subsequent to issuance of this report without our observations, testing, and approval.

As stated previously, it is not within the scope of our services to provide quality control oversight for surface or subsurface drainage construction or retaining wall sealing and base of wall drain construction. It is the responsibility of the contractor and/or their retained construction inspection service provider to verify proper wall sealing, geofabric installation, protection board (if needed), drain depth below interior floor or yard surface, pipe percent slope to the outlet, etc.

It is the responsibility of the owner and/or developer to ensure that the recommendations summarized in this report are carried out in the field operations and that our recommendations for design of this project are incorporated in the structural plans. We should be retained to review the project plans once they are available, to verify that our recommendations are adequately incorporated in the plans. Additional or revised recommendations may be necessary after our review.

This firm does not practice or consult in the field of safety engineering. We do not direct the contractor's operations, and we cannot be responsible for the safety of personnel other than our own. The safety of others is the responsibility of the contractor. The contractor should notify the owner if any of the recommended actions presented herein are considered to be unsafe.

The firm of ***Geotechnical Exploration, Inc.*** shall not be held responsible for changes to the physical condition of the property, such as addition of fill soils or changing drainage patterns, which occur subsequent to issuance of this report and the changes are made without our observations, testing, and approval.

This opportunity to be of service is sincerely appreciated. Should any questions arise concerning this report, please feel free to contact the undersigned. Reference to our **Job No. 10-9977.1** will expedite a reply to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

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Exp. 3/31/17

REFERENCES

Job No. 10-9977.1

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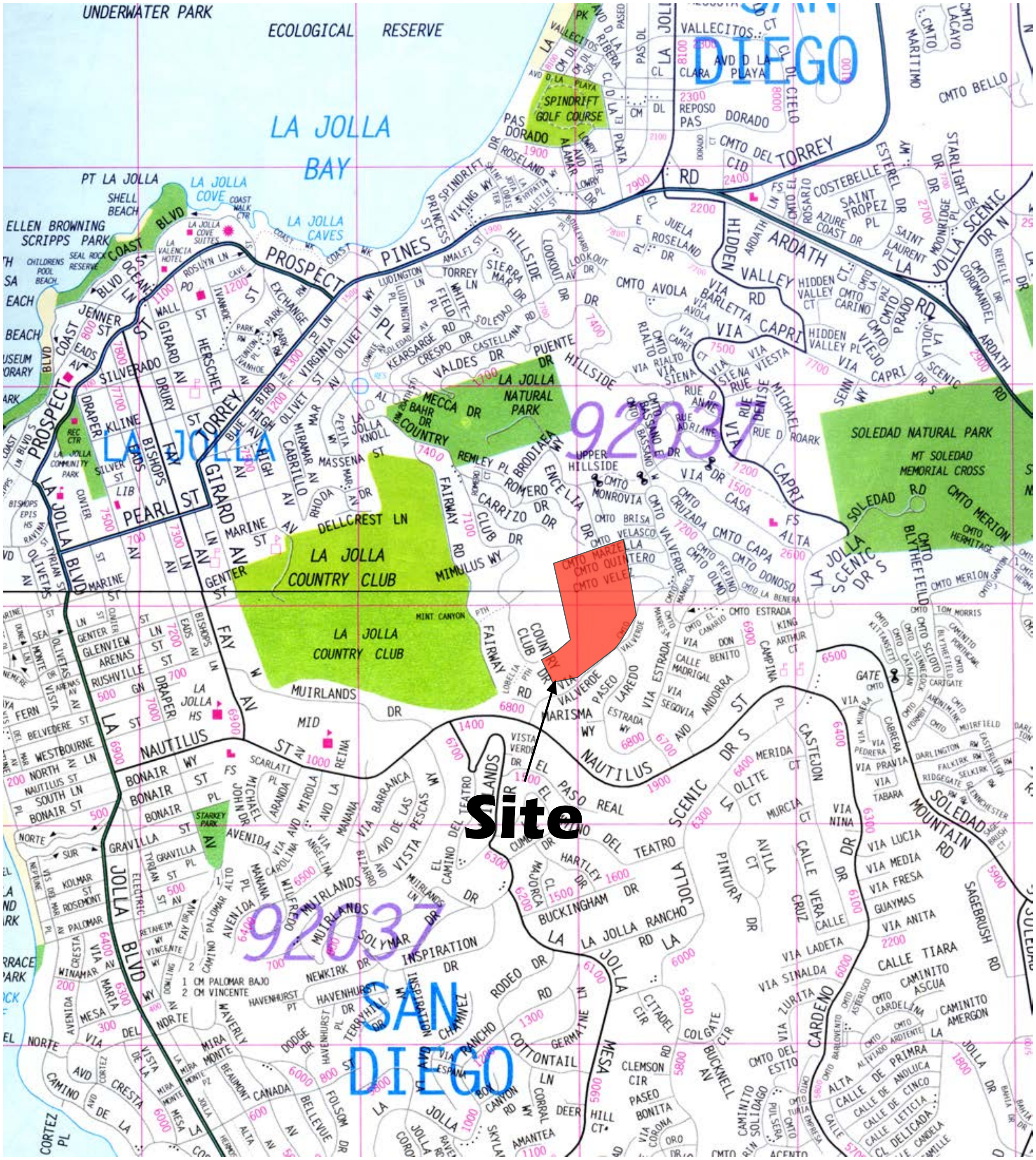
VICINITY MAP



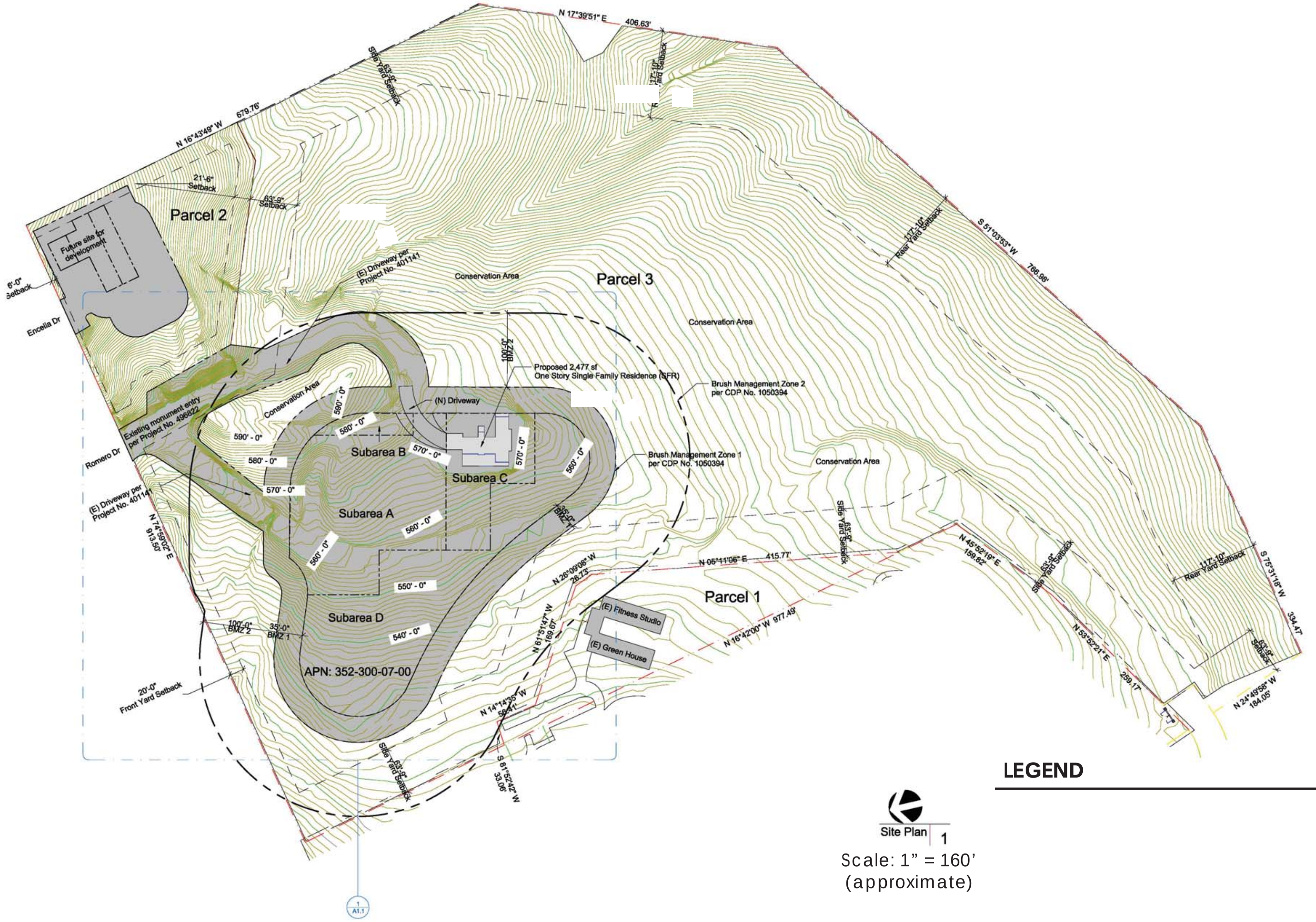
Thomas Bros. Guide San Diego County, pg 1247

The Reserve LLC.
Country Club Drive
La Jolla, CA.

Figure No. Ia
Job No. 10-9977.1

The Reserve LLC.
Country Club Drive
La Jolla, CA.

Figure No. Ib
Job No. 10-9977.1



LEGEND

Site Plan 1
Scale: 1" = 160'
(approximate)

Legend

▨	Limit of Development Area per Project No. 401141
— · · — · · —	Property Line
— — — — —	Setback Line
—·—·—·—·—	Limit of Future City Right of Way
—— - - - ——	Limit of Development Area
—-—-—-—-—	Limit of Development Envelope
—— — — ——	Brush Management Zone (BMZ) per Coastal Development Permit (CDP) No. 1050394

The Reserve: Single Family Residence
Romero Drive
La Jolla, CA 92037

Site Plan

Revision Schedule

Kent Coston
C-29876
Ren: 08/2017

DATE	07/26/2016
DRAWN	AV
PROJECT	1677

A1.0

SITE PLAN

Proposed Single Family Residence
The Reserve LLC.
Country Club Drive
La Jolla, CA.
Figure No. IIa
Job No. 10-9977.1

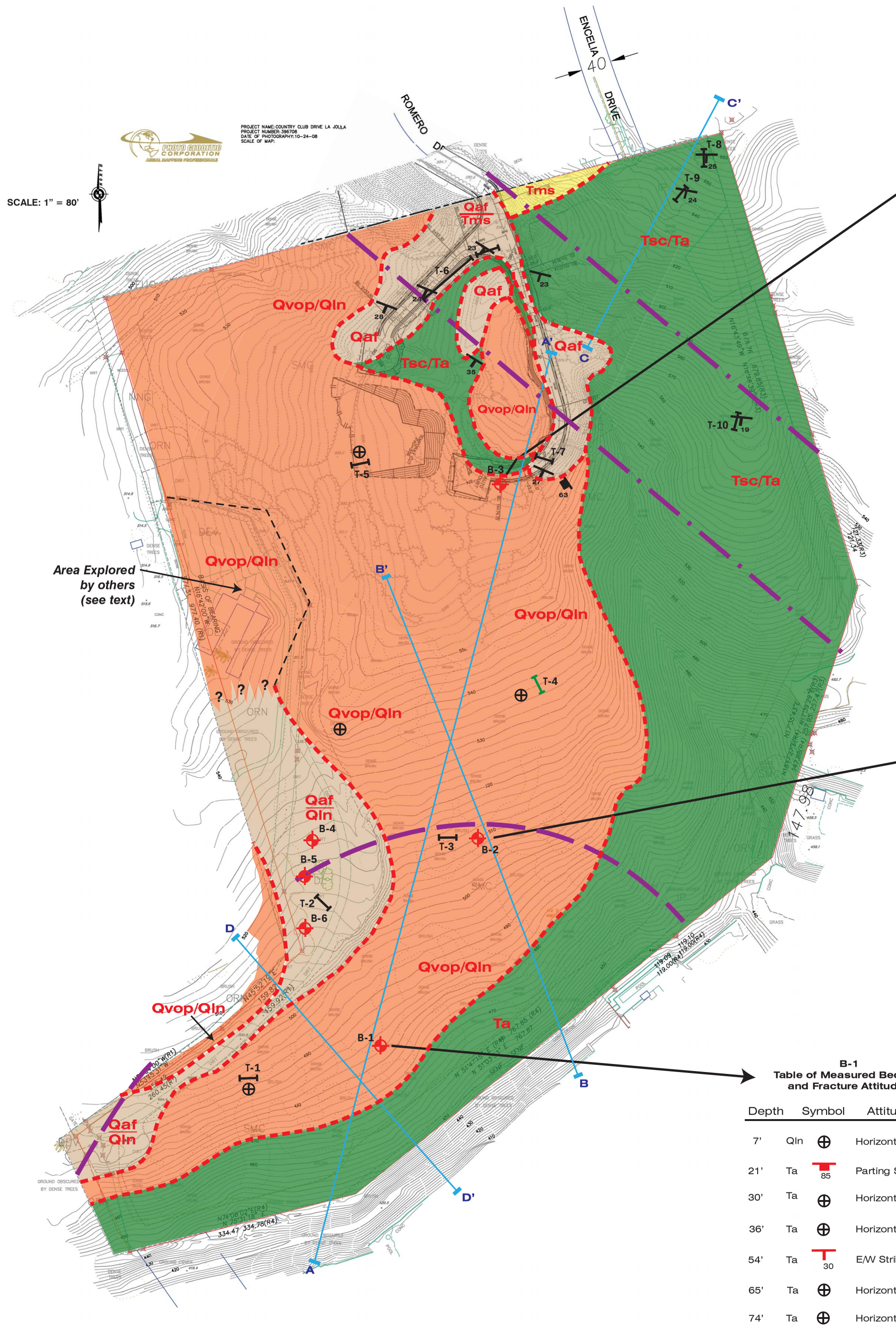
Geotechnical Exploration, Inc

(October 2016)

REFERENCE: This Plot Plan is not to be used for legal purposes. Locationss and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

REFERENCE: This Plot Plan was prepared from an existing SITE PLAN by COSTON ARCHITECTS INC. dated 10/11/2016 and from on-site field reconnaissance performed by GEI.

10-9977-1-P7.ai



B-3
Table of Measured Bedding and Fracture Attitudes

Depth	Symbol	Attitude/Feature
4'3"	Qln 8	N/S Strike 80° W Gravel Lag S80W Vertical Clay-filled Joint
4'3"	Tsc/Ta 9	N20W9°S Mineralized Band
16'3"	Tsc/Ta 4	S80W4°S N Color Band
20'6"	Tsc/Ta 23	N30W23°S Iron Mineralization/clay Module Bed
25'8"	Tsc/Ta ⊕	Horizontal Color Band
26'2"	Tsc/Ta ⊕	Horizontal Color Band
29'	Tsc/Ta 28	N60W23°S 1/2" Thick Sand-filled Fracture
29'9"	Tsc/Ta ⊕	Horizontal Color Band
32'	Tsc/Ta 28	N60W28°S Concentric Color Band
36'9"	Tsc/Ta 23	N55W23°S Band Laminae
41'4"	Tsc/Ta 10	N60E10°N Iron Module Bed
44'5"	Tsc/Ta 25	N10W25°N Iron Mineralized Bed
47'3"	Tsc/Ta 24	E/W Strike 24°S Concentrically Banded Sands
51'1"	Tsc/Ta 24	N60W24°S Sand Bed
67'2"	Tsc/Ta 20	E/W Strike 20° S Sand Bed
69'5"	Tsc/Ta 30	E/W Strike 30° S Sand Laminae
76'2"	Tsc/Ta 25	E/W Strike 25° S Clay Bed
76'-86'	Tsc/Ta 23	E/W Strike 23° S Clay and Sand Beds

B-2
Table of Measured Bedding and Fracture Attitudes

Depth	Symbol	Attitude/Feature
11'	Qln 10	E/W Strike 10° S Gravel Lag
15'5"	Ta 7	E/W Strike 7°S Color Band
19'3"	Ta 80	N25E80°N Mineralized Joint Surface
21'	Ta 10	N80W10°S Color Bands
23'4"	Ta 10	N80W10°S Color Band
29'	Ta 18	N72W18°S Color Band
31'	Ta 10	N80W10°S Color Band
35'	Ta 5	E/W Strike 5°S Color Band
37'2"	Ta 10	N80W10°S Color Band
41'4"	Ta 10	N80W10°S Color Band

B-1
Table of Measured Bedding and Fracture Attitudes

Depth	Symbol	Attitude/Feature
7'	Qln ⊕	Horizontal
21'	Ta 85	Parting Surface E/W Strike 85°S
30'	Ta ⊕	Horizontal Color Band
36'	Ta ⊕	Horizontal Color Band
54'	Ta 30	E/W Strike 30°S
65'	Ta ⊕	Horizontal Color Band
74'	Ta ⊕	Horizontal Color Band

LEGEND

T-1	Exploratory Trench Location
B-3	Exploratory Boring Location
	Approximate Geologic Contact
Qaf	Artificial Fill
Qvop/Qln	Qvop = Very Old Paralic Deposits/ Qln = Quaternary Lindavista Formation
Tsc/Ta	Tertiary Scripps Formation/Ardath Shale Undifferentiated
Ta	Tertiary Ardath Formation
Tms	Tertiary Mount Soledad Formation
65	Strike and Dip Joints
24	Strike and Dip of Bedding
	Vertical/Subvertical Joint
⊕	Horizontal/Subhorizontal Bedding
D D'	Line of Cross Section
	City of San Diego GeoHazard Zone 22
	City of San Diego GeoHazard Zone 12

(UPDATED) SITE PLAN AND GEOLOGIC MAP

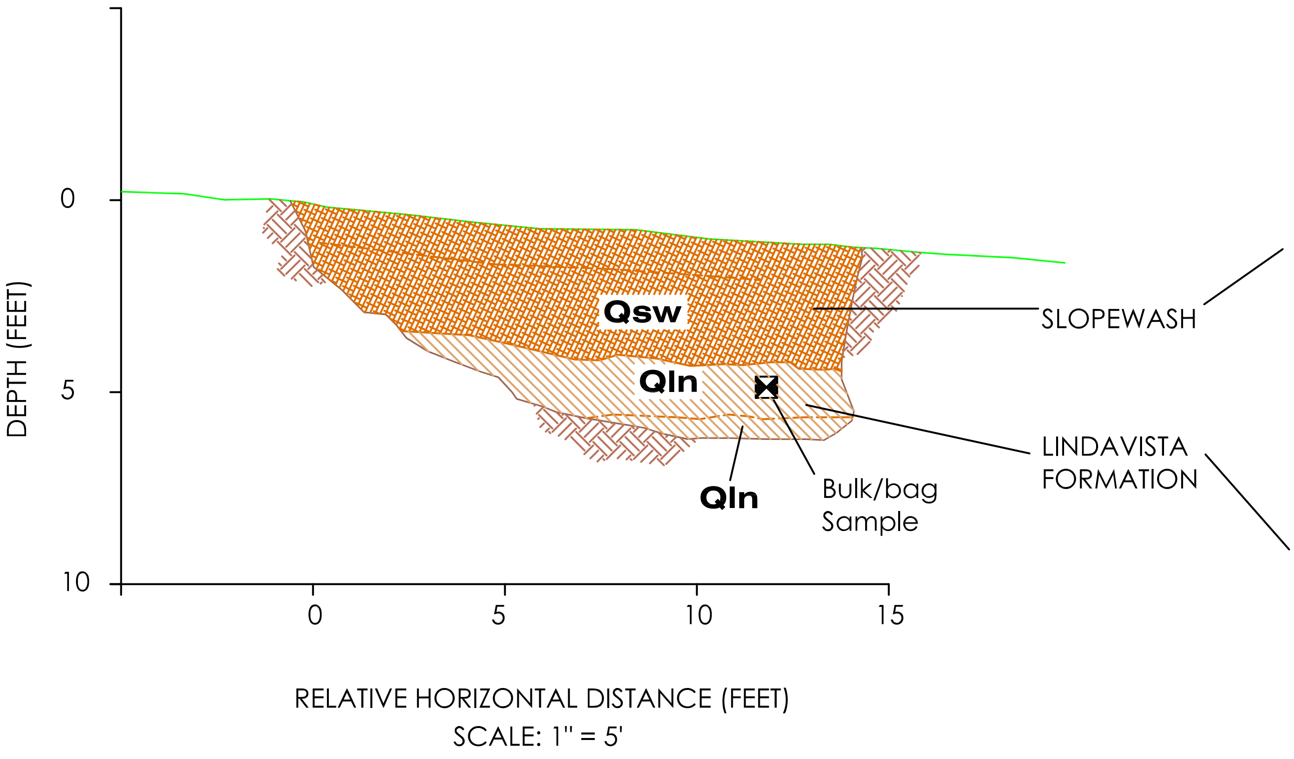
THE RESERVE RESIDENTIAL PROJECT
La Jolla, CA.
Figure No. IIb
Job No. 10-9977.1

Geotechnical Exploration, Inc.

June 2012
(updated October 2016)

EXPLORATORY TRENCH T-4

Copley Press Residential Project
Country Club Drive
La Jolla, CA.



SILTY SAND, fine-medium. Loose, dry, dark brown, abundant roots and organic materials >50% in upper 1foot from native vegetation becomes
SILTY SAND, fine-medium. Loose-medium dense, pale brown with iron pisoliths. mildly cemented in lower 1'-1.5' (SM)

CLAYEY SAND, fine-medium. Dense-very dense, moist, strong brown and gray with gray clay lenses grades into well cemented sandstone lower 1 foot (SC/SM)

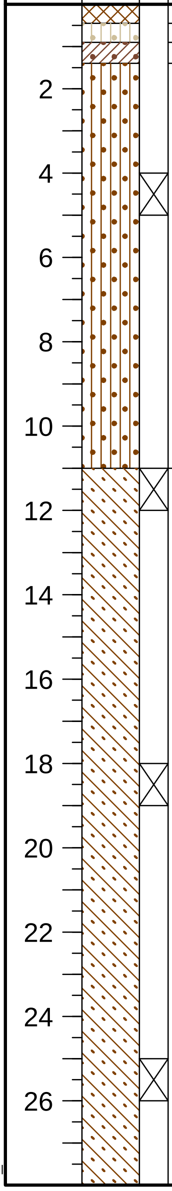
Figure No. IIIi
Job No. 10-9977.1

Geotechnical Exploration, Inc.

October 2011

10-9977-T-4

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Truck-mounted Bucket/Auger Drill Rig	**30-inch diameter boring**	**9-6-11**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 509' Mean Sea Level	**Not Encountered**	**DV/SO/LDR**



FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
SILTY SAND, fine- to medium-grained. Medium dense. Damp to moist. Brown. **FILL (Qaf)**	SM									
SILTY SAND, fine- to medium-grained. Loose. Dry. Pale brown. **SLOPEWASH (Qsw)**	SM	3.5						8		
CLAYEY SAND, fine- to medium-grained. Dense. Damp. Brown. **WEATHERED LINDAVISTA FORMATION (Qln)**	SC									
SILTY SAND, fine- to medium-grained. Medium dense to dense. Damp to moist. Pale orange to strong brown. **LINDAVISTA FORMATION (Qln)** -- 19% passing #200 sieve. -- gravel in spoil. @ 10'10"-11'7" -- 9" thick lag gravel layer in Qln, E-W strike, 10°S dip in silty sand matrix; cobble to 8" in diameter.	SM									
SILTSTONE/ MUDSTONE Hard. Damp to moist. Gray to light brown with dark mineral coating on parting surfaces. **ARDATH SHALE FORMATION (Ta)** . @ 15'5" -- north sidewall, 1-1/2" thick dark color band, E-W strike, 7°S dip. @ 19'3" -- mineralized joint surface, N25E, 80°N. @ 21' -- north sidewall, 4" thick distinct color bands (1-1/2" gray clay, not remolded; 1-1/2" buff to reddish brown silt; 1" gray clay, not remolded); N80°W, 10°SW. @ 23'4" -- 2" dark gray clay, not remolded; 1" light gray clay; 3/4" tan silty clay; N80°W, 10°SW. -- 90% passing #200 sieve.	ML-CL	5.0								

- PERCHED WATER TABLE
- ☒ LOOSE BAG SAMPLE
- 1 IN-PLACE SAMPLE
- ■ MODIFIED CALIFORNIA SAMPLE
- S FIELD DENSITY TEST
- ▨ STANDARD PENETRATION TEST

JOB NAME
The Reserve LLC

SITE LOCATION
7007 Country Club Drive, La Jolla, CA

JOB NUMBER
10-9977.1

REVIEWED BY
LDR/JAC

Geotechnical Exploration, Inc.

LOG No.
B-2

FIGURE NUMBER
IIIa

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Truck-mounted Bucket/Auger Drill Rig	**30-inch diameter boring**	**9-6-11**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 509' Mean Sea Level	**Not Encountered**	**DV/SO/LDR**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
30			**SILTSTONE/ MUDSTONE** Hard. Damp to moist. Gray to light brown with dark mineral coating on parting surfaces.	ML-CL									
32			**ARDATH SHALE FORMATION (Ta)** @ 29' -- 1" thick red-brown color band; N72°W, 18°SW.										
34			@ 31' -- 1" thick dark gray color band; N80°W, 10°SW and high angle 1/32-1/64" mineralized parting surface.										
36		1	@ 35' -- 3/4" thick reddish brown color band; E-W strike, 5°S.		18.7	112.9							
38			@ 37'2" -- 1" thick tan color band, scattered iron concretions; N80°W, 10°SW.										
40		☒	@ 40' -- slabs/"chips" coming up; fissility. -- 98% passing #200 sieve.		7.1								
42			@ 41'4" -- 4" thick red-brown color band with 1/8" diameter iron concretions scattered over color band; N80°W, 10°SW.										
44													
46													
48													
50													
52		☒											
54													

▼ PERCHED WATER TABLE

☒ LOOSE BAG SAMPLE

1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

S FIELD DENSITY TEST

▨ STANDARD PENETRATION TEST

JOB NAME
The Reserve LLC

SITE LOCATION
7007 Country Club Drive, La Jolla, CA

JOB NUMBER
10-9977.1

FIGURE NUMBER
IIIb

REVIEWED BY
LDR/JAC

Geotechnical Exploration, Inc.

LOG No.
B-2

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Truck-mounted Bucket/Auger Drill Rig	**30-inch diameter boring**	**9-6-11**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 509' Mean Sea Level	**Not Encountered**	**DV/SO/LDR**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION		IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
			DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.									
	▨		**SILTSTONE/ MUDSTONE** Hard. Damp to moist. Gray to light brown with dark mineral coating on parting surfaces.	ML-CL									
58	▨												
	▨		**ARDATH SHALE FORMATION (Ta)**										
60	▨												
62	▨	☒	-- 95% passing #200 sieve.		6.1								
64	▨												
66	▨												
68	▨												
70	▨	☒											
72	▨												
74	▨												
76	▨												
78	▨												
80	▨	☒											
			No seeps. No caving.										
82			Bottom @ 80'										

▼ PERCHED WATER TABLE

☒ LOOSE BAG SAMPLE

1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

S FIELD DENSITY TEST

▨ STANDARD PENETRATION TEST

JOB NAME
The Reserve LLC

SITE LOCATION
7007 Country Club Drive, La Jolla, CA

JOB NUMBER
10-9977.1

FIGURE NUMBER
IIIc

REVIEWED BY
LDR/JAC

Geotechnical Exploration, Inc.

LOG No.
B-2

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Truck-mounted Bucket/Auger Drill Rig	**30-inch diameter boring**	**9-7-11**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 575' Mean Sea Level	**Not Encountered**	**DV/SO/LDR**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION		IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
			DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.									
			SILTY SAND, fine- to medium-grained, with fractured cobbles. Loose. Dry to damp. Brown.	SM									
2			**SLOPEWASH (Qsw)** **CLAYEY SAND**, fine- to medium-grained. Medium dense. Moist. Orange- and red-brown.	SC									
4		☒	**LINDAVISTA FORMATION (Qln)** **SILTY SAND**, fine-grained. Medium dense. Moist. Orange- and gray-brown.	SM CL							70		
6			**LINDAVISTA FORMATION (Qln)** **COBBLE LAYER**, 4" thick at contact; N-S strike, 8°W.										
8		☒	**LINDAVISTA FORMATION (Qln)** **SILTY CLAY/ MUDSTONE** Firm to hard. Moist. Gray-brown.		7.7								
10			**SCRIPPS FORMATION/ ARDATH SHALE UNDIFFERENTIATED (Tsc/ Ta)**										
12			@ 4.25' -- steel gray clay-filled joint, S80°W strike, vertical dip; no shearing or fracturing with central hairline-healed fracture. @ 4.5' -- mineralized band; N20°W, 9°SW.										
14			-- 98% passing #200 sieve.										
16			@ 15'4" -- horizontal color band with iron manganese mineralization. @ 16'3" -- light color banding; S80°W, 4°SE. @ 17' -- becomes more blocky with mineralized parting surfaces.										
18		1 ☒	-- 98% passing #200 sieve.		18.2 18.1	108.2							

▼ PERCHED WATER TABLE

☒ LOOSE BAG SAMPLE

1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

S FIELD DENSITY TEST

▨ STANDARD PENETRATION TEST

JOB NAME
The Reserve LLC

SITE LOCATION
7007 Country Club Drive, La Jolla, CA

JOB NUMBER	REVIEWED BY	LOG No.
10-9977.1	**LDR/JAC**	**B-3**
FIGURE NUMBER	Geotechnical Exploration, Inc.	
IIId		

EQUIPMENT **Truck-mounted Bucket/Auger Drill Rig**	DIMENSION & TYPE OF EXCAVATION **30-inch diameter boring**	DATE LOGGED **9-7-11**
SURFACE ELEVATION **± 575' Mean Sea Level**	GROUNDWATER/ SEEPAGE DEPTH **Not Encountered**	LOGGED BY **DV/SO/LDR**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION		IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
			DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.									
	▨		**SILTY CLAY/ MUDSTONE** Firm to hard. Moist. Gray-brown.	CL									
22			**SCRIPPS FORMATION/ ARDATH SHALE UNDIFFERENTIATED (Tsc/ Ta)**										
24		2	@ 20'6" -- N30°W, 23°SW; iron mineralization with clay nodules up to 1/2" thick, no remolding, no fracturing (not Qls related).										
26			@ 25'8" -- 1" thick dark color band, horizontal. @ 26'2" -- 2" thick light color band, horizontal. @ 27' -- brown/orange iron oxide staining.										
28													
30		☒	@ 29' -- N60°W, 28°SW; 1/2" thick sand-filled fracture, gray-tan above and below iron staining on top and bottom. @ 29'9" -- 1-1/2" thick dark color band, horizontal.										
32			@ 32' -- 15"x 1"x 5" thick color zone, reddish tan sand with concentric banding; N60°W.										
34		3			20.7	108.6							
36													
38			@ 36'9" -- low side, very dark brown to black "augen" shaped, N55°W, 23°SW, banded 1/16"- 1/8" laminae, rhythmically interbedded with gray clay beds.										
		☒											

▼	PERCHED WATER TABLE
☒	LOOSE BAG SAMPLE
1	IN-PLACE SAMPLE
■	MODIFIED CALIFORNIA SAMPLE
S	FIELD DENSITY TEST
▨	STANDARD PENETRATION TEST

JOB NAME
The Reserve LLC

SITE LOCATION
7007 Country Club Drive, La Jolla, CA

JOB NUMBER
10-9977.1

REVIEWED BY
LDR/JAC

Geotechnical Exploration, Inc.

LOG No.
B-3

FIGURE NUMBER
IIIe

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Truck-mounted Bucket/Auger Drill Rig	**30-inch diameter boring**	**9-7-11**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 575' Mean Sea Level	**Not Encountered**	**DV/SO/LDR**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
42 44 46	▨		**SILTY CLAY/ MUDSTONE** Firm to hard. Moist. Gray-brown. **SCRIPPS FORMATION/ ARDATH SHALE UNDIFFERENTIATED (Tsc/ Ta)** @ 41'4" -- 4" thick light color band, horizontal; N60°E, 10°NW, reddish-brown accumulation of iron nodules to 1/2" in diameter. @ 44'5" -- iron mineralized zone 3/4" thick, N10°W, 25°NE. @ 47'3" -- thick fine- to medium-grained tan sand over 1/4" thick gray silty clay over 5"	CL									
48 50	⁙	4 ☒	medium-grained tan sand over 10" thick black medium-grained sand "augen" bottoms on clean contact with gray clay; E-W strike, 24°S dip. Entire east side of boring transitions to concentrically banded (laminae) black sand "augen" with vertical light sand stringers penetrating through "augen" extends into.	SM									
52 54 56 58	▨	☒	**SILTY SAND**, fine- to medium-grained. Dense. Moist. Dark brown (manganese stained). **SCRIPPS FORMATION/ ARDATH SHALE UNDIFFERENTIATED (Tsc/ Ta)** @ 51'1" -- 3" thick light tan sand bed; N60°W, 24°SW, over 7" gray silty clay (same attitude) over 4" thick laminated tan sand. @ 52'10" -- 4" thick channel filling light tan sand, thickens to 12 " with 2" relief; channel includes angular gray clay gravel-size clasts; sand is thinly bedded. From 52'10" to 56'6" -- Rhythmically	CL									

▼ PERCHED WATER TABLE

☒ LOOSE BAG SAMPLE

1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

S FIELD DENSITY TEST

▨ STANDARD PENETRATION TEST

JOB NAME		
The Reserve LLC		
SITE LOCATION		
7007 Country Club Drive, La Jolla, CA		
JOB NUMBER **10-9977.1**	REVIEWED BY **LDR/JAC**	LOG No. **B-3**
FIGURE NUMBER **IIIf**	**Geotechnical Exploration, Inc.**	

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Truck-mounted Bucket/Auger Drill Rig	**30-inch diameter boring**	**9-7-11**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 575' Mean Sea Level	**Not Encountered**	**DV/SO/LDR**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION		IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
			DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.									
62 64		5 ☒	bedded 1'- 2" thick gray clay and tan sand beds with iron mineral accumulations on high-angle joints which cross through clay/ sand contacts without offset; tabular (flat) and irregular erosional contacts. From 57'4" to 67' -- uniform **SILTY CLAY**. Hard. Damp to moist. Tan and gray. **SILTY CLAY/ MUDSTONE** Firm to hard. Moist. Gray-brown.	CL	16.9								
66			**SCRIPPS FORMATION/ ARDATH SHALE UNDIFFERENTIATED (Tsc/ Ta)** -- 95% passing #200 sieve.										
68			@ 67'2" -- 2" thick, tan, fine- to medium-grained sand; E-W strike, 20°S dip.										
70		6 ☒	@ 69'5" to 69'11" -- very fine-grained tan sand over 12" thick black concentrically banded black sand laminae "augen" sand lens; E-W strike, 30°S dip, over **SILTY CLAY**.										
72		7	@ 72'- 74' -- manganese staining layer.										
74													
76		8 ☒	@ 76' -- 2"- 3" thick gray clay, E-W strike, 25°S dip. From 76' to 78' -- dark gray-brown **CLAY**, remolded.		21.9	98.6							
78		9	From 76'- 86' -- Gray **CLAY**, with iron stained **SAND**, E-W strike, 23°S dip.										

▼ PERCHED WATER TABLE

☒ LOOSE BAG SAMPLE

1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

S FIELD DENSITY TEST

▨ STANDARD PENETRATION TEST

JOB NAME
The Reserve LLC

SITE LOCATION
7007 Country Club Drive, La Jolla, CA

JOB NUMBER
10-9977.1

REVIEWED BY
LDR/JAC

Geotechnical Exploration, Inc.

FIGURE NUMBER
IIIg

LOG No.
B-3

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Truck-mounted Bucket/Auger Drill Rig	**30-inch diameter boring**	**9-7-11**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 575' Mean Sea Level	**Not Encountered**	**DV/SO/LDR**

DEPTH (feet)	SYMBOL	SAMPLE	FIELD DESCRIPTION AND CLASSIFICATION DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.	IN-PLACE MOISTURE (%)	IN-PLACE DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)	DENSITY (% of M.D.D.)	EXPAN. + CONSOL. - (%)	EXPANSION INDEX	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
	▨		**SILTY CLAY/ MUDSTONE** Firm to hard. Moist. Gray-brown. **SCRIPPS FORMATION/ ARDATH SHALE UNDIFFERENTIATED (Tsc/ Ta)**	CL									
82	▨	☒	-- 94% passing #200 sieve.		4.4								
84	▨												
	▨	10			14.7	119.9							
86			Bottom @ 86'										
88													
90													
92													
94													
96													
98													

▼	PERCHED WATER TABLE
☒	LOOSE BAG SAMPLE
1	IN-PLACE SAMPLE
■	MODIFIED CALIFORNIA SAMPLE
S	FIELD DENSITY TEST
▨	STANDARD PENETRATION TEST

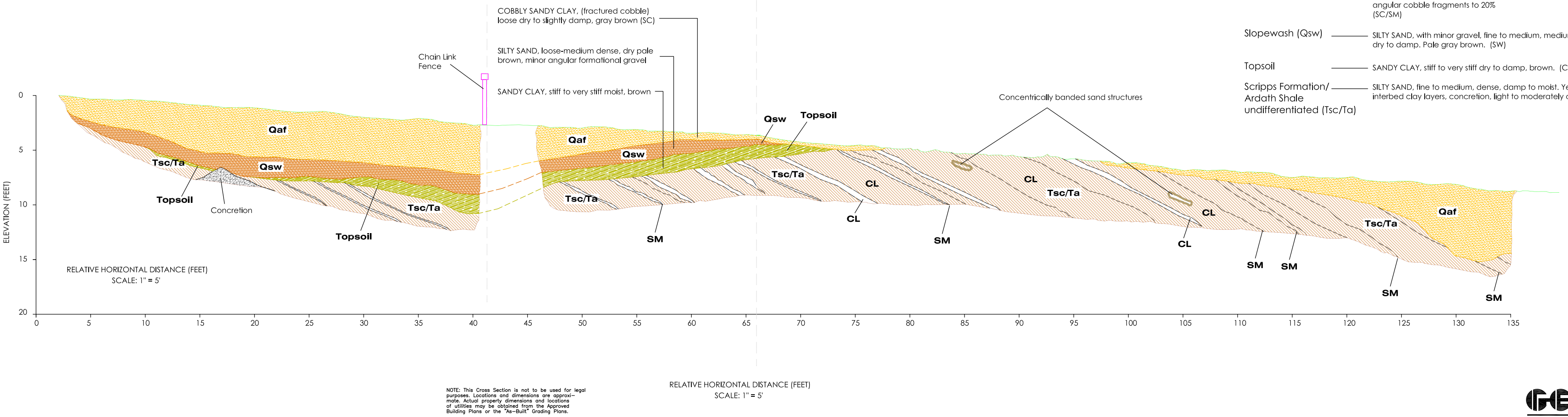
JOB NAME
The Reserve LLC

SITE LOCATION
7007 Country Club Drive, La Jolla, CA

JOB NUMBER	REVIEWED BY	LOG No.
10-9977.1	**LDR/JAC**	**B-3**
FIGURE NUMBER	Geotechnical Exploration, Inc.	
IIIh		

EXPLORATORY TRENCH T-6

The Reserve
Country Club Drive
La Jolla, CA.



Artificial Fill (Qaf)	CLAYEY SAND/SILTY SAND, fine-medium, loose to medium dense dry to slightly dry, yellow and gray brown, fragments of formation angular cobble fragments to 20% (SC/SM)
Slopewash (Qsw)	SILTY SAND, with minor gravel, fine to medium, medium dense, dry to damp. Pale gray brown. (SW)
Topsoil	SANDY CLAY, stiff to very stiff dry to damp, brown. (CL)
Scripps Formation/ Ardath Shale undifferentiated (Tsc/Ta)	SILTY SAND, fine to medium, dense, damp to moist. Yellow brown interbed clay layers, concretion, light to moderately cemented. (SM)

NOTE: This Cross Section is not to be used for legal purposes. Locations and dimensions are approximate. Actual property dimensions and locations of utilities may be obtained from the Approved Building Plans or the "As-Built" Grading Plans.

Figure No. IIIj
Job No. 10-9977.1

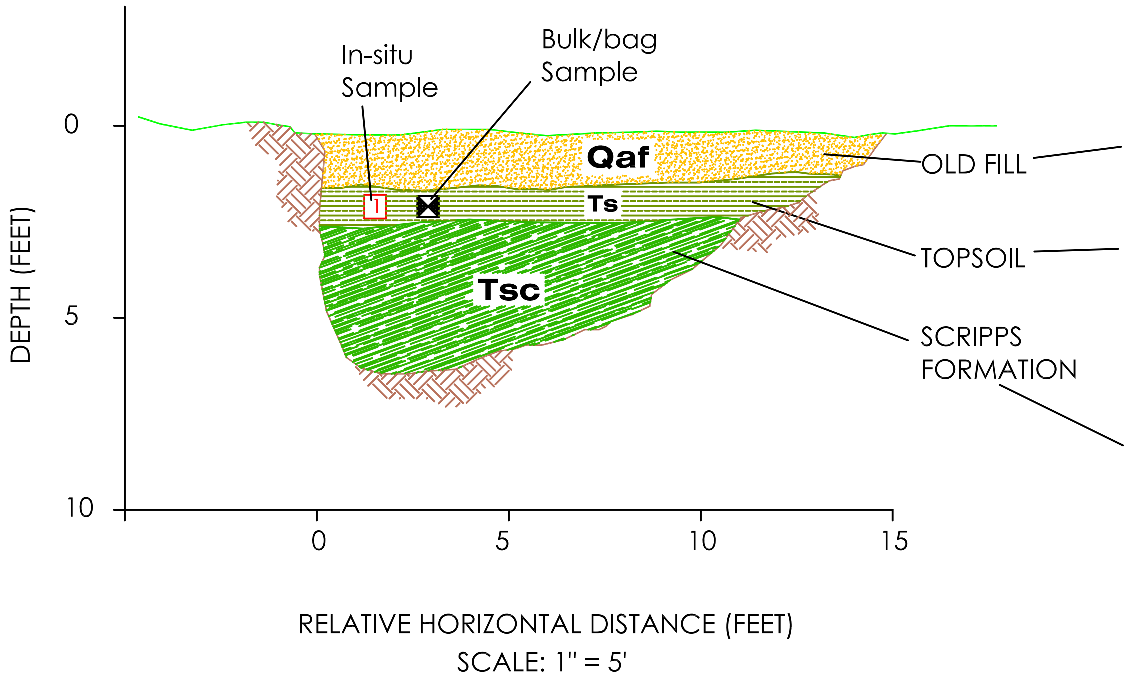
Geotechnical Exploration, Inc.

October 2011

10-9977-T-6

EXPLORATORY TRENCH T-7

Copley Press Residential Project
Country Club Drive
La Jolla, CA.



SANDY CLAY/CLAYEY SAND Soft/loose-firm/medium dense, damp, gray brown (CL/SC)

SANDY CLAY Firm-stiff, moist brown (CL)

SANDY CLAY (shale) Firm-stiff, moist,tan and strong brown bedding N66-7OW 27-32°SW High angle fractures. Joints @ 1"-3" spacing N50W 63°SW and N25W 76°SW

Figure No. IIIk
Job No. 10-9977.1

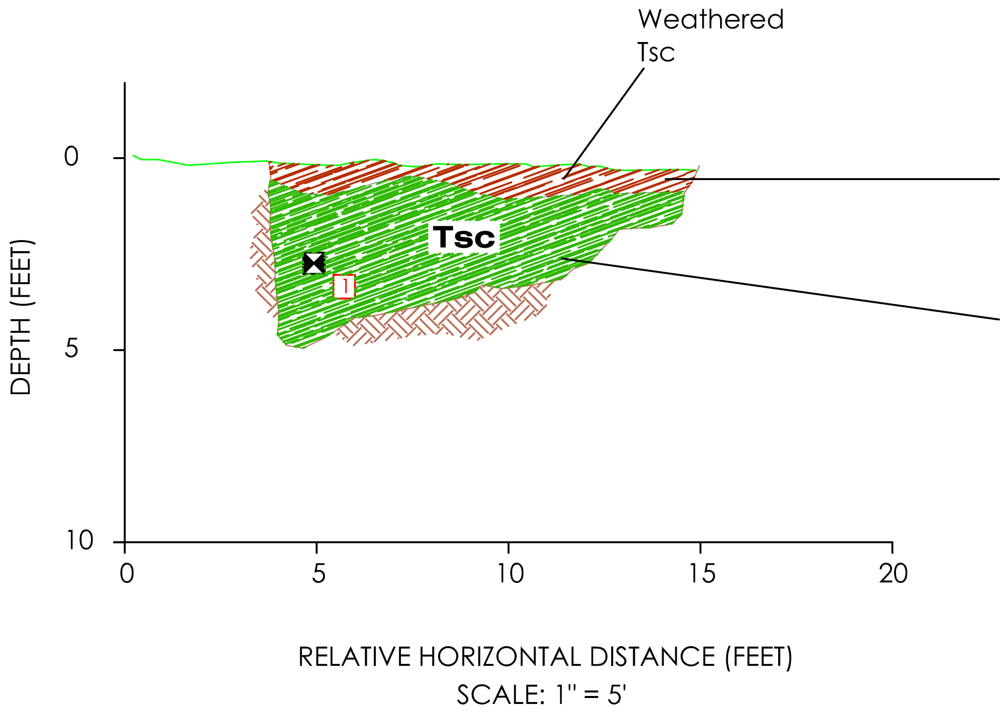
Geotechnical Exploration, Inc.

October 2011

10-9977-T-7

EXPLORATORY TRENCH T-10

The Reserve
Country Club Drive
La Jolla, CA.



WEATHERED SCRIPPS FORMATION
SANDY CLAY. Firm, damp, gray brown,
highly fractured.
(SC) N83E 19°S
@ 8"-28" grades into

SCRIPPS FORMATION
SILTY SAND/SAND SILT (SILTSTONE)
Medium dense-sense/stiff-hard,
damp-moist. Tan-brown and red brown
occassional intraformational rounded
gravel-cobble clasts.
concretion @ 4' 1' x 2'
JOINTS 1'-3' spacing subvertical S10W
N30W 60°NE N15W 84° NE

Figure No. IIIL
Job No. 10-9977.1

Geotechnical Exploration, Inc.

October 2011

10-9977-T-10

Sheet 1 of 1

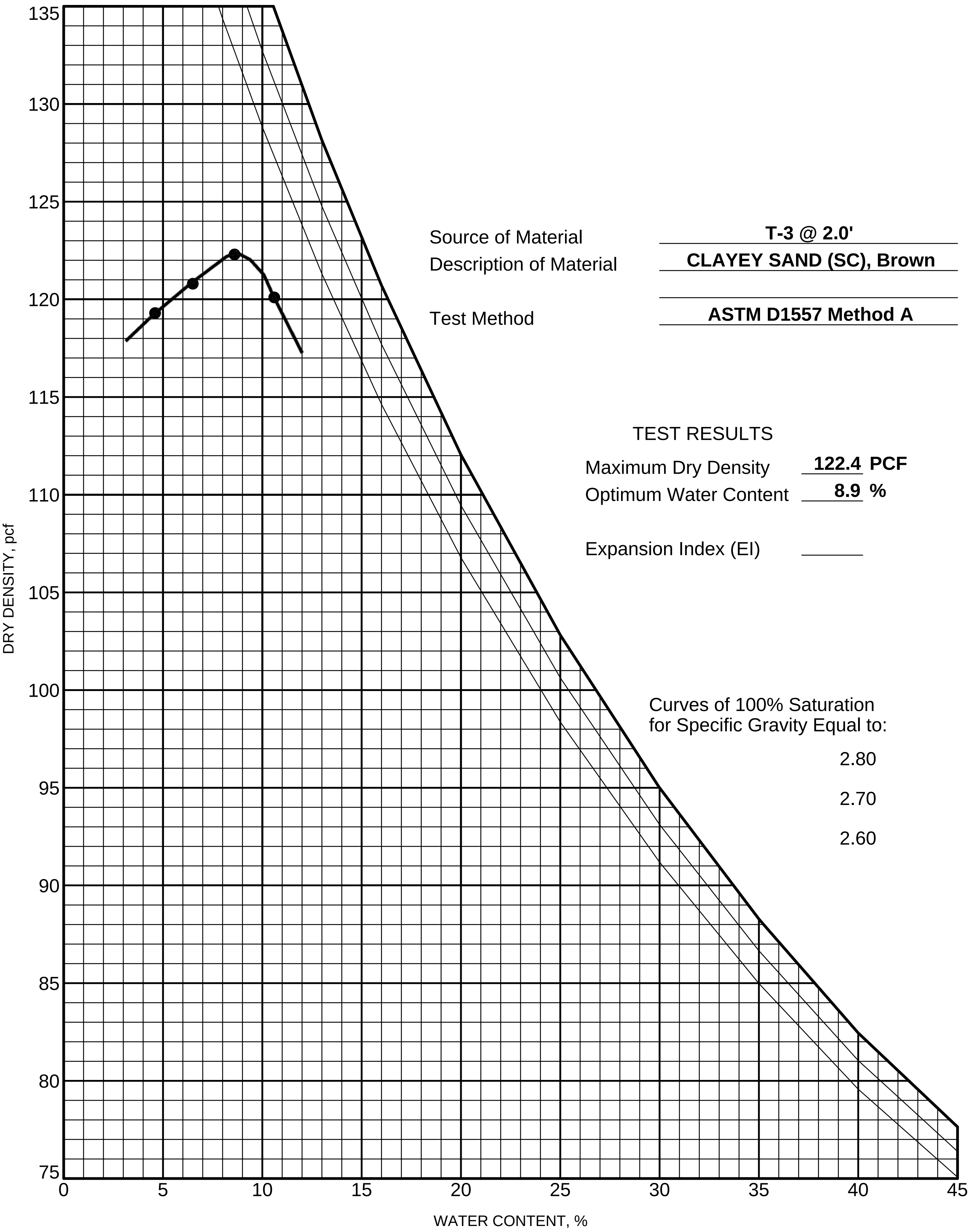
Excavation	Depth	Liquid Limit	Plastic Limit	Plasticity Index	#200 Sieve Size (mm)	%<#200 Sieve	Class-ification	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
B-1	2.0							11.2	113.4		
B-1	5.0				0.075	40		4.3			
B-1	18.0	63	22	41	0.075	98	CH				
B-1	42.0							5.6			
B-1	48.0							18.8	111.5		
B-1	54.0	57	23	34	0.075	96	CH				
B-1	63.0							16.7	115.5		
B-1	68.0							18.3	115.9		
B-1	69.0				0.075	94		18.3			
B-2	4.0							3.5			
B-2	25.0							5.0			
B-2	35.0							18.7	112.9		
B-2	40.0	55	25	30				7.1			
B-2	61.0				0.075	95		6.1			
B-2	79.0	58	24	34							
B-3	8.0							7.7			
B-3	18.0	51	25	26	0.075	98	CH	18.2	108.2		
B-3	18.5							18.1			
B-3	34.0							20.7	108.6		
B-3	60.0	54	24	30	0.075	95	CH				
B-3	60.5							16.9			
B-3	76.0	70	28	42				21.9	98.6		
B-3	82.0				0.075	94		4.4			
B-3	85.0							14.7	119.9		
T-1	1.5				0.075	16					
T-1	4.5							4.8	118.5		
T-1	5.0				0.075	8					
T-10	4.0							15.3	114.5		
T-2	8.0							15.5	118.5		
T-2	9.5							8.2	118.3		
T-3	2.0				0.075	16					
T-4	4.5				0.075	27		5.8	121.5		
T-5	1.5				0.075	20					
T-5	3.0							4.2	120.2		
T-5	5.0							5.6	120.9		
T-7	2.0				0.075	99					
T-7	2.5							19.6	109.3		
T-8	1.0							12.7	106.2		
T-8	1.5				0.075	49					
T-8	3.0				0.075	44					
T-9	4.0							12.7	116.9		

Geotechnical Exploration, Inc.

Summary of Laboratory Results

Figure Number: IVa
Job Name: The Reserve LLC
Site Location: 7007 Country Club Drive, La Jolla, CA
Job Number: 10-9977.1

US LAB SUMMARY 9977 COPLEY.GPJ GEO EXPL.GDT 11/15/11



Source of Material **T-3 @ 2.0'**
Description of Material **CLAYEY SAND (SC), Brown**

Test Method **ASTM D1557 Method A**

TEST RESULTS

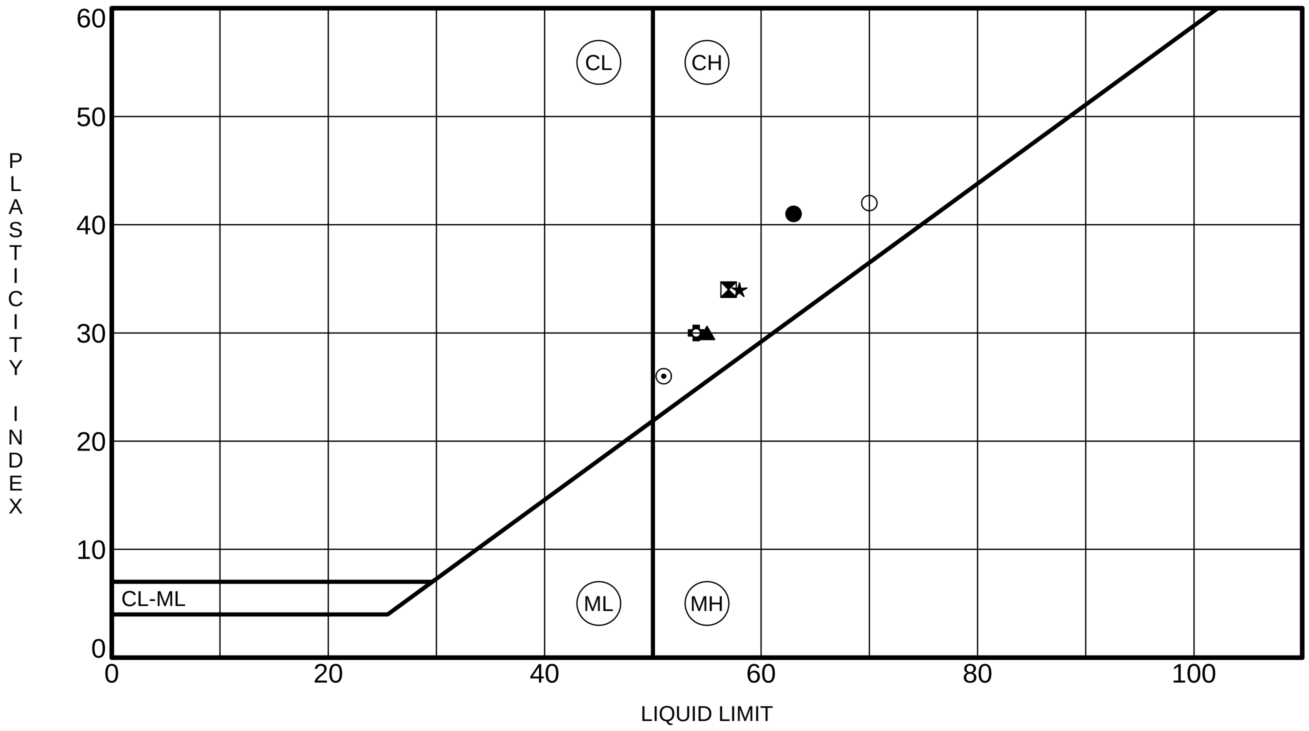
Maximum Dry Density **122.4 PCF**
Optimum Water Content **8.9 %**

Expansion Index (EI) ______

Geotechnical Exploration, Inc.

MOISTURE-DENSITY RELATIONSHIP

Figure Number: IVb
Job Name: The Reserve LLC
Site Location: 7007 Country Club Drive, La Jolla, CA
Job Number: 10-9977.1



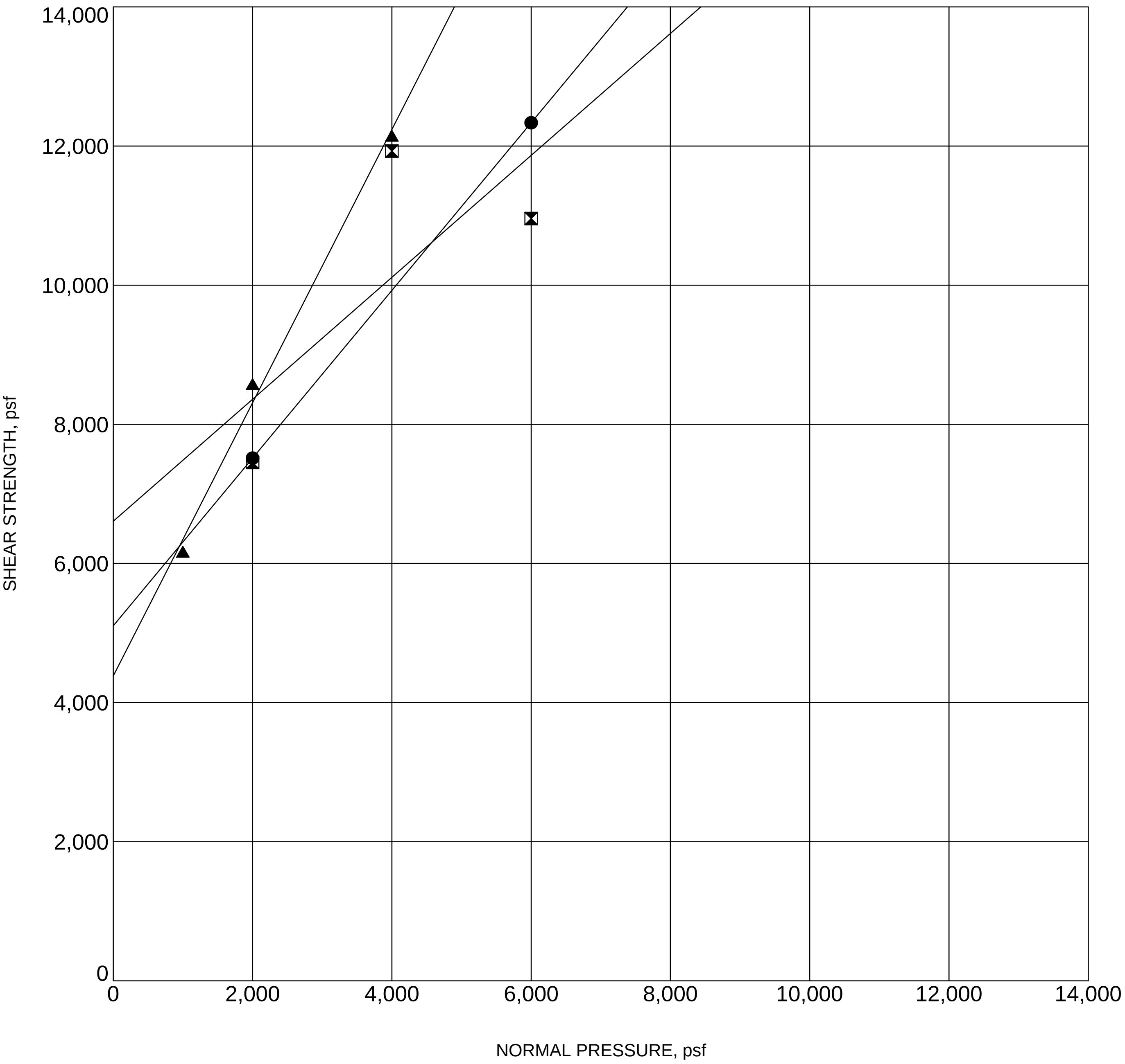
	Specimen Identification	LL	PL	PI	Fines	Classification
●	**B-1 @ 18.0'**	**63**	**22**	**41**	**98**	**CLAY (CH), Yellowish Brown**
⧗	**B-1 @ 54.0'**	**57**	**23**	**34**	**96**	**CLAY (CH), Yellowish Brown**
▲	**B-2 @ 40.0'**	**55**	**25**	**30**		**CLAY (CH), Yellowish to Grayish Brown**
★	**B-2 @ 79.0'**	**58**	**24**	**34**		**CLAY (CH), Yellowish Brown**
⊙	**B-3 @ 18.0'**	**51**	**25**	**26**	**98**	**CLAY (CH), Yellowish Brown with Gray**
✤	**B-3 @ 60.0'**	**54**	**24**	**30**	**95**	**CLAY (CH), Yellowish Brown with Gray**
○	**B-3 @ 76.0'**	**70**	**28**	**42**		**CLAY (CH), Yellowish Brown with Gray**

Geotechnical Exploration, Inc.

ATTERBERG LIMITS' RESULTS

Figure Number: IVc
Job Name: The Reserve LLC
Site Location: 7007 Country Club Drive, La Jolla, CA
Job Number: 10-9977.1

US ATTERBERG LIMITS 9977 COPLEY.GPJ GEO EXPL.GDT 10/18/11



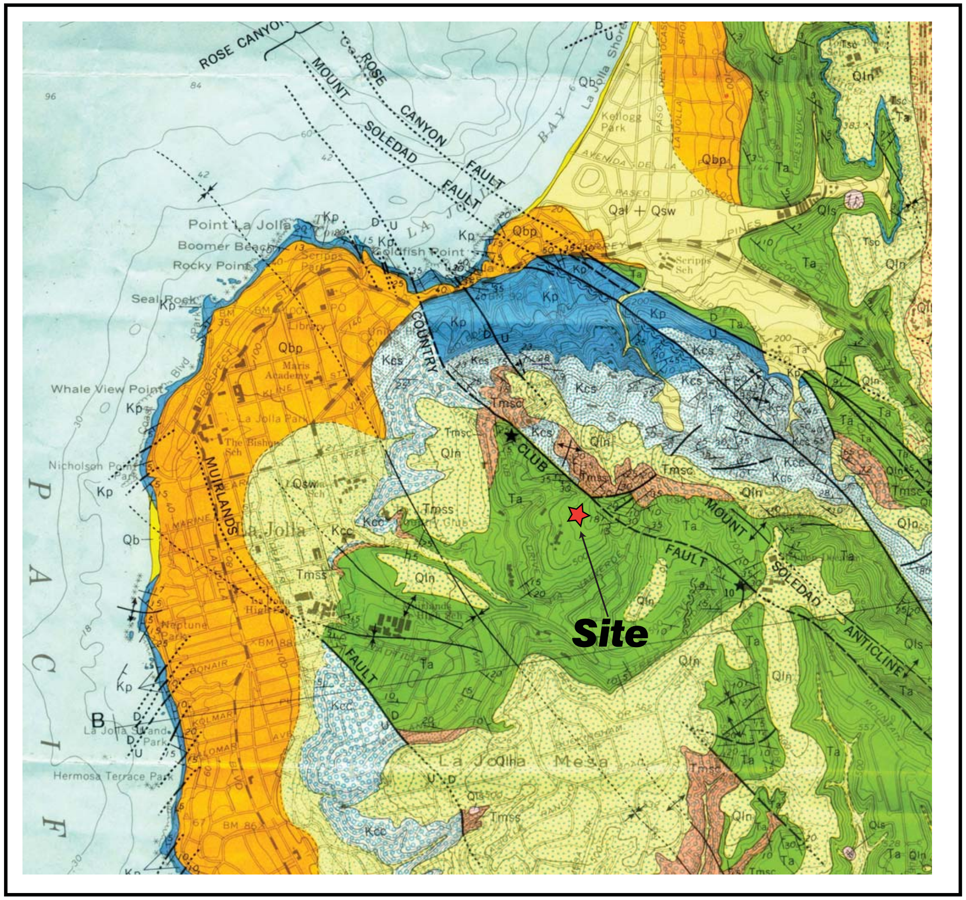
Specimen Identification		Classification	γ_d	MC%	c	ϕ
●	**B-1 @ 49.0'**	**CLAY (CH), Yellowish Brown**			**5103**	**50**
☒	**B-2 @ 25.0'**	**CLAY (CL), Yellowish Brown**			**6606**	**41**
▲	**B-3 @ 18.0'**	**CLAY (CH), Yellowish Brown with Gray**			**4383**	**63**

Geotechnical Exploration, Inc.

DIRECT SHEAR TEST

Figure Number: IVd
Job Name: The Reserve LLC
Site Location: 7007 Country Club Drive, La Jolla, CA
Job Number: 10-9977.1

GEOLOGY MAP
1975
by Michael P. Kennedy



The Reserve LLC
Country Club Drive
La Jolla, CA.

GEOLOGY OF THE LA JOLLA QUADRANGLE SAN DIEGO COUNTY, CALIFORNIA

by Michael P. Kennedy

1975

EXPLANATION

QUATERNARY

Holocene

Qaf
Artificial fill

Qb
Beach sand

Qal | Qsw | Qal + Qsw
Alluvium and Slopewash

Qal, Alluvium; Qsw, Slopewash; Qal & Qsw, Alluvium and Slopewash undifferentiated.

Qls
Landslide deposits

Pleistocene

Qt
Stream-terrace deposits

Qbp
Bay Point Formation

Qln | Qlb
Lindavista Formation

Qln, nearshore deposits; Qlb, beach deposits.

TERTIARY

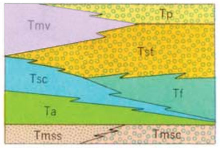
Pliocene

Tsd
San Diego Formation

Miocene

Tba
Basaltic andesite

Eocene



Poway and La Jolla Groups

Tp, Pomerado Conglomerate; Tmv, Mission Valley Formation; Tst, Stadium Conglomerate; Tf, Friars Formation; Tsc, Scripps Formation; Ta, Ardath Shale; Tmss, Mount Soledad Formation (sandstone part); Tmsc, Mount Soledad Formation (conglomerate part).

CRETACEOUS

Upper Cretaceous

Kcs | Kcc | Kp
Rosario Group

Kcs, Cabrillo Formation (sandstone part); Kcc, Cabrillo Formation (conglomerate part); Kp, Point Loma Formation.

SYMBOLS

Contact

(dashed where approximately located; dotted where concealed)

U D ↓75
Fault, showing dip

(dashed where approximately located; dotted where concealed; U, upthrown side; D, downthrown side; ≈≈ shear zone).

Anticline, showing direction of plunge.

Syncline, showing direction of plunge.

5
Strike and dip of bedding.

45
Strike and dip of joint.

Strike of vertical joint.

Qls
Landslide with direction of movement indicated by arrows.

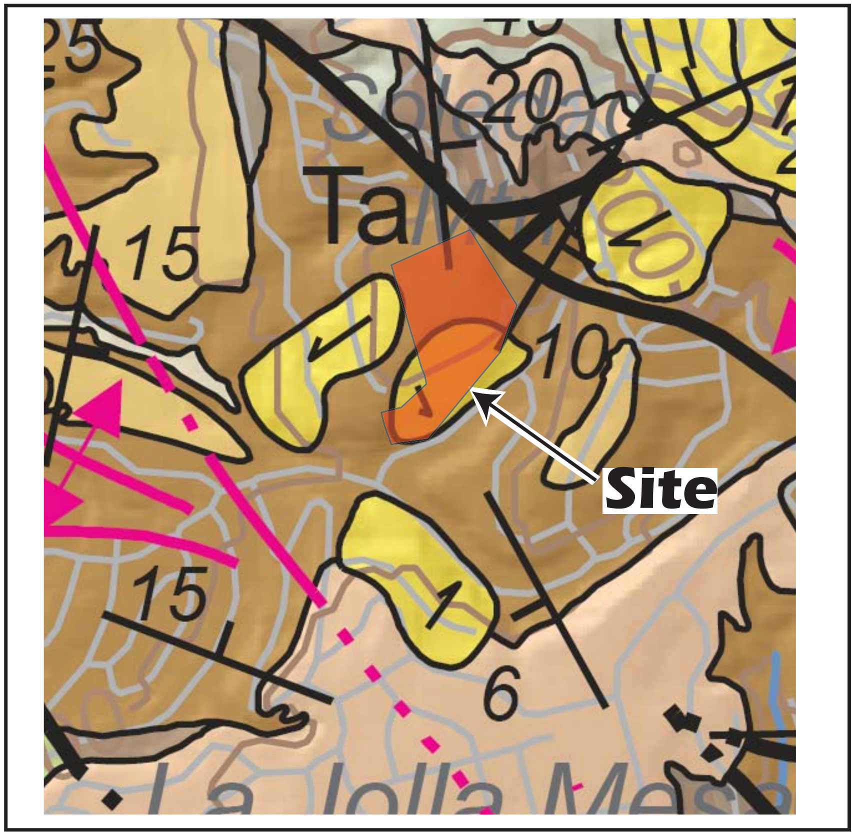
△ C-15-18
Clay sample locality.

⊕ 3
Fossil mollusk locality.

★ 10
Fossil coccolith locality.

• 3
Fossil vertebrate locality.

Figure No. V
Job No. 10-9977.1



The Reserve LLC
Country Club Drive
La Jolla, CA.

Base Map

Onshore base (hypsography, hydrography, and transportation) from U.S.G.S. digital line graph (DLG) data, San Diego 30' x 60' metric quadrangle. Shaded topographic base from U.S.G.S. digital elevation models (DEM's). Offshore bathymetric contours and shaded bathymetry from N.O.A.A. single and multibeam data. Projection is UTM, zone 11, North American Datum 1927.

This map was funded in part by the U.S. Geological Survey National Cooperative Geologic Mapping Program, STATEMAP Award no. 98HQAG2049.

Prepared in cooperation with the U.S. Geological Survey, Southern California Areal Mapping Project.

The-Reserve-combo-2008-geo.ai

EXCERPT FROM GEOLOGIC MAP OF THE SAN DIEGO 30' x 60' QUADRANGLE, CALIFORNIA

By

Michael P. Kennedy[1] and Siang S. Tan[1]

2008

Digital preparation by

Kelly R. Bovard[2], Anne G. Garcia[2], Diane Burns[2], and Carlos I. Gutierrez[1]

1. Department of Conservation, California Geological Survey
2. U.S. Geological Survey, Department of Earth Sciences, University of California, Riverside

ONSHORE MAP SYMBOLS

	Contact - Contact between geologic units; dotted where concealed.
70 U D	Fault - Solid where accurately located; dashed where approximately located; dotted where concealed. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.
	Anticline - Solid where accurately located; dashed where approximately located; dotted where concealed. Arrow indicates direction of axial plunge.
	Syncline - Solid where accurately located; dotted where concealed. Arrow indicates direction of axial plunge.
Qls?	Landslide - Arrows indicate principal direction of movement. Queried where existence is questionable.
	Strike and dip of beds
70	Inclined
	Strike and dip of igneous joints
60	Inclined
	Vertical
	Strike and dip of metamorphic foliation
55	Inclined

DESCRIPTION OF MAP UNITS

Ta	**Ardath Shale (middle Eocene)**—Mostly uniform, weakly fissile olive-gray silty shale. The upper part contains thin beds of medium-grained sandstone, similar to thicker ones in the overlying Scripps Formation, and concretionary beds with molluscan fossils. The type section of the Ardath Shale is on the east side of Rose Canyon, 800 m south of the Ardath Road intersection with Interstate 5 (Kennedy and Moore, 1971)

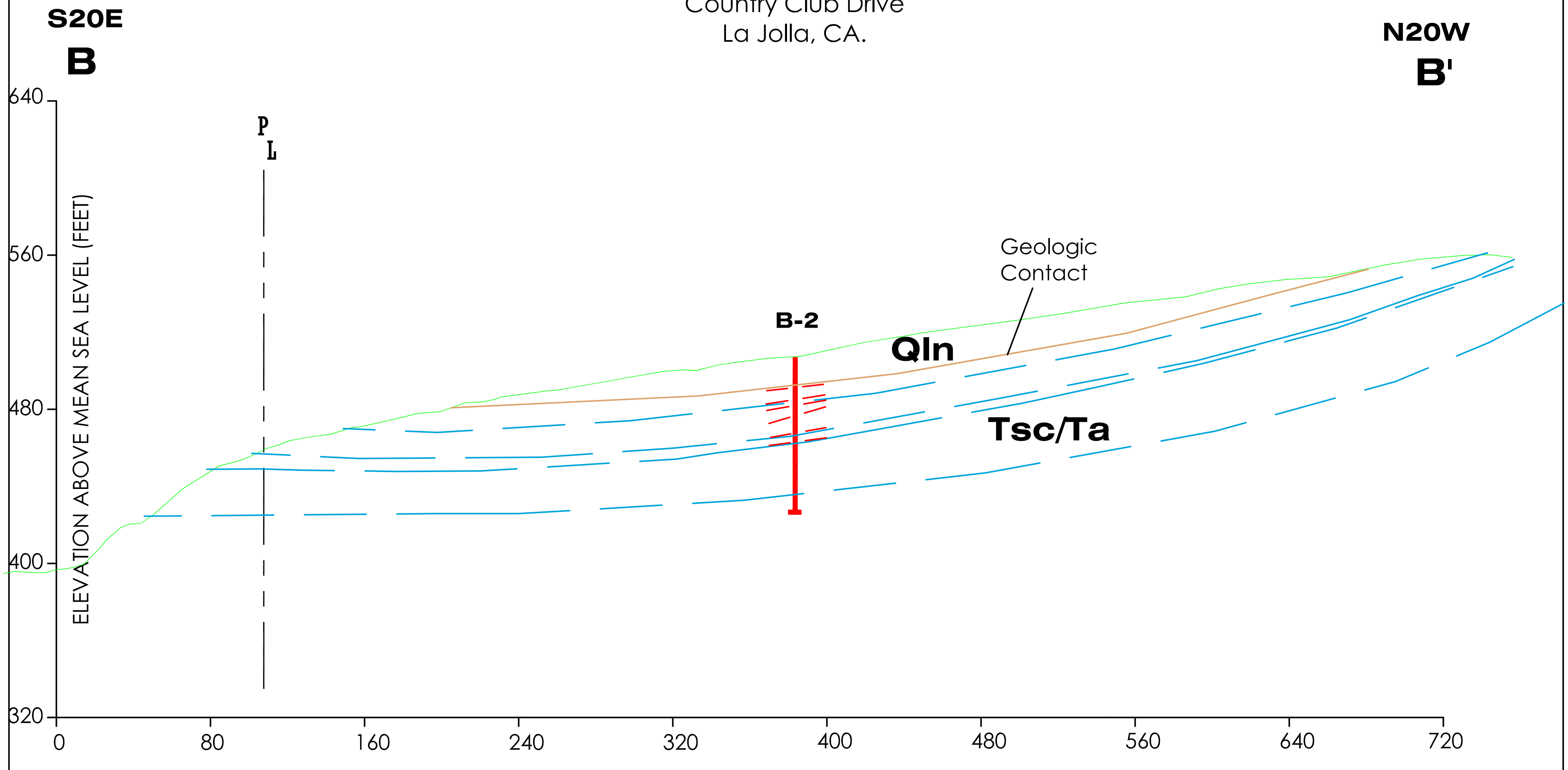
Figure No. Vb
Job No. 10-9977.1

Geotechnical Exploration, Inc.

October 2016

CROSS SECTION B-B'

The Reserve
Country Club Drive
La Jolla, CA.



RELATIVE HORIZONTAL DISTANCE (FEET)
SCALE: 1" = 80'

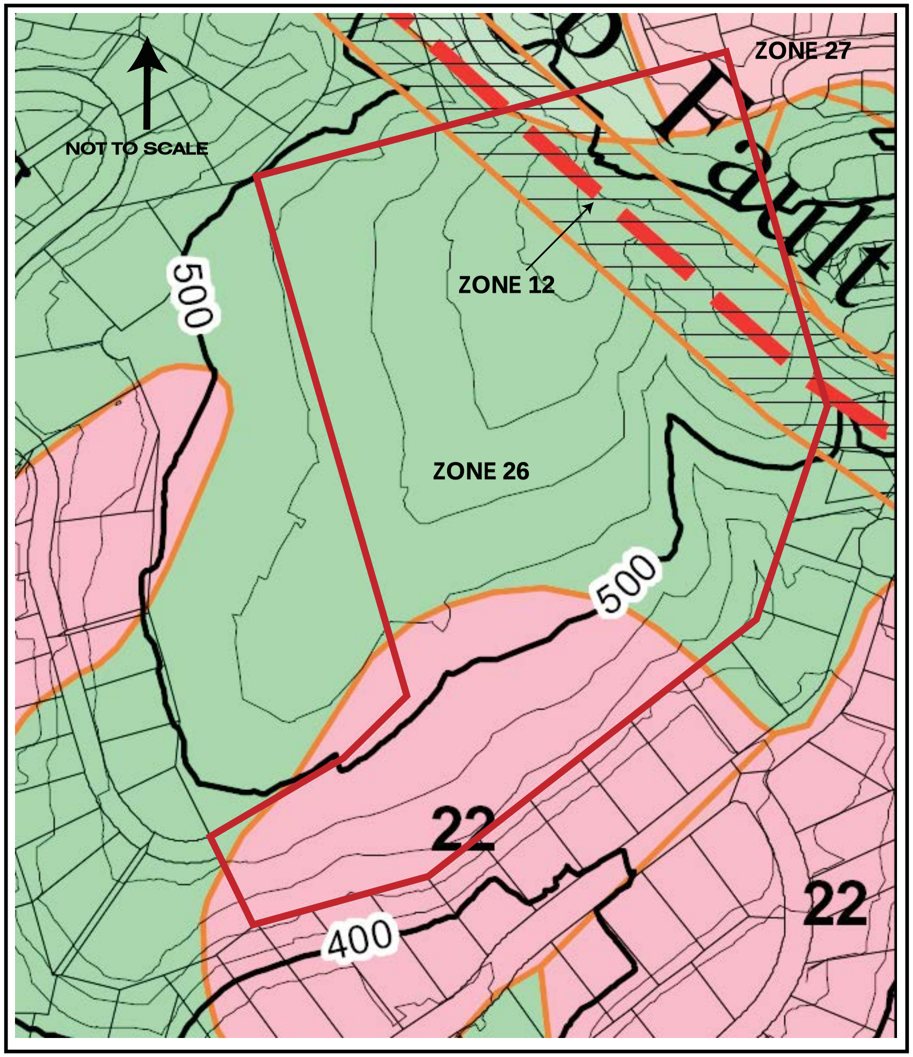
Figure No. VI
Job No. 10-9977

Geotechnical Exploration, Inc.

October 2011

10-9977-BB

Excerpted from City of San Diego
Seismic Safety Study
Geologic/Hazards and Faults
Sheet 29



LEGEND

Geologic Hazard Categories

FAULT ZONES
- ▨ 11 Active, Alquist-Priolo Earthquake Fault Zone
- ☰ 12 Potentially Active, Inactive, Presumed Inactive, or Activity Unknown
- ■ 13 Downtown special fault zone

LANDSLIDES
- ■ 21 Confirmed, known, or highly suspected
- ■ 22 Possible or conjectured

SLIDE-PRONE FORMATIONS
- ■ 23 Friars: neutral or favorable geologic structure
- ■ 24 Friars: unfavorable geologic structure
- ■ 25 Ardath: neutral or favorable geologic structure
- ■ 26 Ardath: unfavorable geologic structure
- ■ 27 Otay, Sweetwater, and others

LIQUEFACTION
- ■ 31 High Potential -- shallow groundwater major drainages, hydraulic fills
- ■ 32 Low Potential -- fluctuating groundwater minor drainages

COASTAL BLUFFS
- ■ 41 Generally unstable Numerous landslides, high steep bluffs, severe erosion, unfavorable geologic structure
- ■ 42 Generally unstable Unfavorable bedding plains, high erosion
- ■ 43 Generally unstable Unfavorable jointing, local high erosion
- ■ 44 Moderately stable Mostly stable formations, local high erosion
- ■ 45 Moderately stable Some minor landslides, minor erosion
- ■ 46 Moderately stable Some unfavorable geologic structure, minor or no erosion
- ■ 47 Generally stable Favorable geologic structure, minor or no erosion, no landslides
- ■ 48 Generally stable Broad beach areas, developed harbor

OTHER TERRAIN
- ■ 51 Level mesas -- underlain by terrace deposits and bedrock nominal risk
- ■ 52 Other level areas, gently sloping to steep terrain, favorable geologic structure, Low risk
- ■ 53 Level or sloping terrain, unfavorable geologic structure, Low to moderate risk
- ■ 54 Steeply sloping terrain, unfavorable or fault controlled geologic structure, Moderate risk
- ■ 55 Modified terrain (graded sites) Nominal risk

Water (Bays and Lakes)
- ■

FAULTS
- Fault
- Inferred Fault
- Concealed Fault
- Shear Zone

City of San Diego
SEISMIC SAFETY STUDY
Geologic Hazards and Faults

Development Services Department

DATE: 4/3/2008

The Reserve
Country Club Drive
La Jolla, CA.

Figure No. VII
Job No. 10-9977

Geotechnical Exploration, Inc.

LOCAL FAULT MAP

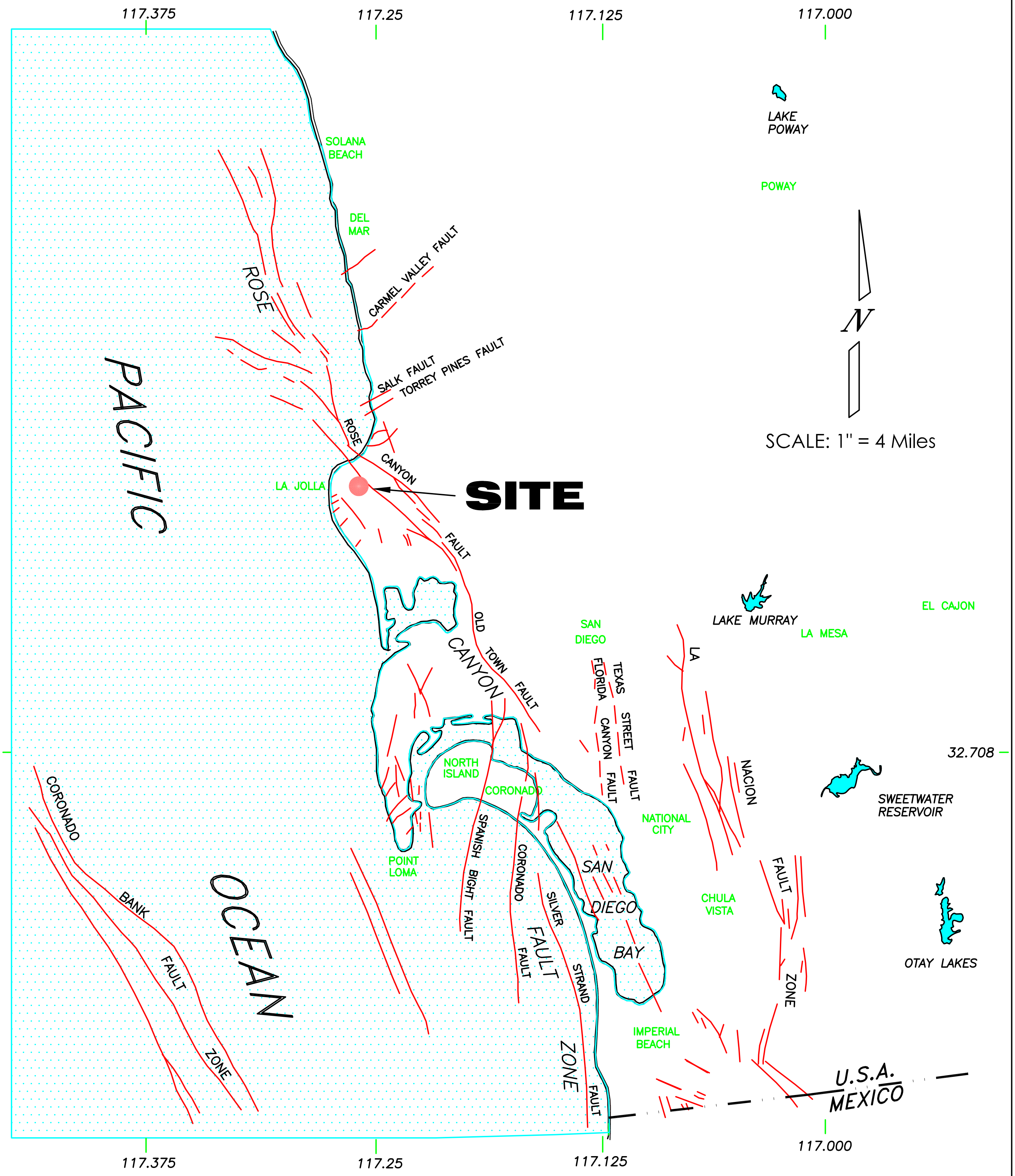


Figure No. VIIIb

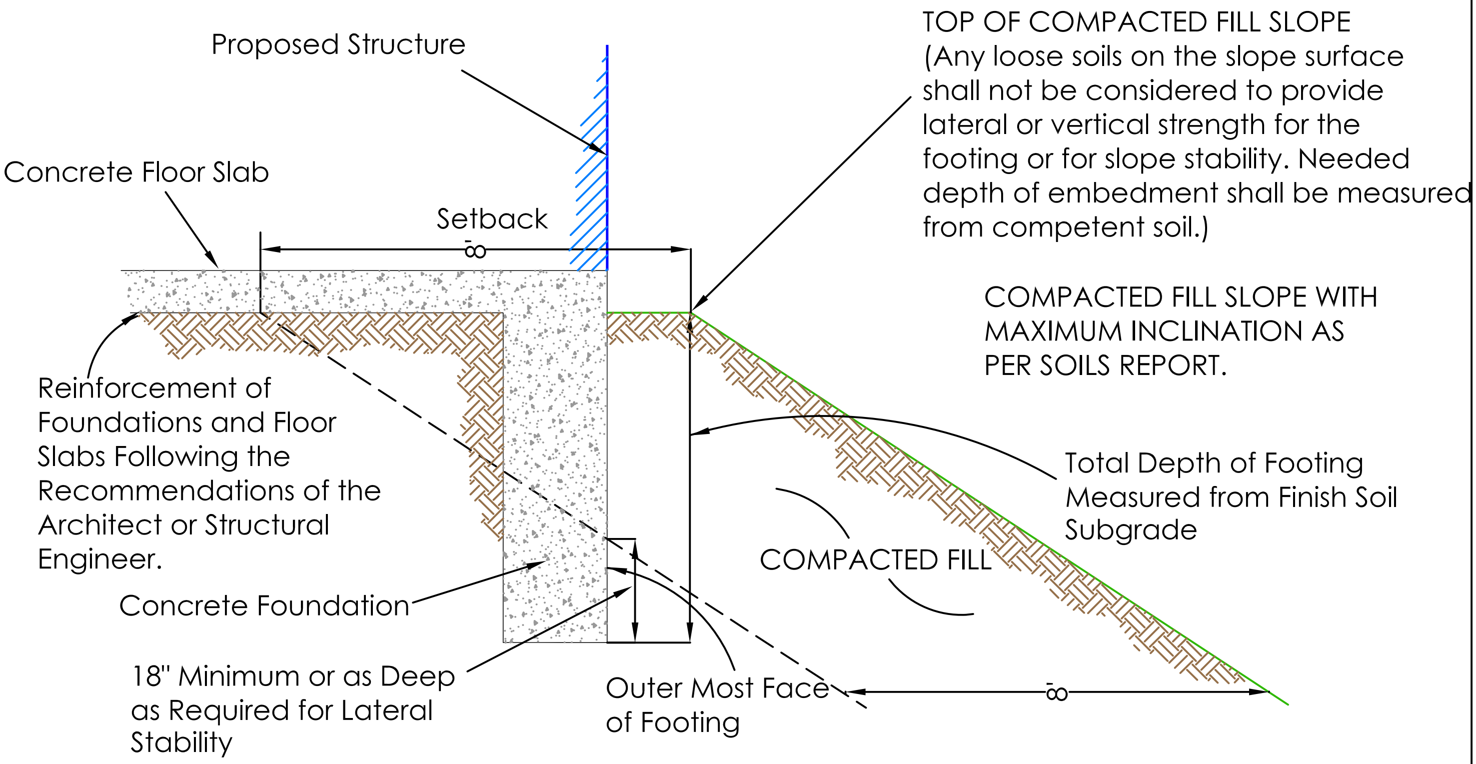
Job No. 10-9977

Geotechnical Exploration, Inc.

COMPILED FROM C.D.M.G. MAPS

10-9977-VIIIb

FOUNDATION REQUIREMENTS NEAR SLOPES



TYPICAL SECTION

(Showing Proposed Foundation Located Within 8 Feet of Top of Slope)

18" FOOTING / 8' SETBACK

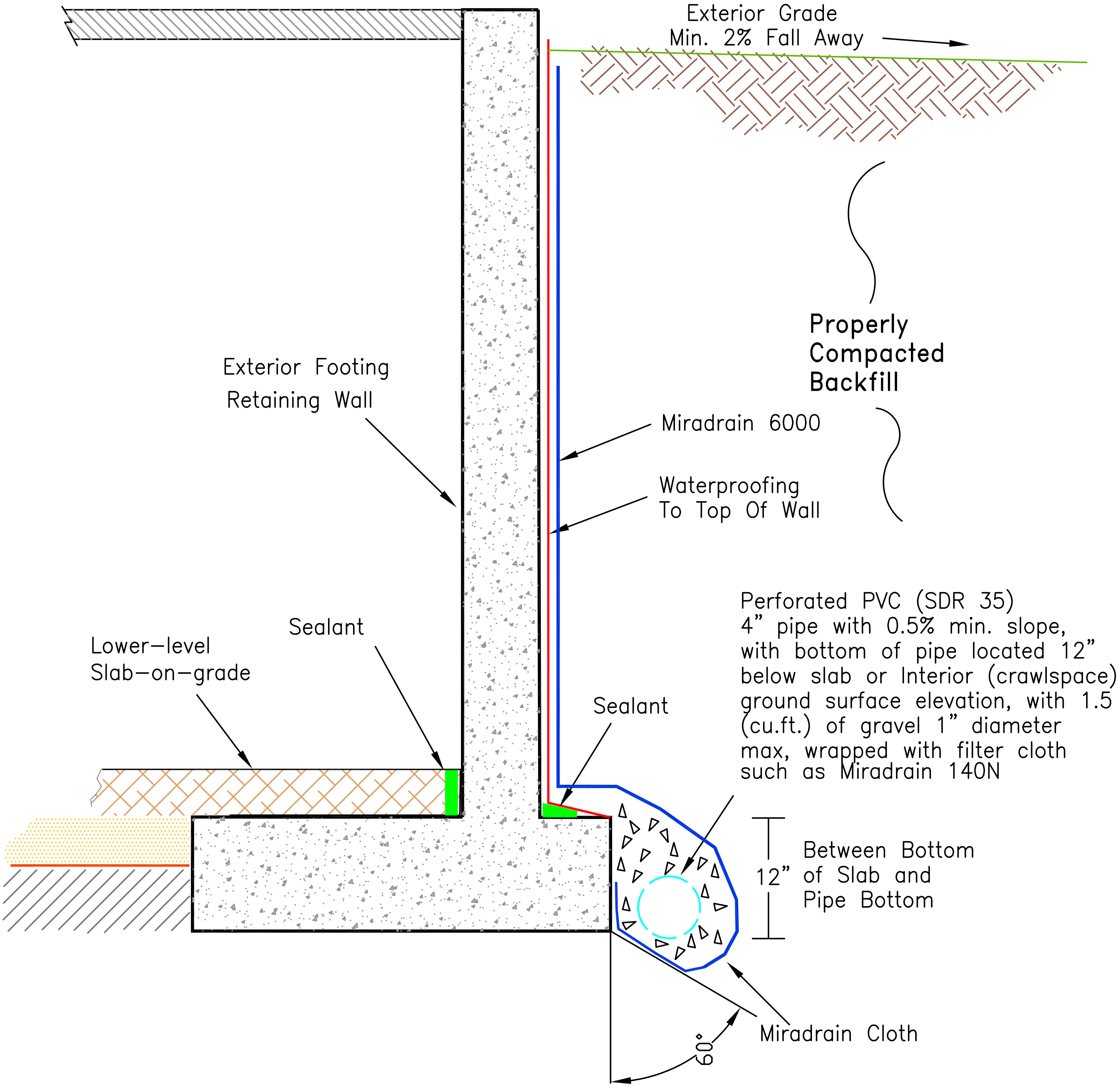
Distance From Top of Slope	Total Depth of Footing	
	1.5:1.0 SLOPE *	2.0:1.0 SLOPE
0	82"	66"
2'	66"	54"
4'	51"	42"
6'	34"	30"
8'	18"	18"

* when applicable

Figure No. IX
Job No. 10-9977.1

Geotechnical Exploration, Inc.

SUBGRADE RETAINING WALL DRAINAGE RECOMMENDATIONS



NOT TO SCALE

Figure No. X
Job No. 10-9977

Geotechnical Exploration, Inc.

APPENDIX A
UNIFIED SOIL CLASSIFICATION CHART
SOIL DESCRIPTION

Coarse-grained (More than half of material is larger than a No. 200 sieve)

GRAVELS, CLEAN GRAVELS (More than half of coarse fraction is larger than No. 4 sieve size, but smaller than 3”)	GW	Well-graded gravels, gravel and sand mixtures, little or no fines.
	GP	Poorly graded gravels, gravel and sand mixtures, little or no fines.
GRAVELS WITH FINES (Appreciable amount)	GC	Clay gravels, poorly graded gravel-sand-silt mixtures
SANDS, CLEAN SANDS (More than half of coarse fraction is smaller than a No. 4 sieve)	SW	Well-graded sand, gravelly sands, little or no fines
	SP	Poorly graded sands, gravelly sands, little or no fines.
SANDS WITH FINES (Appreciable amount)	SM	Silty sands, poorly graded sand and silty mixtures.
	SC	Clayey sands, poorly graded sand and clay mixtures.

Fine-grained (More than half of material is smaller than a No. 200 sieve)

SILTS AND CLAYS

Liquid Limit Less than 50	ML	Inorganic silts and very fine sands, rock flour, sandy silt and clayey-silt sand mixtures with a slight plasticity
	CL	Inorganic clays of low to medium plasticity, gravelly clays, silty clays, clean clays.
	OL	Organic silts and organic silty clays of low plasticity.
Liquid Limit Greater than 50	MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
	CH	Inorganic clays of high plasticity, fat clays.
	OH	Organic clays of medium to high plasticity.
HIGHLY ORGANIC SOILS	PT	Peat and other highly organic soils

APPENDIX B

Slope Stability Analysis

The Reserve Property, Job No.10-9977 Sectn. A-A'

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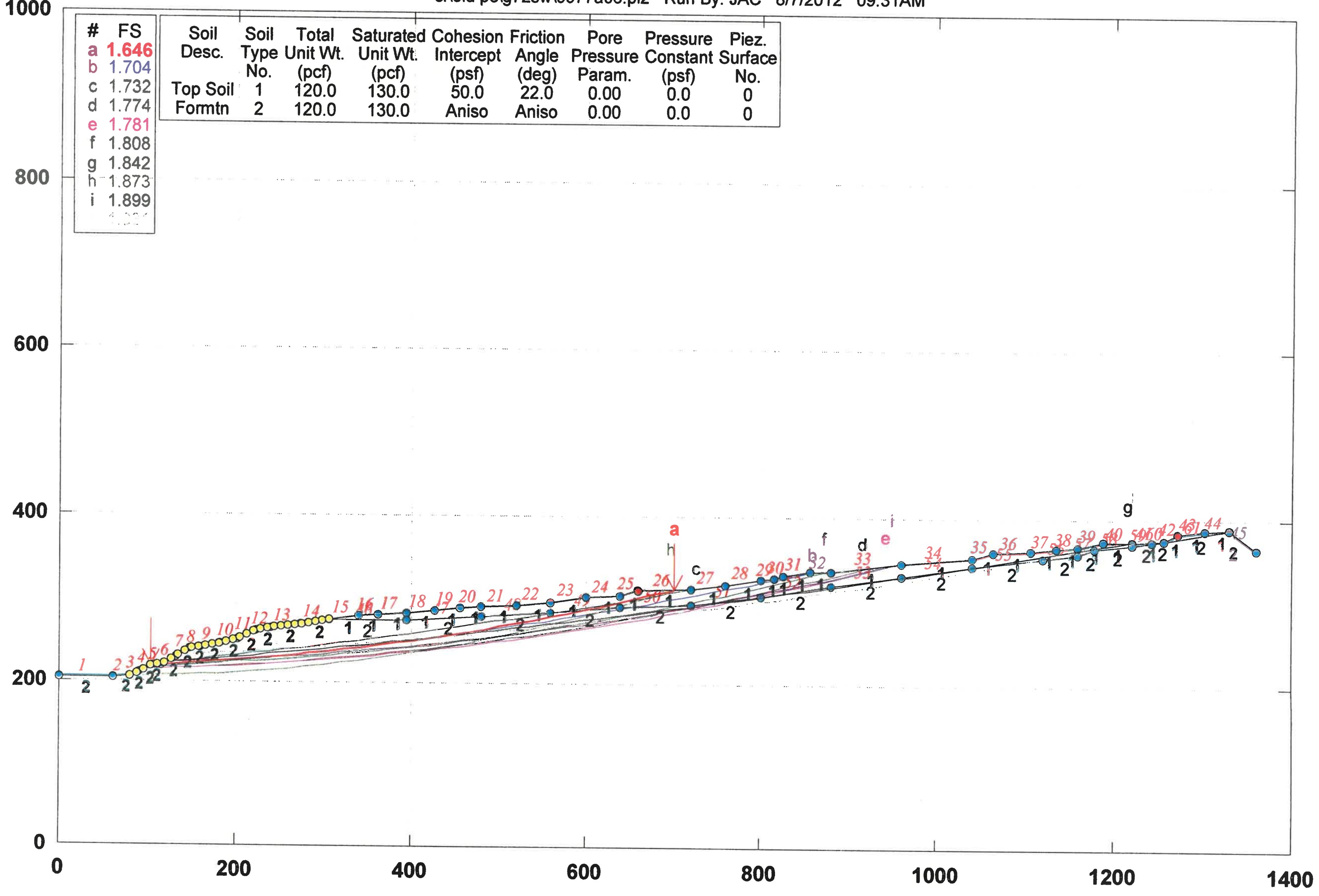

#	FS
a	1.646
b	1.704
c	1.732
d	1.774
e	1.781
f	1.808
g	1.842
h	1.873
i	1.899

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.
Top Soil	1	120.0	130.0	50.0	22.0	0.00	0.0	0
Formtn	2	120.0	130.0	Aniso	Aniso	0.00	0.0	0

GSTABL7 v.2 FSmin=1.646

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

The Reserve Property, Job No.10-9977 Sectn. B-B'

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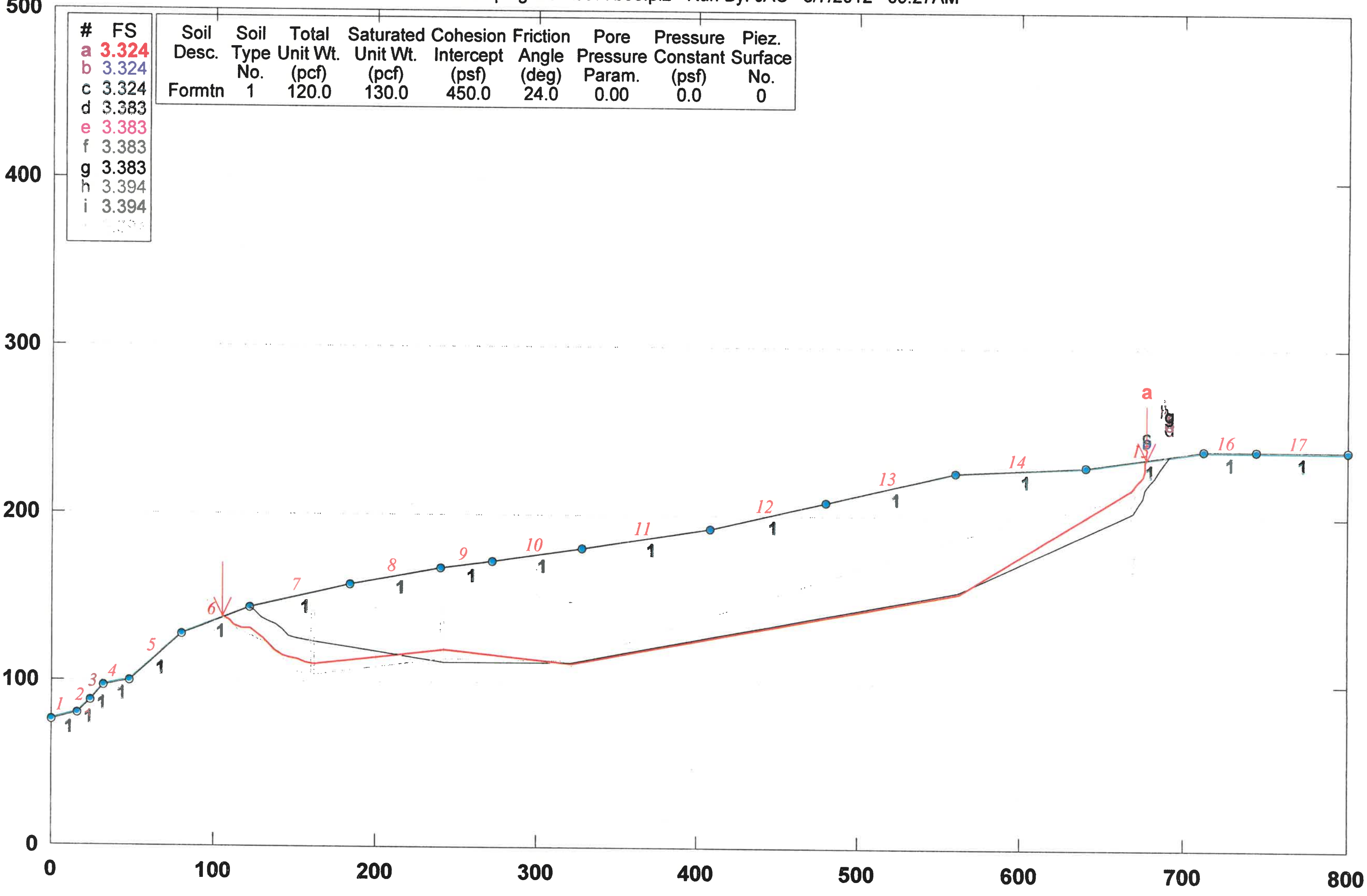

#	FS
a	3.324
b	3.324
c	3.324
d	3.383
e	3.383
f	3.383
g	3.383
h	3.394
i	3.394

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.
Formtn	1	120.0	130.0	450.0	24.0	0.00	0.0	0

GSTABL7 v.2 FSmin=3.324

Safety Factors Are Calculated By The Simplified Janbu Method

The Reserve Property, Job No.10-9977 Sectn. B-B' Anisotropic Soil Condition

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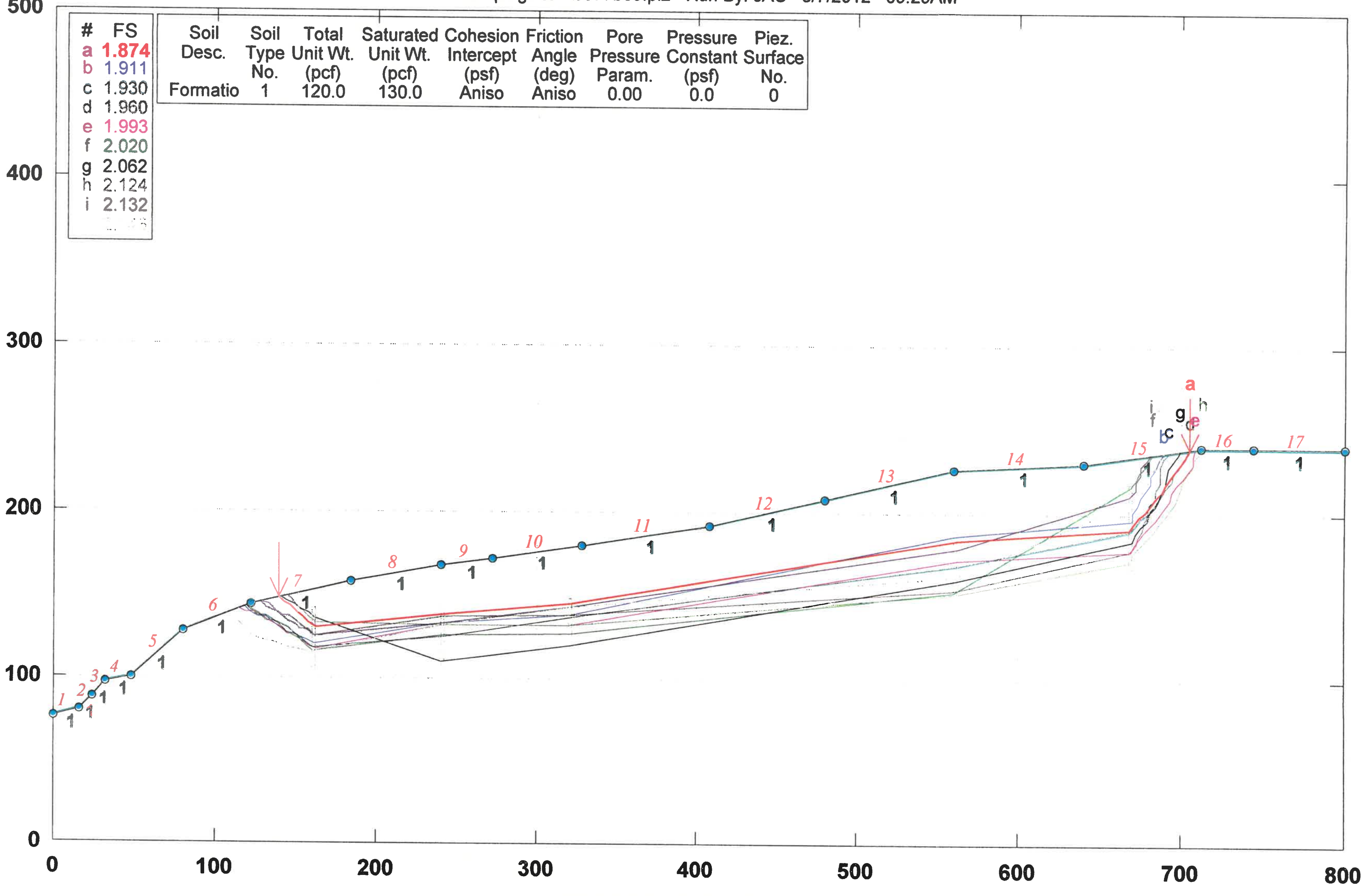

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Formatio	1	120.0	130.0	Aniso	Aniso	0.00	0.0	0

GSTABL7 v.2 FSmin=1.874

Safety Factors Are Calculated By The Simplified Janbu Method

The Reserve Property, Job No.10-9977 Sectn. B-B' Anisotropic Soil Condition

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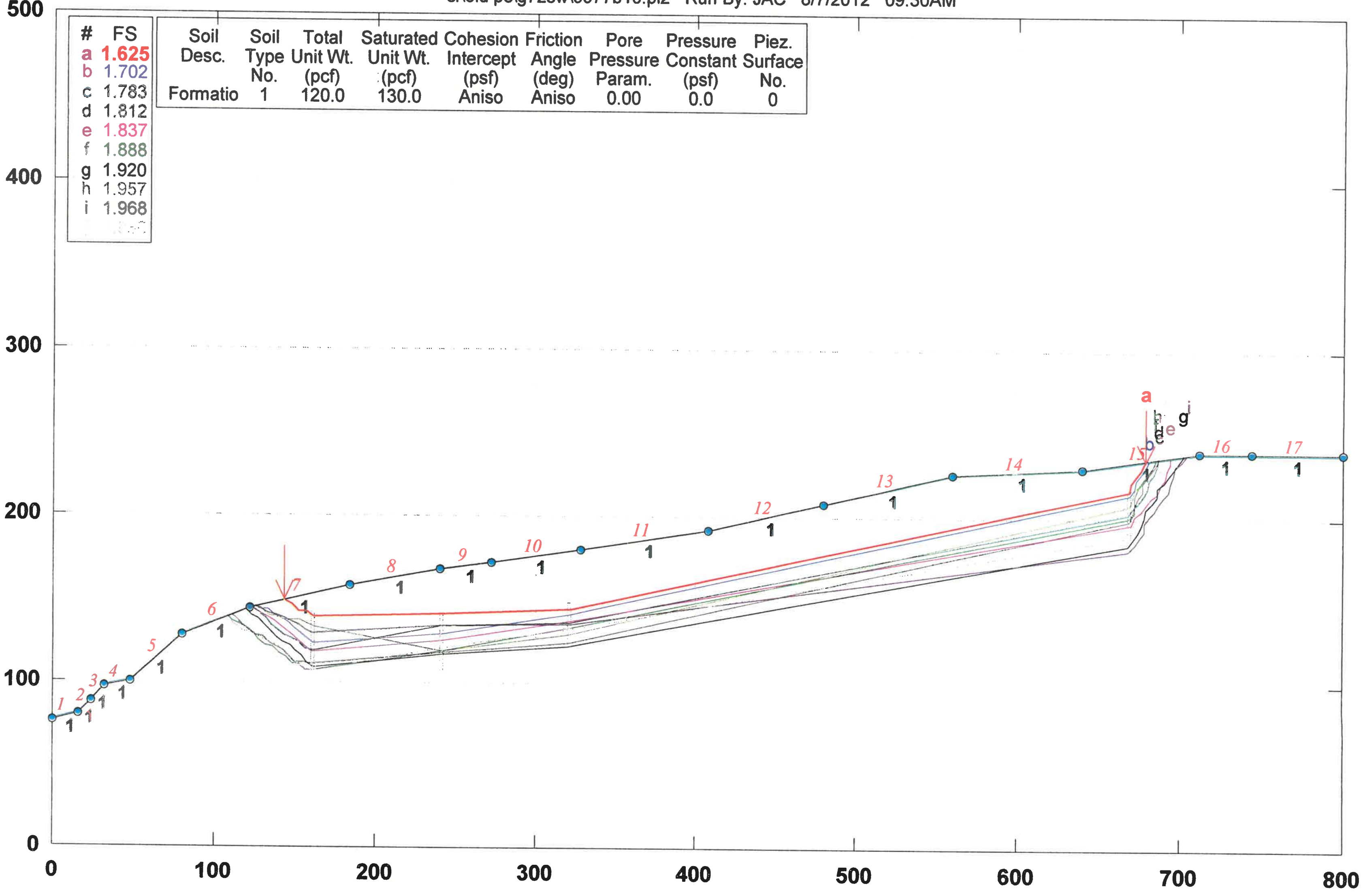

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Formatio	1	120.0	130.0	Aniso	Aniso	0.00	0.0	0

#	FS
a	1.625
b	1.702
c	1.783
d	1.812
e	1.837
f	1.888
g	1.920
h	1.957
i	1.968

GSTABL7 v.2 FSmin=1.625

Safety Factors Are Calculated By The Simplified Janbu Method

The Reserve Property, Job No.10-9977 Sectn. C-C'

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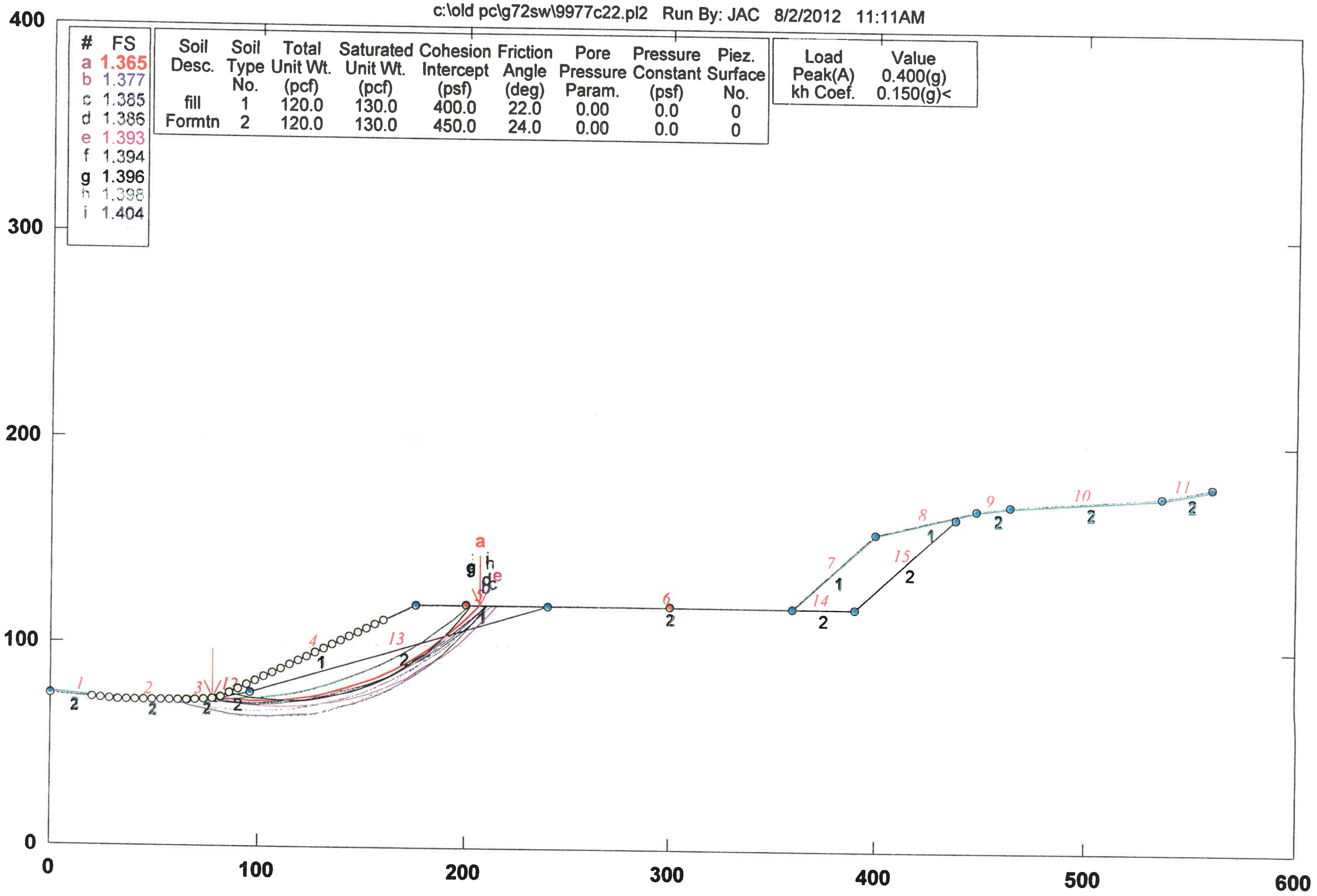

#	FS
a	1.365
b	1.377
c	1.385
d	1.386
e	1.393
f	1.394
g	1.396
h	1.398
i	1.404

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	400.0	22.0	0.00	0.0	0
Formtn	2	120.0	130.0	450.0	24.0	0.00	0.0	0

Load	Value
Peak(A)	0.400(g)
kh Coef.	0.150(g)<

GSTABL7 v.2 FSmin=1.365

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

The Reserve Property, Job No.10-9977 Sectn. C-C'

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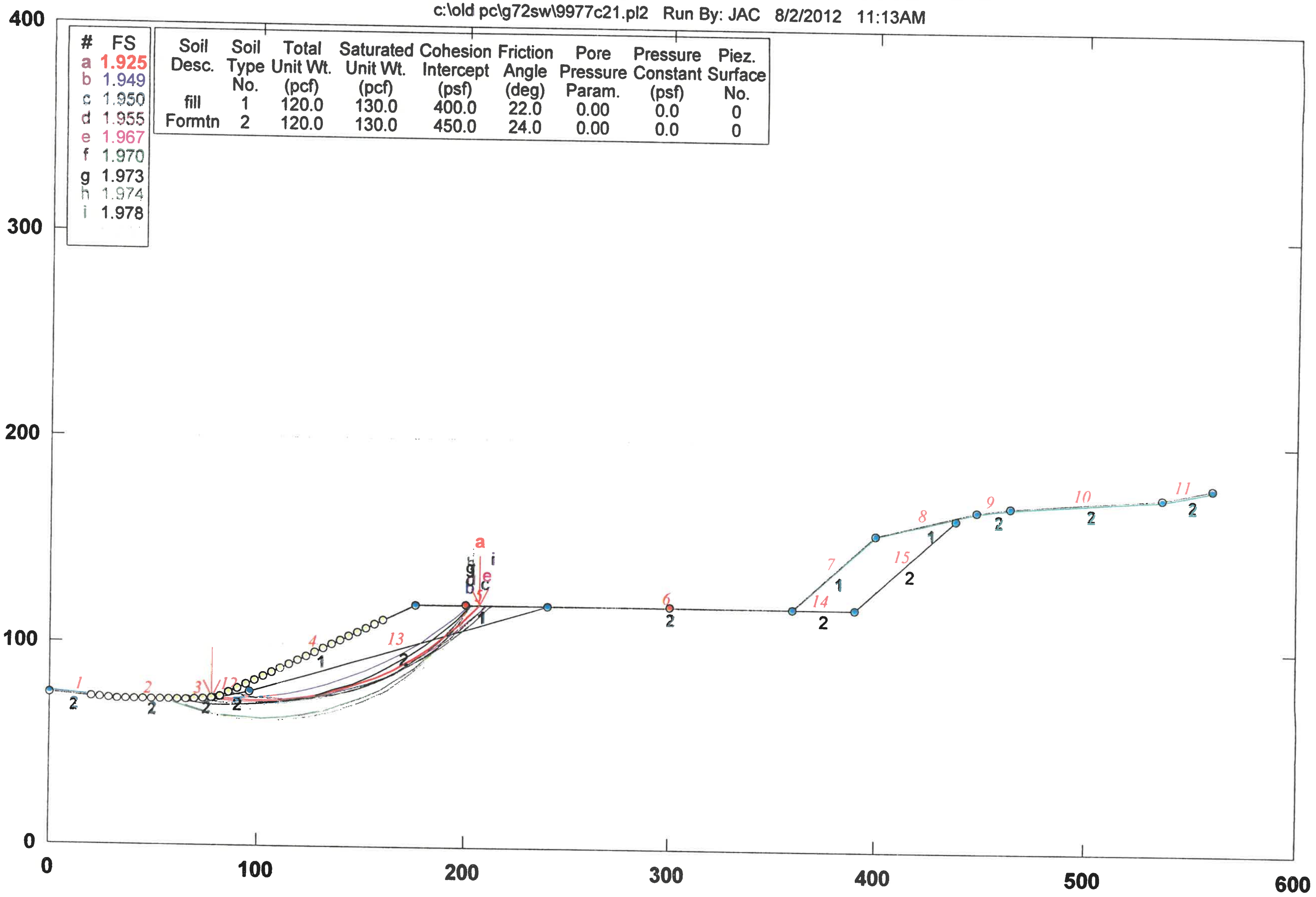

#	FS
a	1.925
b	1.949
c	1.950
d	1.955
e	1.967
f	1.970
g	1.973
h	1.974
i	1.978

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	400.0	22.0	0.00	0.0	0
Formtn	2	120.0	130.0	450.0	24.0	0.00	0.0	0

GSTABL7 v.2 FSmin=1.925

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

The Reserve Property, Job No.10-9977 Sectn. C-C'

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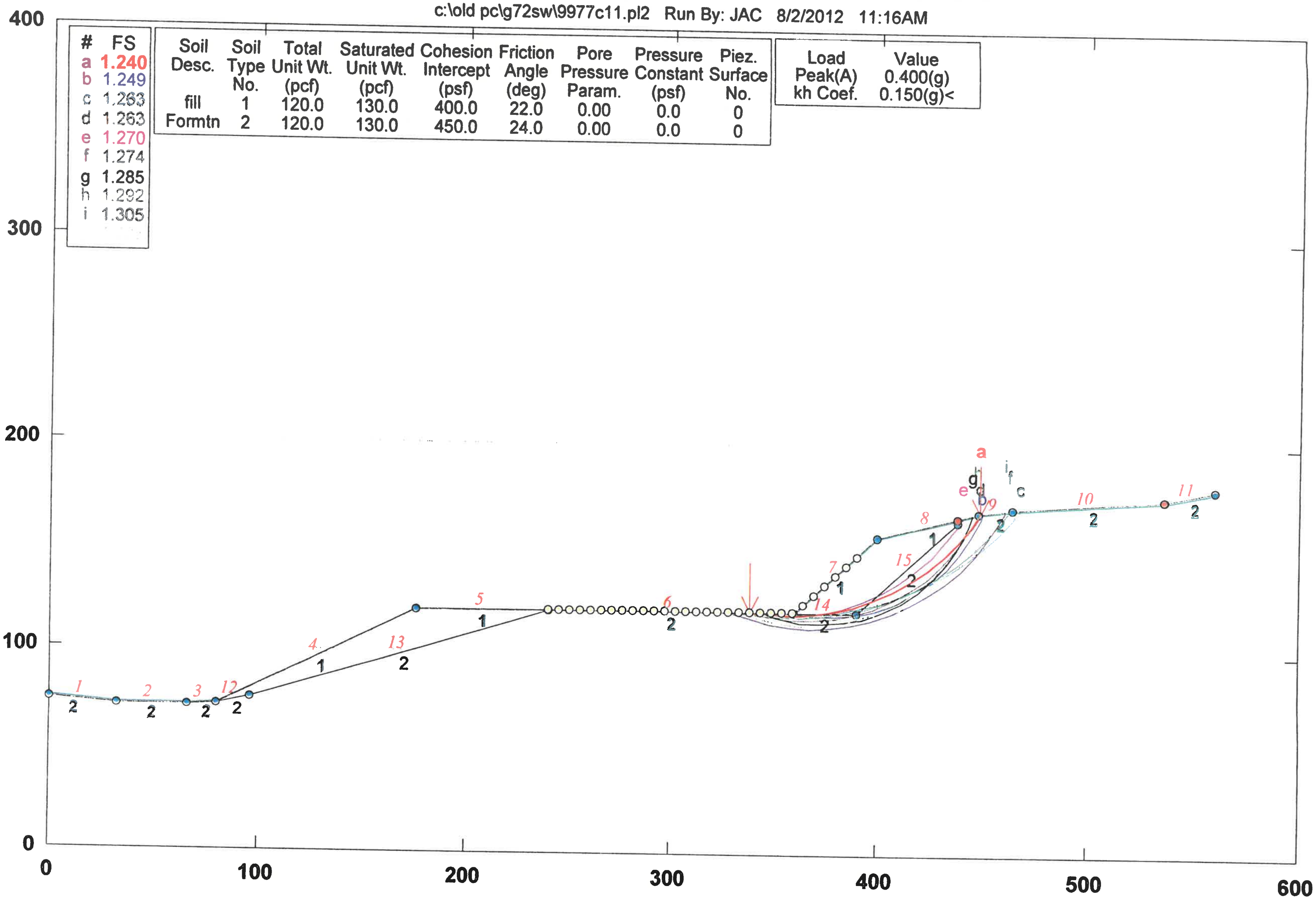

#	FS
a	1.240
b	1.249
c	1.263
d	1.263
e	1.270
f	1.274
g	1.285
h	1.292
i	1.305

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	400.0	22.0	0.00	0.0	0
Formtn	2	120.0	130.0	450.0	24.0	0.00	0.0	0

Load	Value
Peak(A)	0.400(g)
kh Coef.	0.150(g)<

GSTABL7 v.2 FSmin=1.240

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

The Reserve Property, Job No.10-9977 Sectn. C-C'

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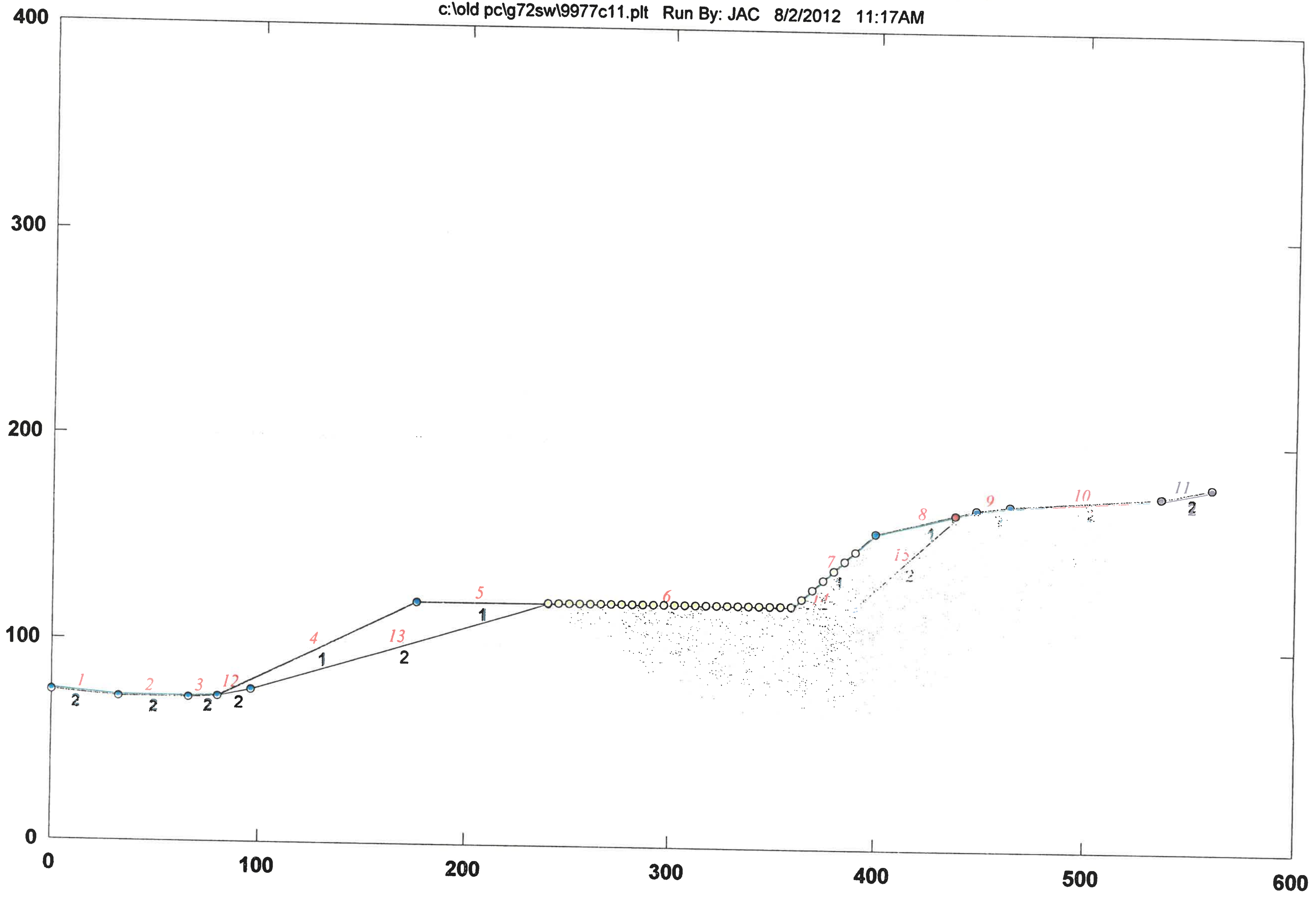

The Reserve Property, Job No.10-9977 Sectn. C-C'

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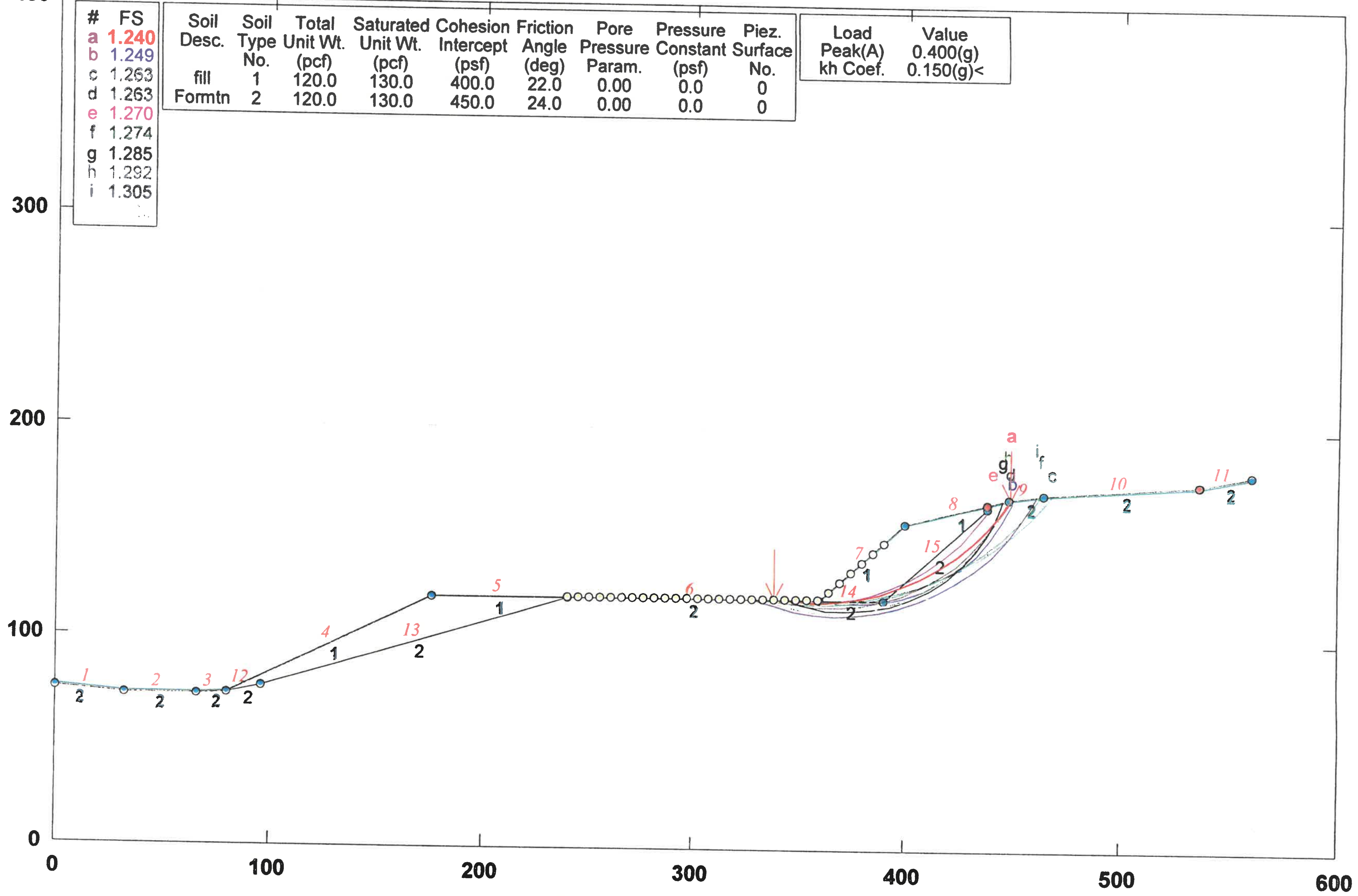

#	FS
a	1.240
b	1.249
c	1.263
d	1.263
e	1.270
f	1.274
g	1.285
h	1.292
i	1.305

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	400.0	22.0	0.00	0.0	0
Formtn	2	120.0	130.0	450.0	24.0	0.00	0.0	0

Load	Value
Peak(A)	0.400(g)
kh Coef.	0.150(g)<

GSTABL7 v.2 FSmin=1.240

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

The Reserve Property, Job No.10-9977 Sectn. C-C'

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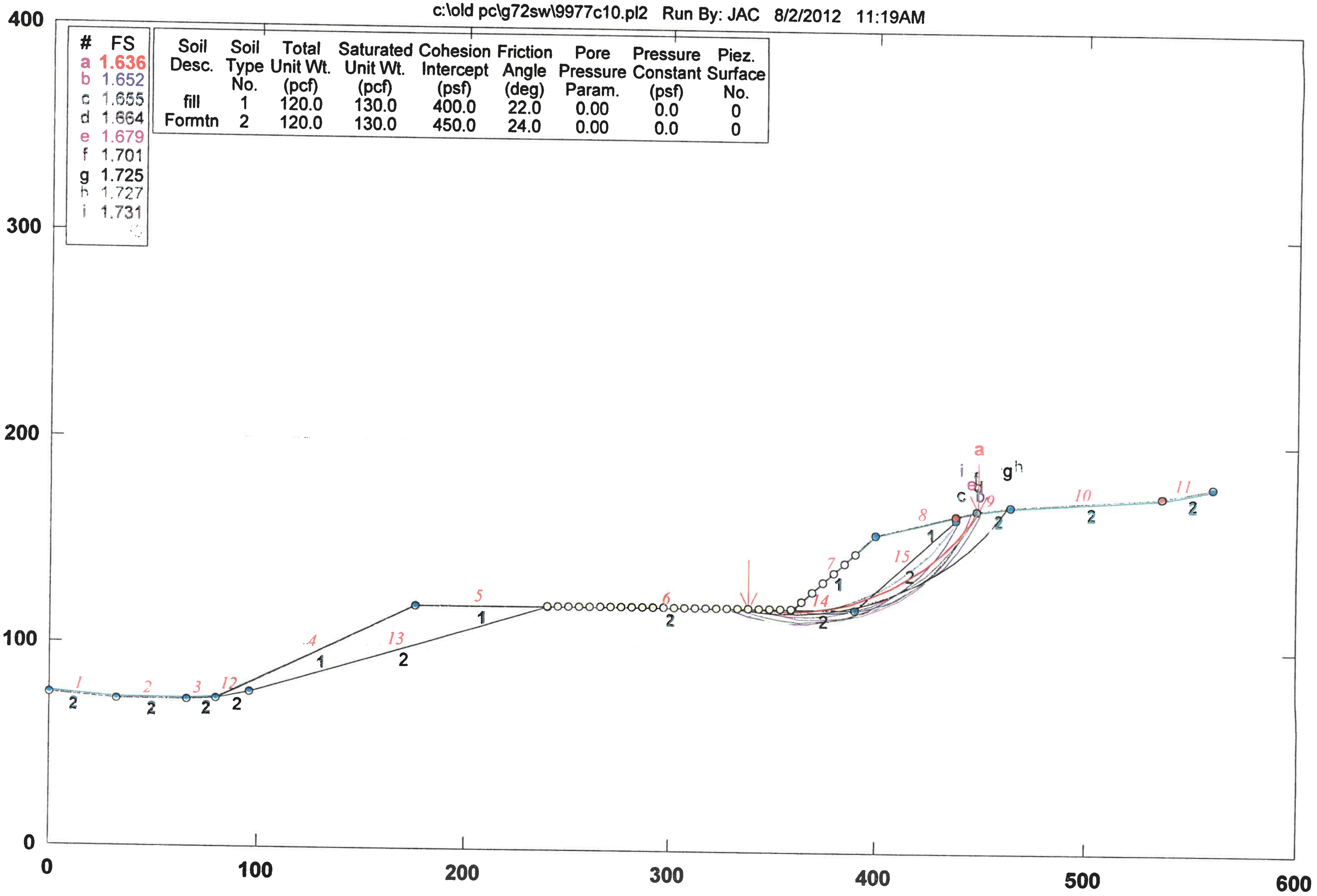

#	FS
a	1.636
b	1.652
c	1.655
d	1.664
e	1.679
f	1.701
g	1.725
h	1.727
i	1.731

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	400.0	22.0	0.00	0.0	0
Formtn	2	120.0	130.0	450.0	24.0	0.00	0.0	0

GSTABL7 v.2 FSmin=1.636

Safety Factors Are Calculated By The Modified Bishop Method

GSTABL7

The Reserve Property, Job No.10-9977 Sectn. C-C'

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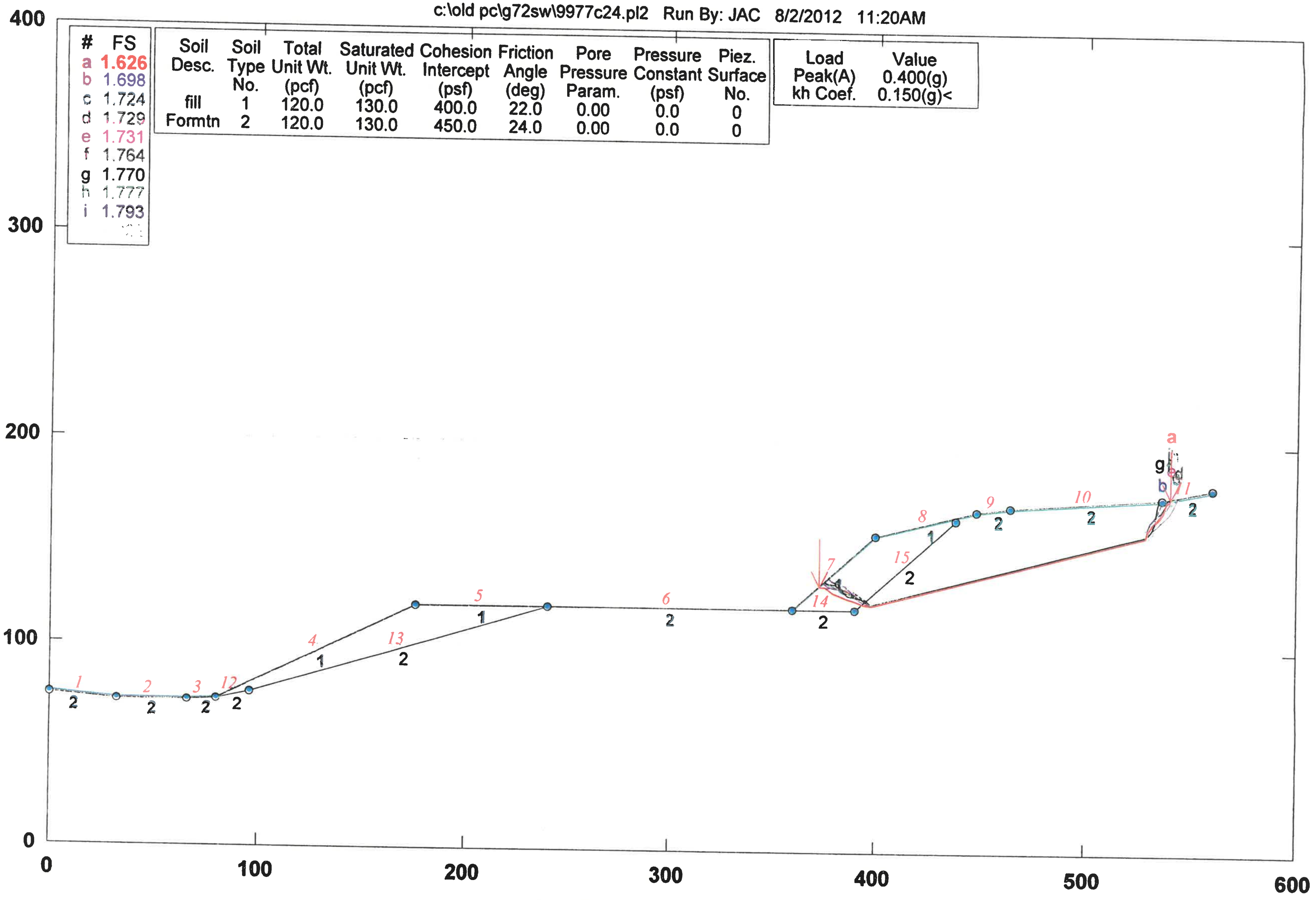

#	FS
a	1.626
b	1.698
c	1.724
d	1.729
e	1.731
f	1.764
g	1.770
h	1.777
i	1.793

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	400.0	22.0	0.00	0.0	0
Formtn	2	120.0	130.0	450.0	24.0	0.00	0.0	0

Load	Value
Peak(A)	0.400(g)
kh Coef.	0.150(g)<

GSTABL7 v.2 FSmin=1.626

Safety Factors Are Calculated By The Simplified Janbu Method

The Reserve Property, Job No.10-9977 Sectn. C-C'

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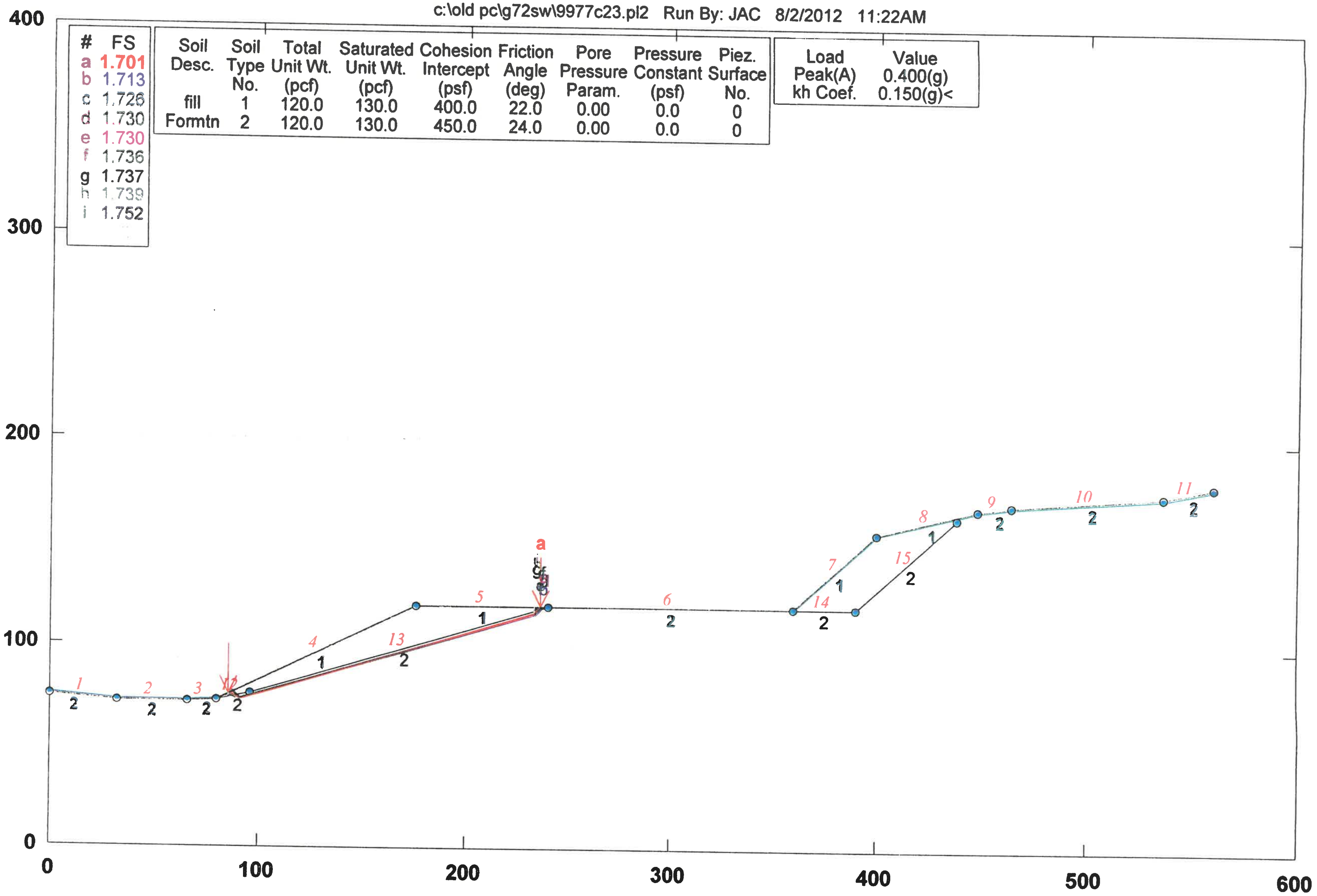

#	FS
a	1.701
b	1.713
c	1.726
d	1.730
e	1.730
f	1.736
g	1.737
h	1.739
i	1.752

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	400.0	22.0	0.00	0.0	0
Formtn	2	120.0	130.0	450.0	24.0	0.00	0.0	0

Load	Value
Peak(A)	0.400(g)
kh Coef.	0.150(g)<

GSTABL7 v.2 FSmin=1.701

Safety Factors Are Calculated By The Simplified Janbu Method

GSTABL7

The Reserve Property, Job No.10-9977 Borrow Area

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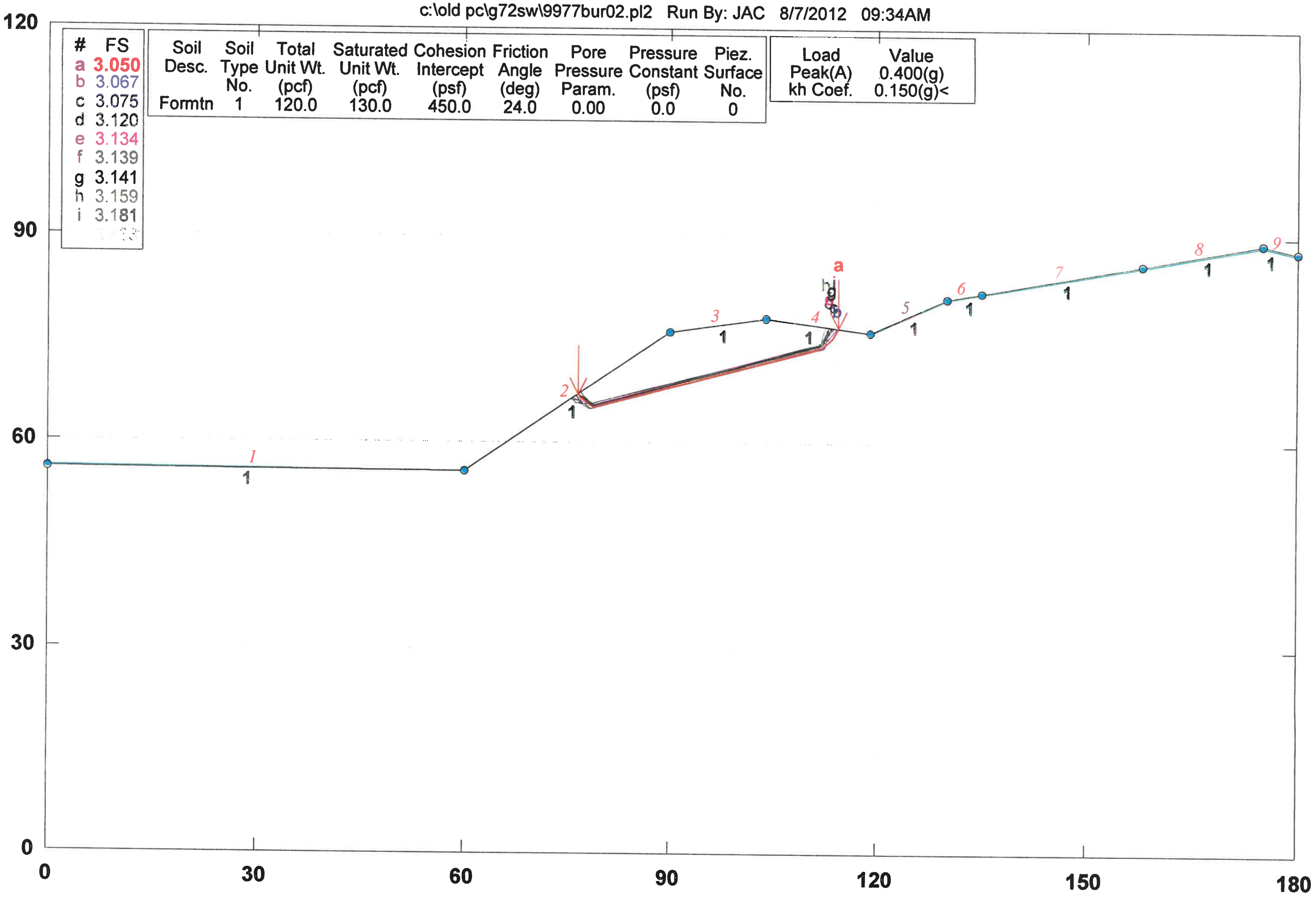

#	FS
a	3.050
b	3.067
c	3.075
d	3.120
e	3.134
f	3.139
g	3.141
h	3.159
i	3.181

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.
Formtn	1	120.0	130.0	450.0	24.0	0.00	0.0	0

Load	Value
Peak(A)	0.400(g)
kh Coef.	0.150(g)<

GSTABL7 v.2 FSmin=3.050

Safety Factors Are Calculated By The Simplified Janbu Method

GSTABL7

The Reserve Property, Job No.10-9977 Borrow Area

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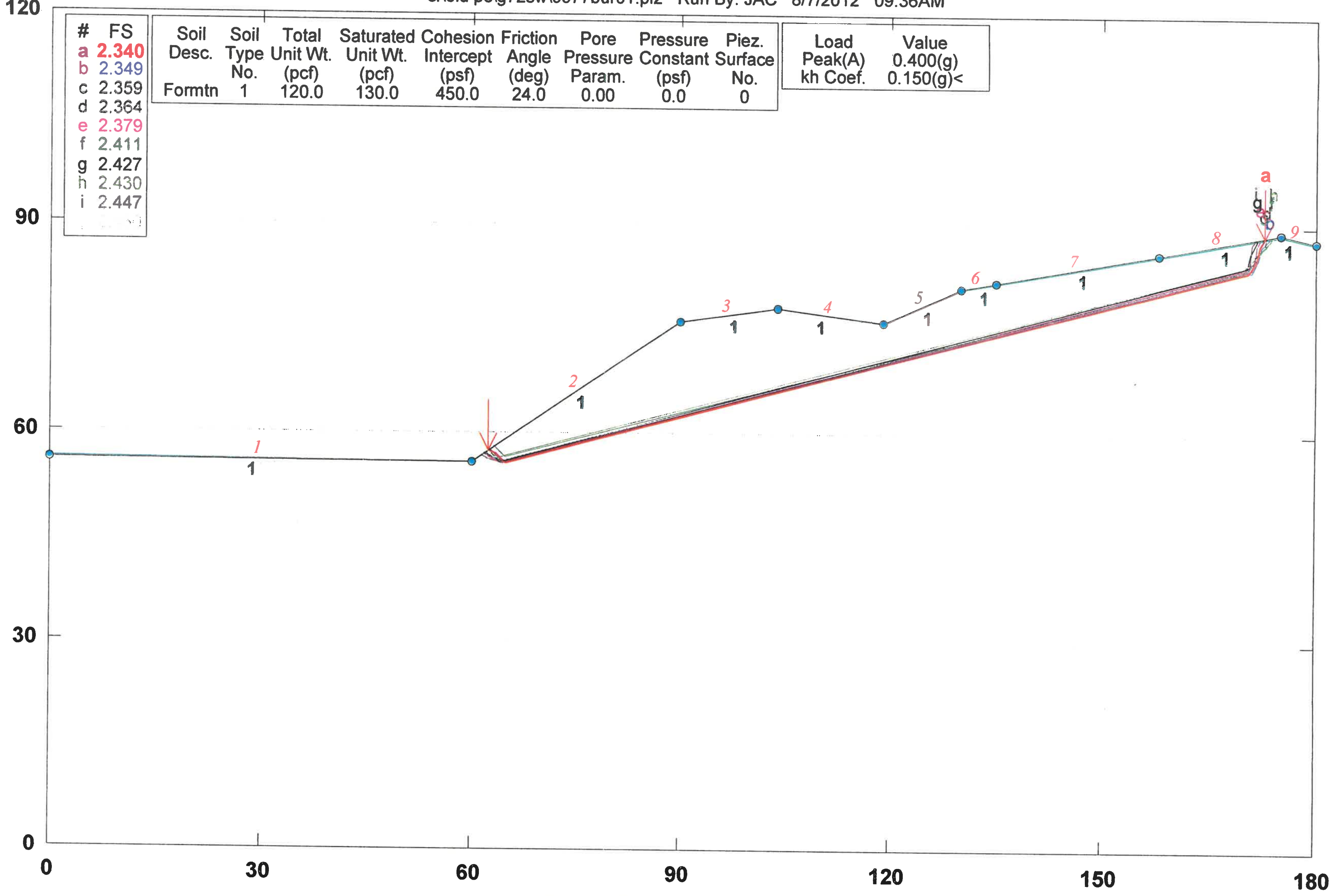

#	FS
a	2.340
b	2.349
c	2.359
d	2.364
e	2.379
f	2.411
g	2.427
h	2.430
i	2.447

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.
Formtn	1	120.0	130.0	450.0	24.0	0.00	0.0	0

Load	Value
Peak(A)	0.400(g)
kh Coef.	0.150(g)<

GSTABL7 v.2 FSmin=2.340

Safety Factors Are Calculated By The Simplified Janbu Method

The Reserve Property, Job No.10-9977 Sectn. Lot 2'

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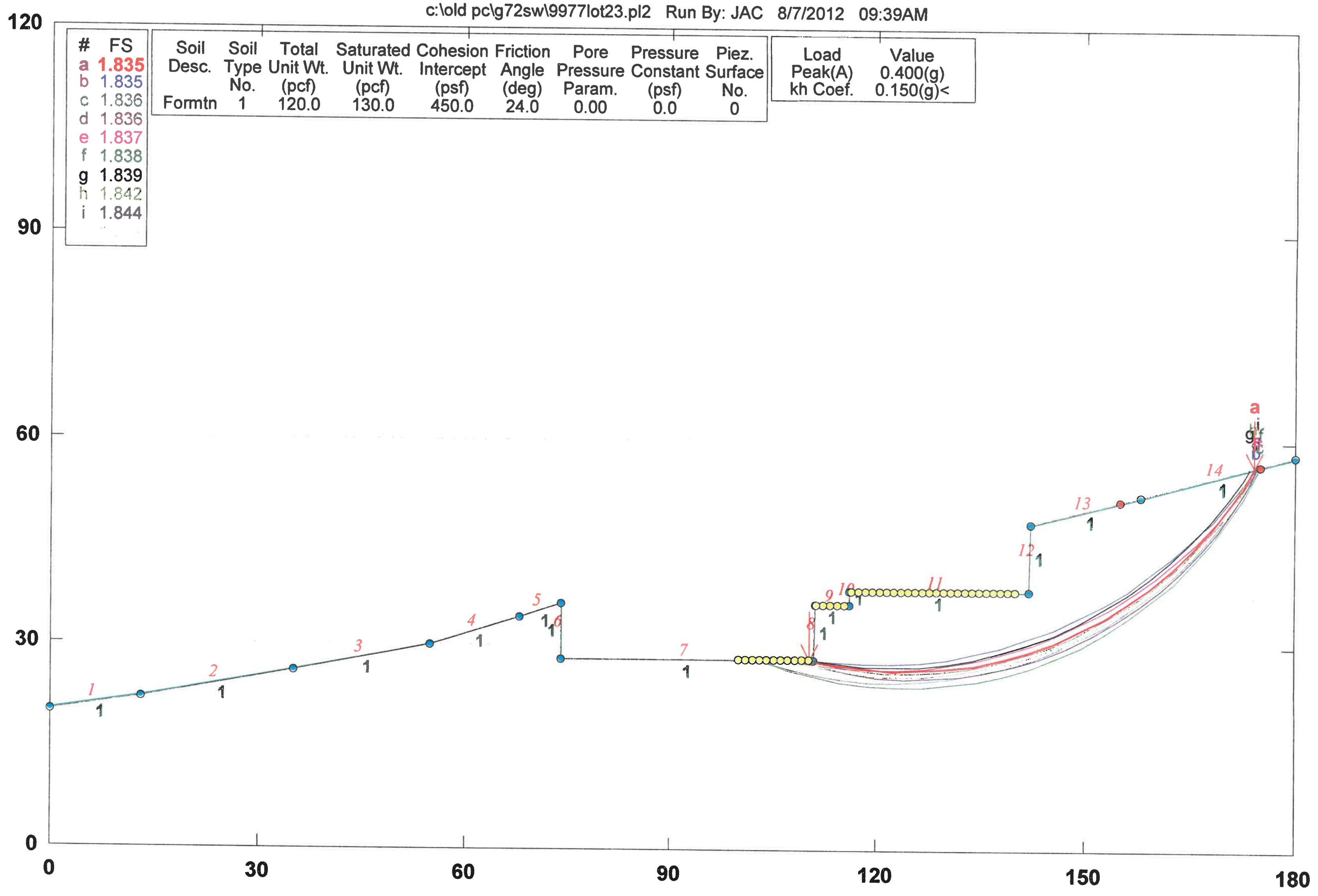

#	FS
a	1.835
b	1.835
c	1.836
d	1.836
e	1.837
f	1.838
g	1.839
h	1.842
i	1.844

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.
Formtn	1	120.0	130.0	450.0	24.0	0.00	0.0	0

Load	Value
Peak(A)	0.400(g)
kh Coef.	0.150(g)<

GSTABL7 v.2 FSmin=1.835

Safety Factors Are Calculated By The Modified Bishop Method

The Reserve Property, Job No.10-9977 Sectn. Lot 2'

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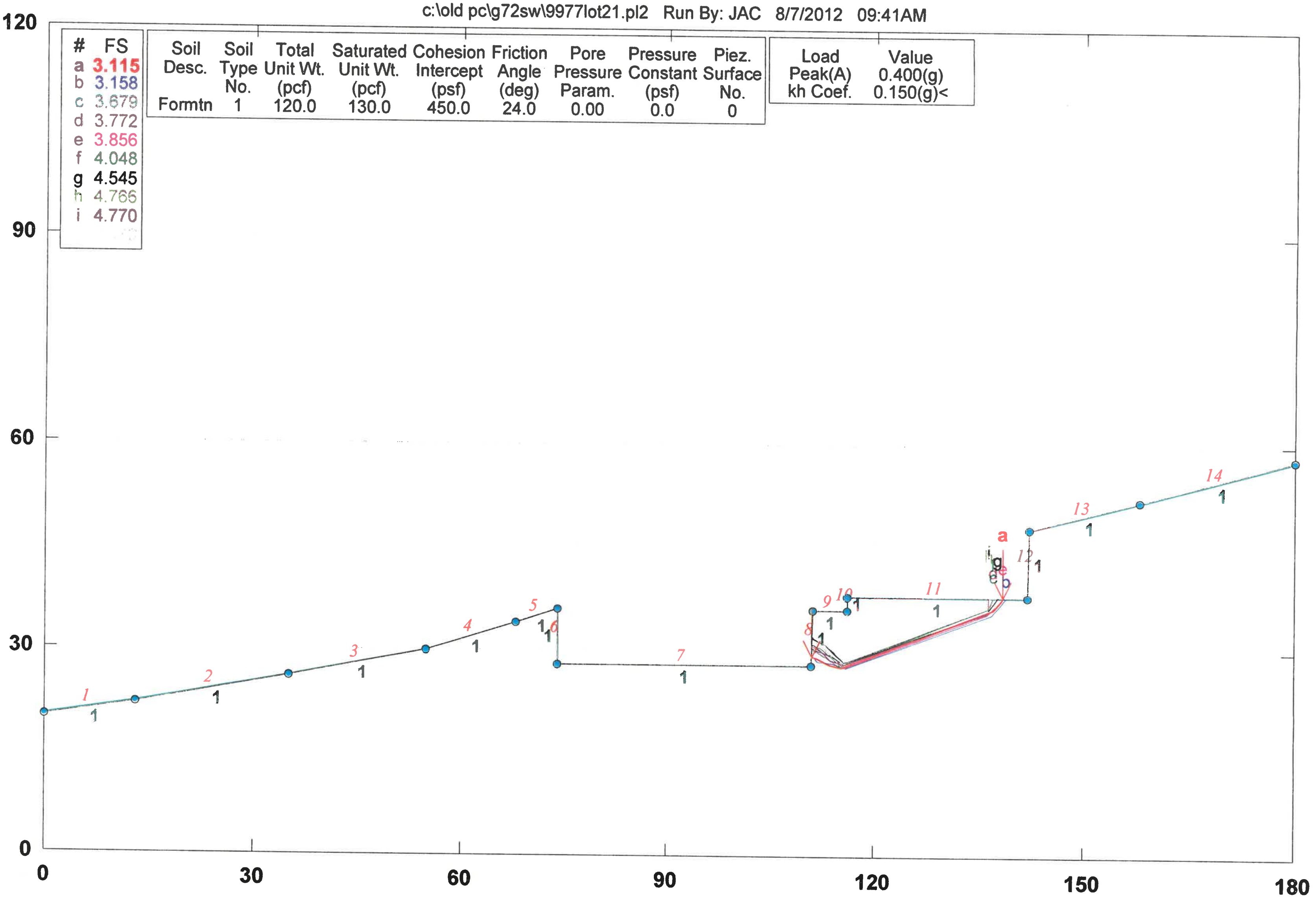

#	FS
a	3.115
b	3.158
c	3.679
d	3.772
e	3.856
f	4.048
g	4.545
h	4.766
i	4.770

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.
Formtn	1	120.0	130.0	450.0	24.0	0.00	0.0	0

Load	Value
Peak(A)	0.400(g)
kh Coef.	0.150(g)<

GSTABL7 v.2 FSmin=3.115

Safety Factors Are Calculated By The Simplified Janbu Method

The Reserve Property, Job No.10-9977 Sectn. Lot 2'

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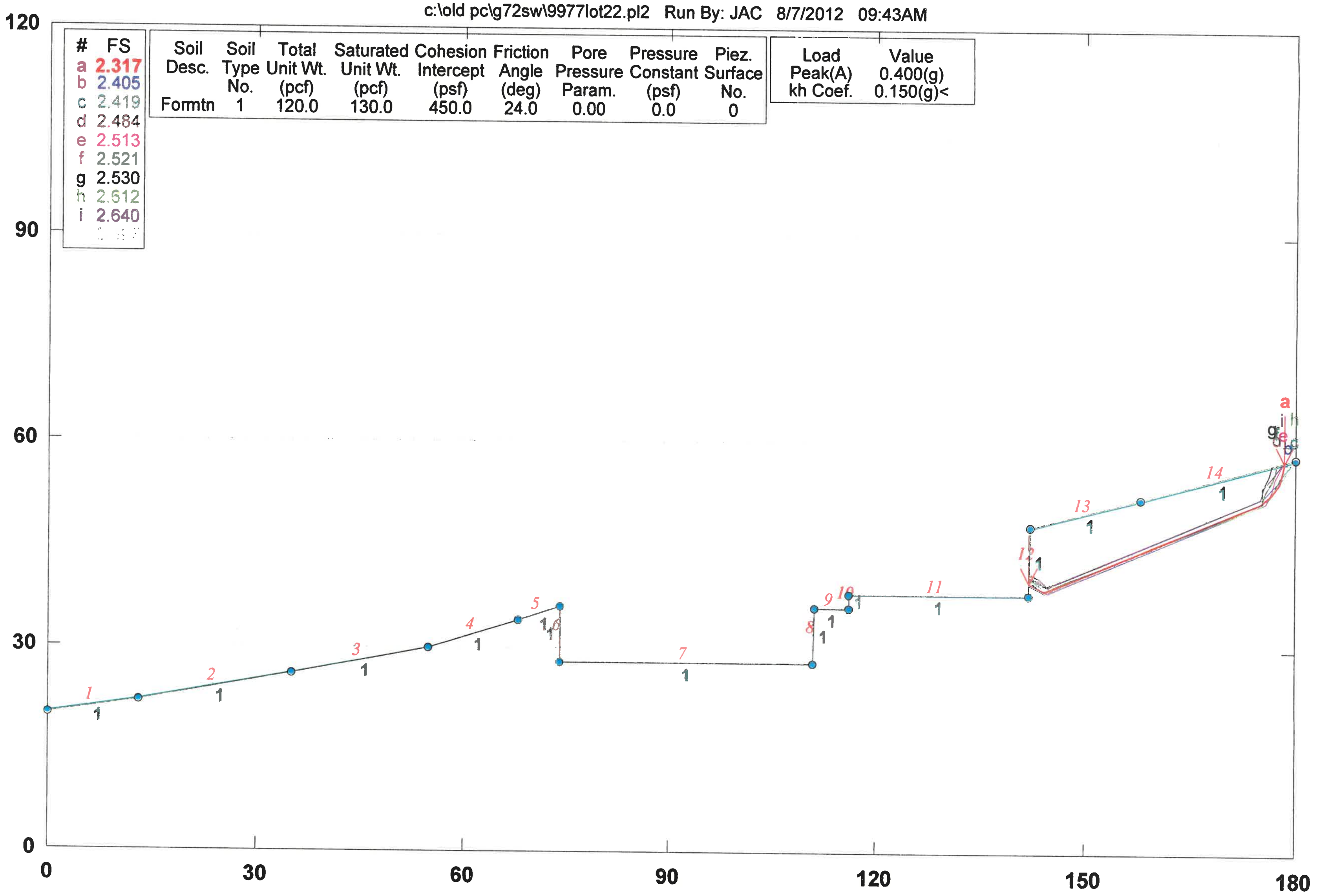


GSTABL7 v.2 FSmin=2.317

Safety Factors Are Calculated By The Simplified Janbu Method

APPENDIX C

USGS Design Map Summary Sheet

USGS Design Maps Summary Report

User–Specified Input

Report Title	The Reserve, La Jolla, CA Tue October 18, 2016 15:28:45 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	32.837°N, 117.2581°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

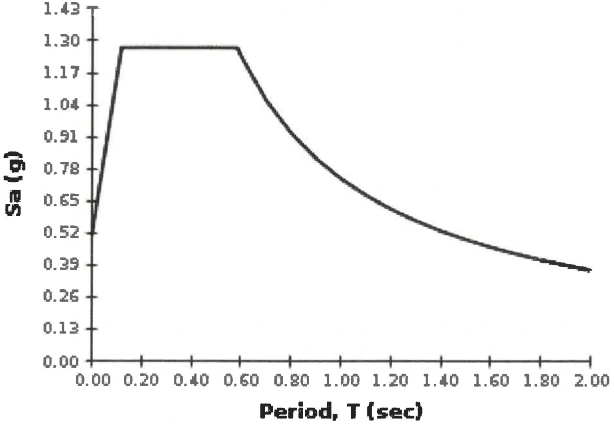
USGS–Provided Output

$S_S =$ 1.272 g $S_{MS} =$ 1.272 g $S_{DS} =$ 0.848 g

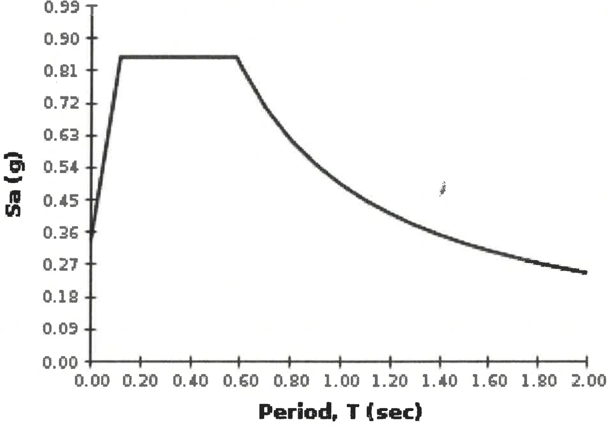
$S_1 =$ 0.491 g $S_{M1} =$ 0.741 g $S_{D1} =$ 0.494 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (32.837°N, 117.2581°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 1.272$ g

From Figure 22-2 [2] $S_1 = 0.491$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.272 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.491 g, F_v = 1.509

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 1.272 = 1.272$ g

---

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.509 \times 0.491 = 0.741$ g

---

Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–3): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 1.272 = 0.848$ g

---

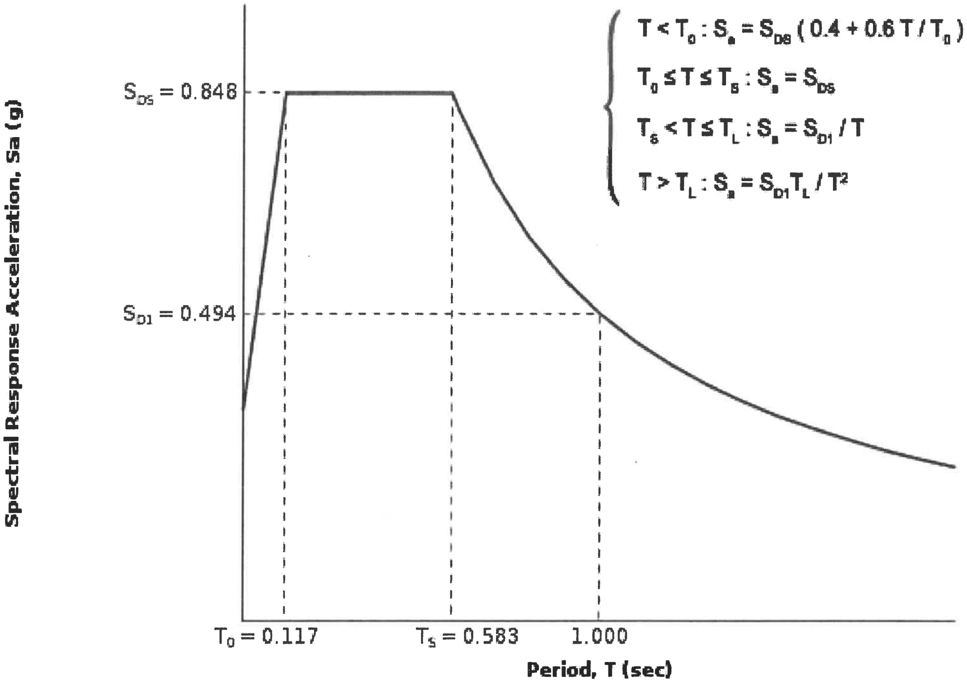
Equation (11.4–4): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 0.741 = 0.494$ g

---

Section 11.4.5 — Design Response Spectrum

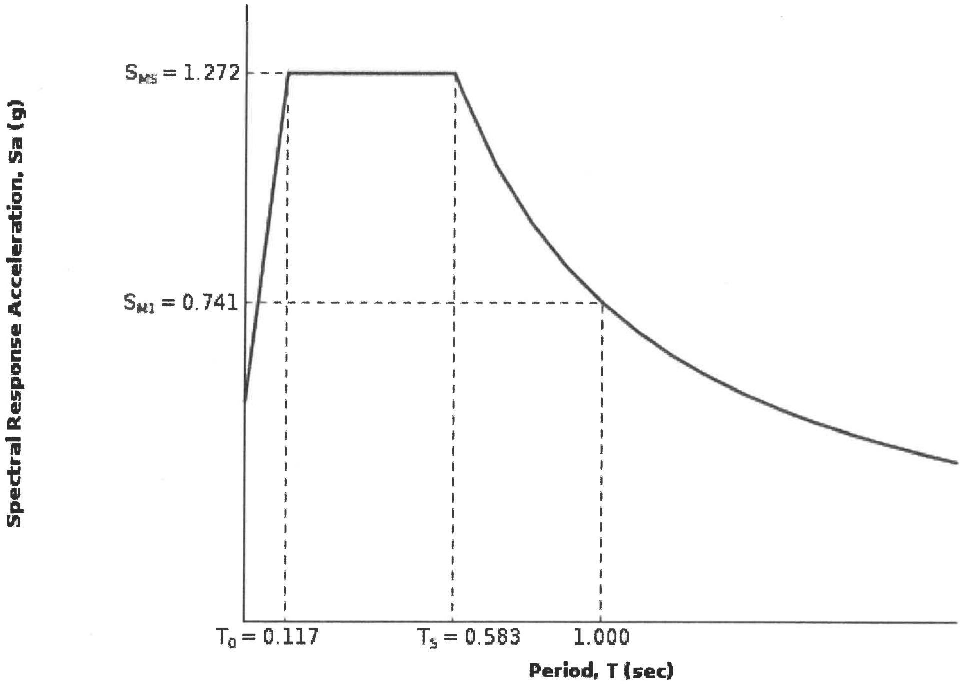
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.571

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.571 = 0.571$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.571 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] $C_{RS} = 0.842$

From Figure 22-18 [6] $C_{R1} = 0.874$

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.848 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.494 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

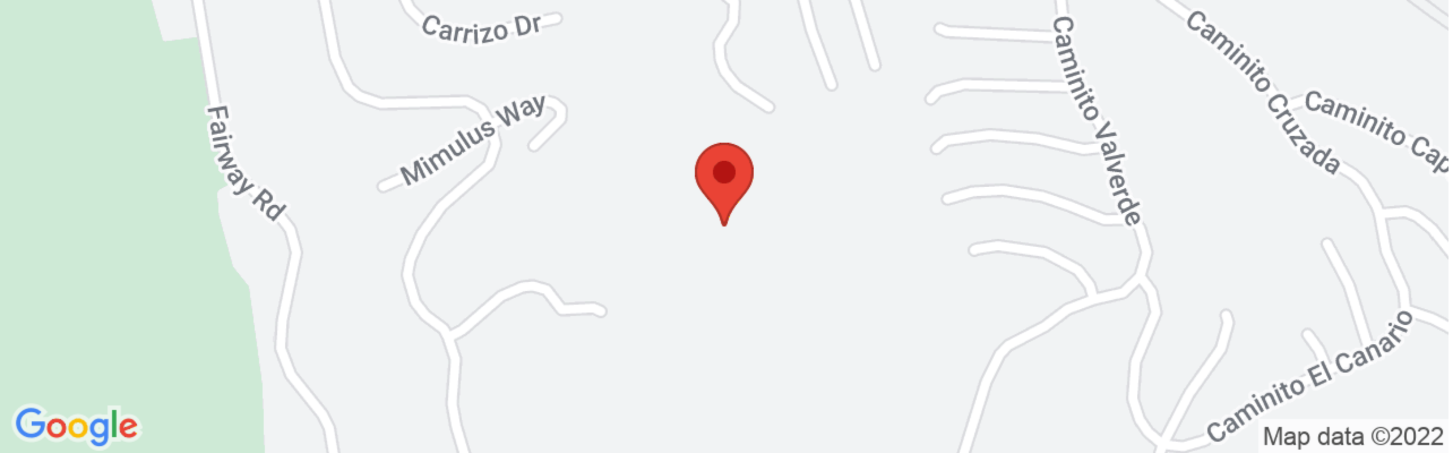
References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

APPENDIX B
UPDATED 10/14/22

6850 Country Club Drive, La Jolla, CA

Latitude, Longitude: 32.8379, -117.2583



Date	10/4/2022, 2:43:21 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	**Value**	**Description**
S_S	1.387	MCE_R ground motion. (for 0.2 second period)
S_1	0.485	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.387	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8 0.880	Site-modified spectral acceleration value
S_{DS}	0.925	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8 0.587	Numeric seismic design value at 1.0 second SA

Type	**Value**	**Description**
SDC	null -See Section 11.4.8 D	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8 1.814	Site amplification factor at 1.0 second
PGA	0.633	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.696	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.387	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.601	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	2.221	Factored deterministic acceleration value. (0.2 second)
S1RT	0.485	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.547	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.782	Factored deterministic acceleration value. (1.0 second)
PGAd	0.921	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.633	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.867	Mapped value of the risk coefficient at short periods

APPENDIX C

Updated Slope Stability Analysis October 2022

APPENDIX C

SLOPE STABILITY CALCULATIONS WITH SLIDE 6 COMPUTER PROGRAM

The Reserve Residential Project

Job No. 10-9977

We performed gross slope stability calculations using the *SLIDE 6* program by Roc Science. The program is a limit equilibrium method, slope stability program that allows the use of several slope stability methods to calculate the factors of safety against shear failure. On this project, the Bishop Simplified method was used as the basis for calculations when using circular slide and block glide surfaces for analysis through the site geologic cross sections.

The program calculates the factor of safety against shear failure for potential slide surfaces over a selected range. We chose the range of slide surfaces where failures are most likely to occur. The printout shows a block with contours of different colors and shades that correspond to the different factors of safety calculated that can be obtained for the analyzed range of slide surfaces for Section A-A′, B-B′, C-C′, D-D′, and E-E′ which include the most unfavorable slope conditions at the site (see attached printouts). The green circular or block surface with the green value displayed in the printout is the lowest possible factor of safety located within the search range of each analysis. Soil strength values, geometry, and water conditions (seepage was not encountered) used in the program were based on geological information at the site, obtained by our project geologist. Direct shear test results from the on-site soils were performed and were used for the gross stability analysis. Shear strength values were conservatively adjusted.

The Bishop Simplified method was used to calculate the global shear failure surfaces and the localized circular shear and block failure surfaces of the existing slope surface. It is our understanding that the plans are preliminary and have not been finalized. Once finalized, we will analyze the slope with its proposed configuration accordingly.

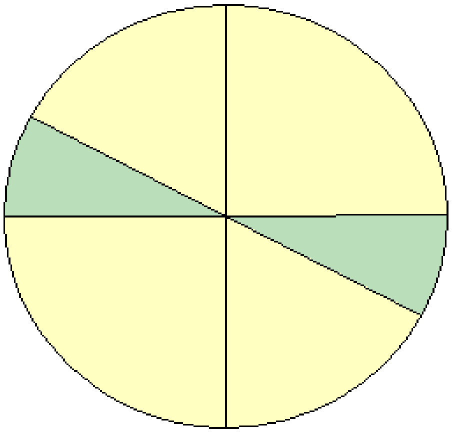
Due to the out of slope bedding encountered at the site for sections A-A′, B-B′, C-C′, and D-D′, we have incorporated the following layer to each analysis.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
BEDDED SCRIPPS-ARDATH (Tsc/Ta)	🟧	125	Generalized Anisotropic			None	0	User Defined 1

The bedded Scripps-Ardath layer consists of two layers in one.

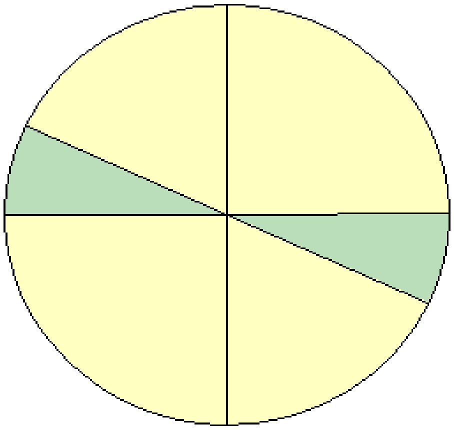
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
STRONG SCRIPPS-ARDATH (Tsc/Ta)	🟨	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	🟩	125	Mohr-Coulomb	125	14	None	0	

Based on the two layers, we assign to the program which angles of dip will have the strong bedding and which angles of dip will be assigned the weak bedding. For the following sections, we have assigned the following bedding to the following sections:



Section A-A’

🟨 90 to 0 degrees:
🟩 0 to -28 degrees:
🟨 -28 to -90 degrees:



Section B-B’

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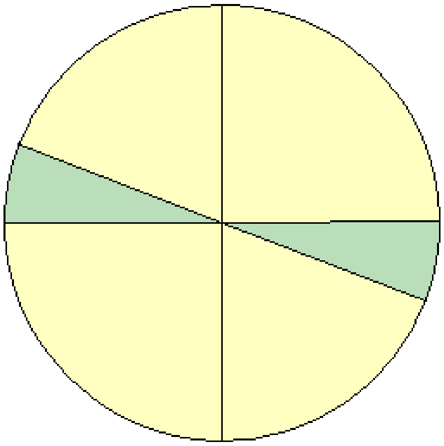
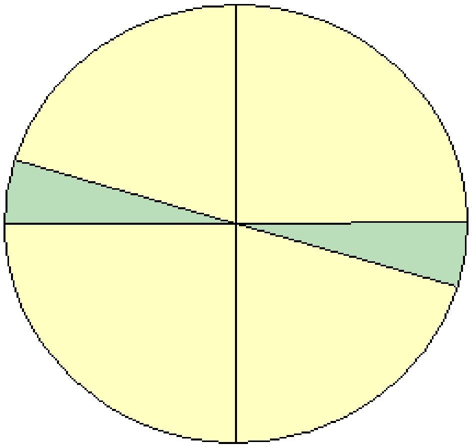
🟩 0 to -25 degrees:
🟨 -25 to -90 degrees:



Section C-C’

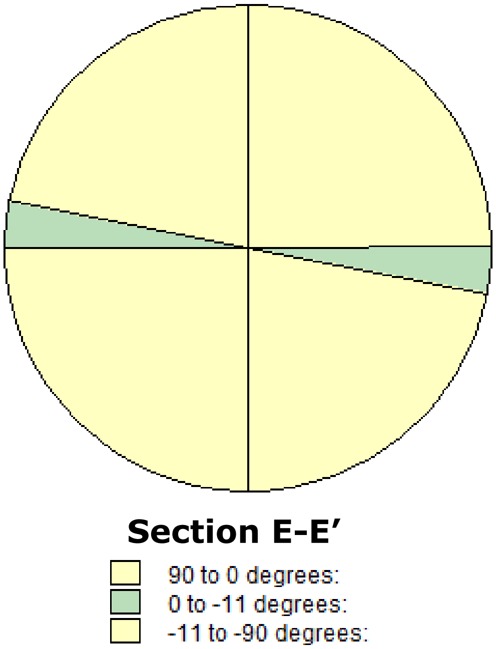
🟨 90 to 0 degrees:
🟩 0 to -21 degrees:
🟨 -21 to -90 degrees:



Section D-D’

🟨 90 to 0 degrees:
🟩 0 to -17 degrees:
🟨 -17 to -90 degrees:

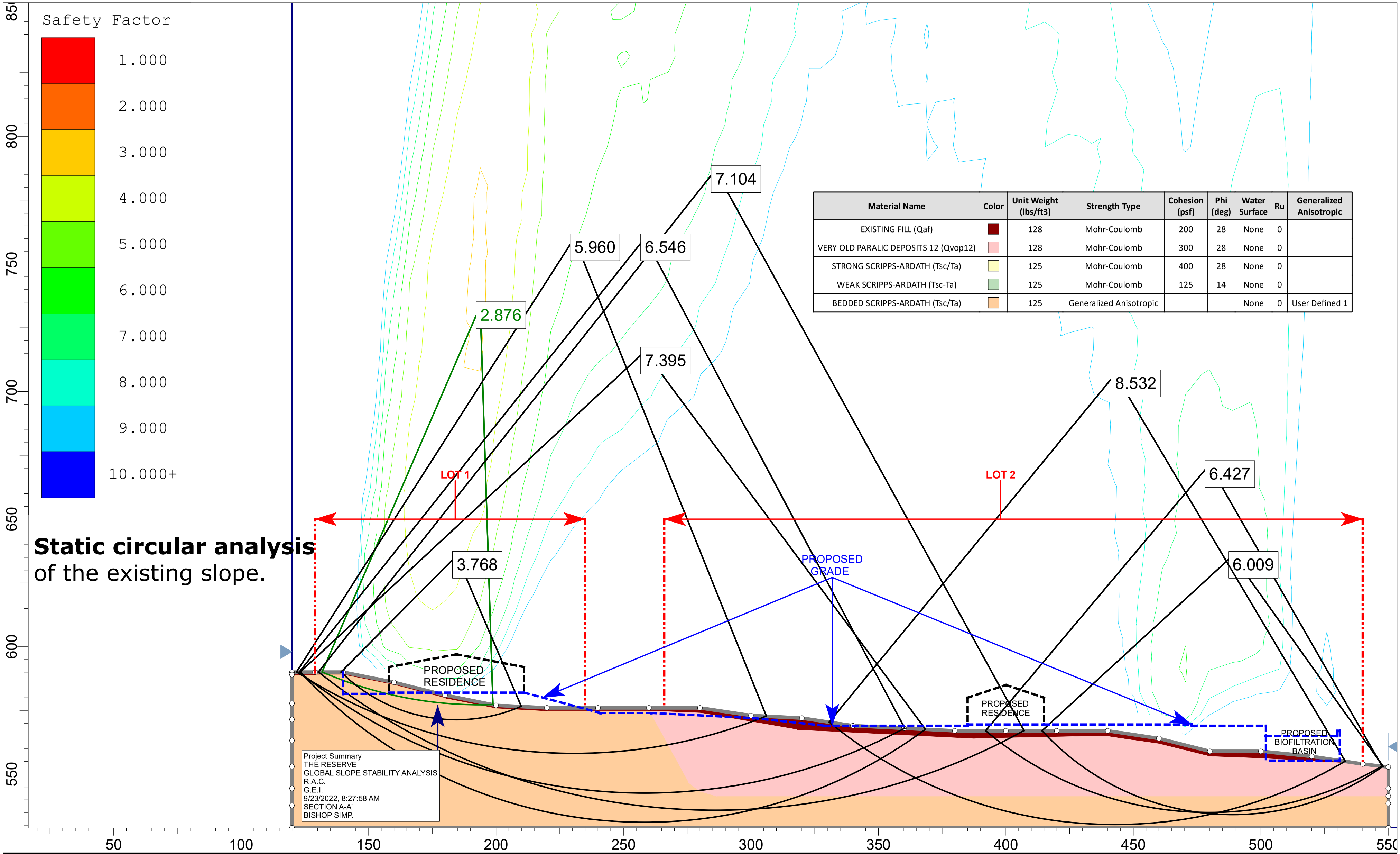
Section E-E’ has favorable bedding into the slope and the analysis has a higher factor of safety compared to the previous sections due to the bedding condition.



The static gross and surficial slope stability factors of safety were calculated and yielded a factor of safety value above 1.5 and greater for circular and block analyses of the existing slope conditions.

Once the static gross stability was determined, a seismic analysis was performed for the same analyzed sections. The seismic analysis yielded a factor of safety value above 1.15 as required by the City of San Diego and the State of California.

The surficial slope stability calculations not performed since as we have previously stated, the proposed grades are preliminary and until the final grade configurations have been determined, we will provide a surficial slope stability analysis.



Static circular analysis of the existing slope.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	🟥	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS 12 (Qvop12)	🟥	128	Mohr-Coulomb	300	28	None	0	
STRONG SCRIPPS-ARDATH (Tsc/Ta)	🟨	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	🟩	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc/Ta)	🟧	125	Generalized Anisotropic			None	0	User Defined 1

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	THE RESERVE		SECTION A-A'
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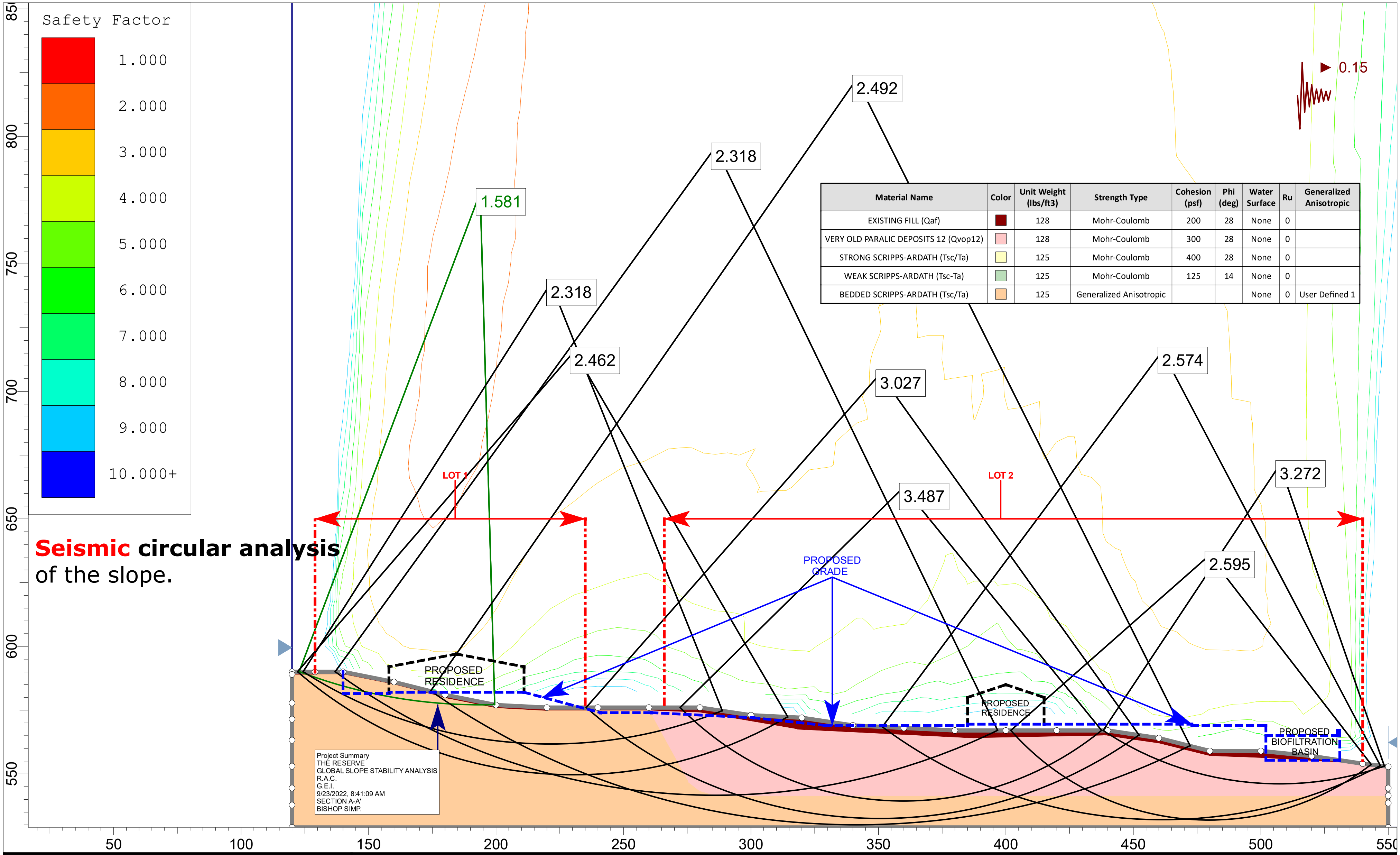

Seismic circular analysis of the slope.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	🟥	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS 12 (Qvop12)	■	128	Mohr-Coulomb	300	28	None	0	
STRONG SCRIPPS-ARDATH (Tsc/Ta)	🟨	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	🟩	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc/Ta)	🟧	125	Generalized Anisotropic			None	0	User Defined 1

Project Summary
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G.E.I.
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SECTION A-A'
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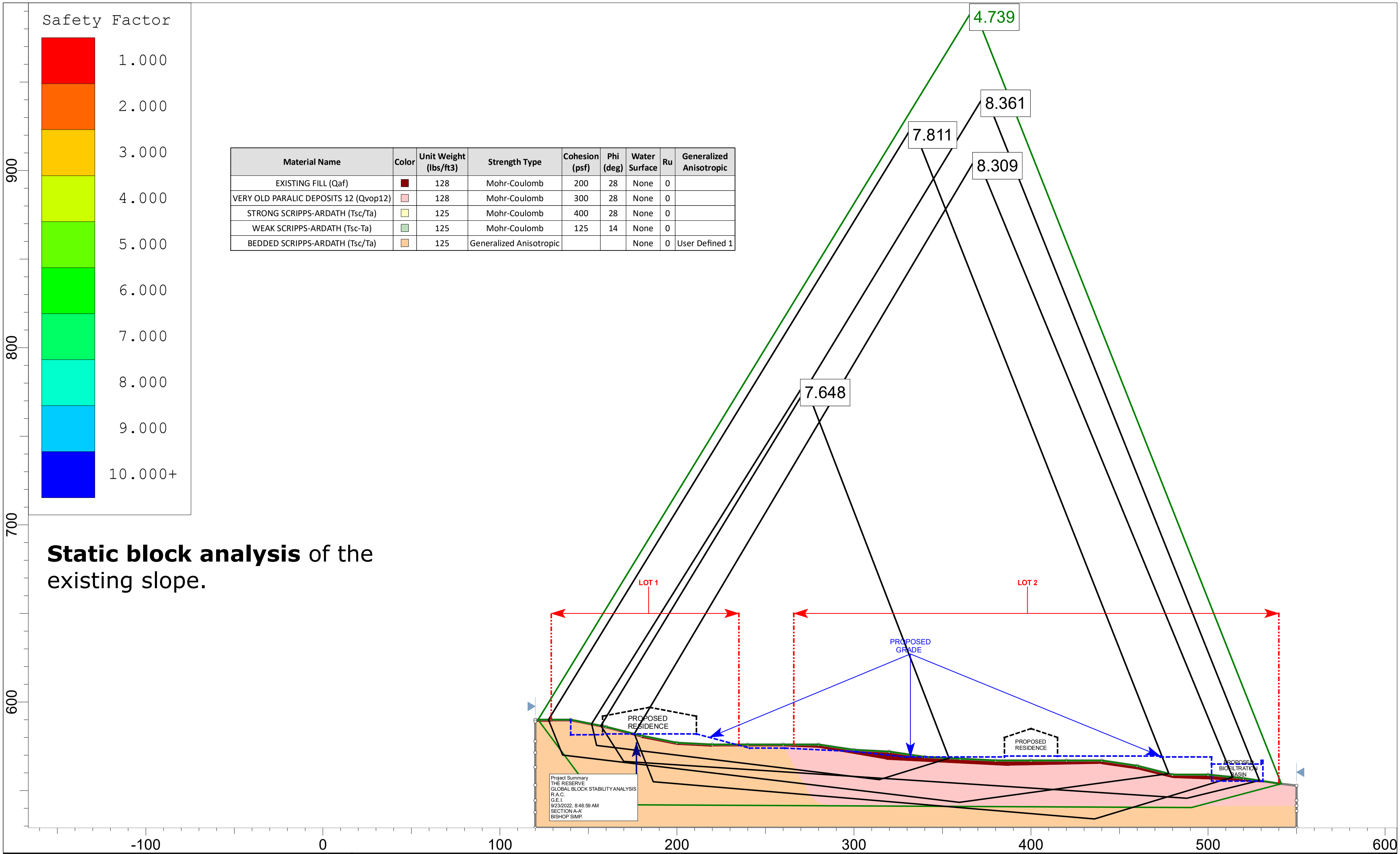

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	🟥	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS 12 (Qvop12)	▨	128	Mohr-Coulomb	300	28	None	0	
STRONG SCRIPPS-ARDATH (Tsc/Ta)	🟨	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	🟩	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc/Ta)	🟧	125	Generalized Anisotropic			None	0	User Defined 1

Project Summary
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Static block analysis of the existing slope.

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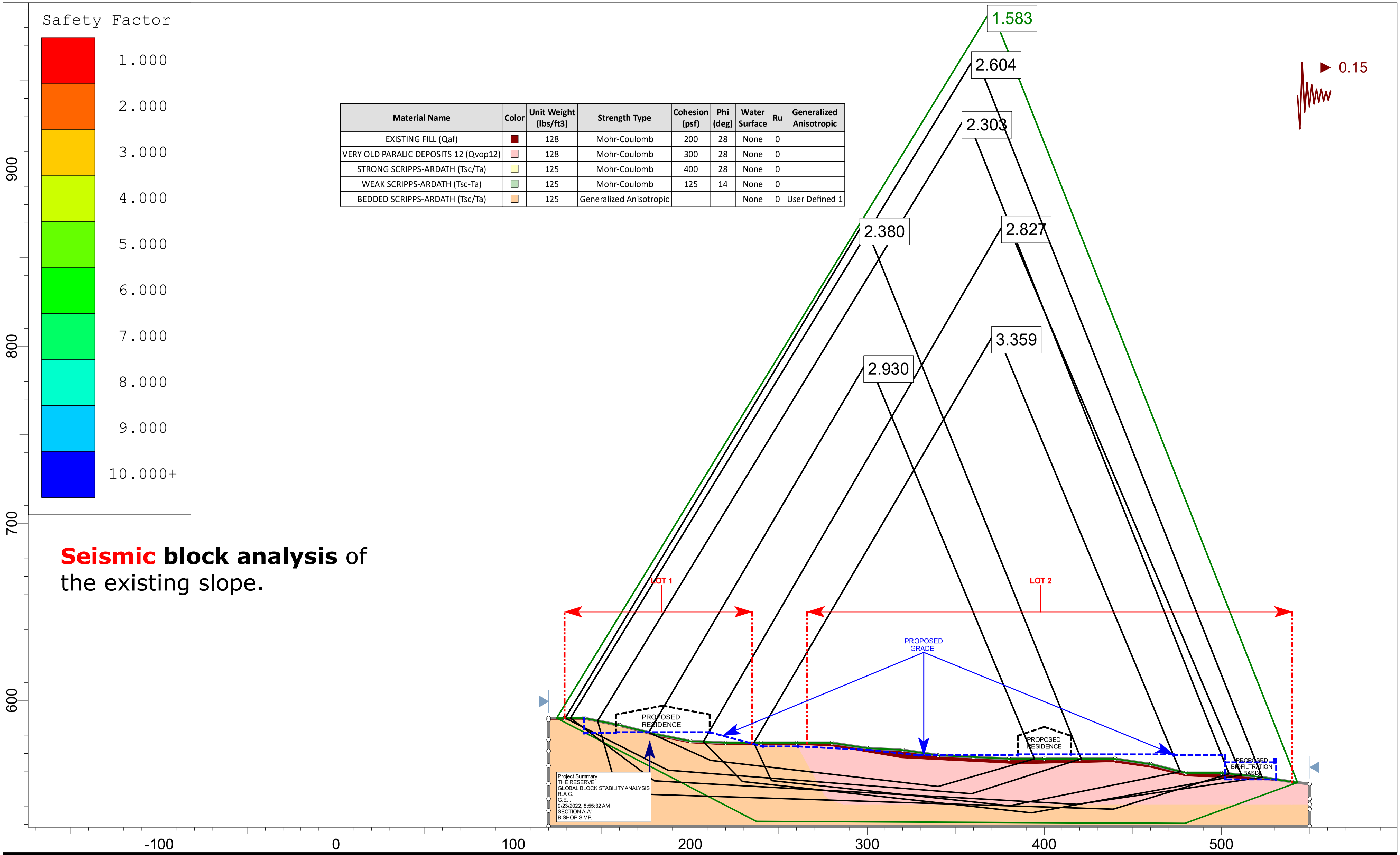

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	■	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS 12 (Qvop12)	■	128	Mohr-Coulomb	300	28	None	0	
STRONG SCRIPPS-ARDATH (Tsc/Ta)	■	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	■	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc/Ta)	■	125	Generalized Anisotropic			None	0	User Defined 1

Seismic block analysis of the existing slope.

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Project THE RESERVE **SECTION A-A'**

Analysis Description GLOBAL BLOCK STABILITY ANALYSIS

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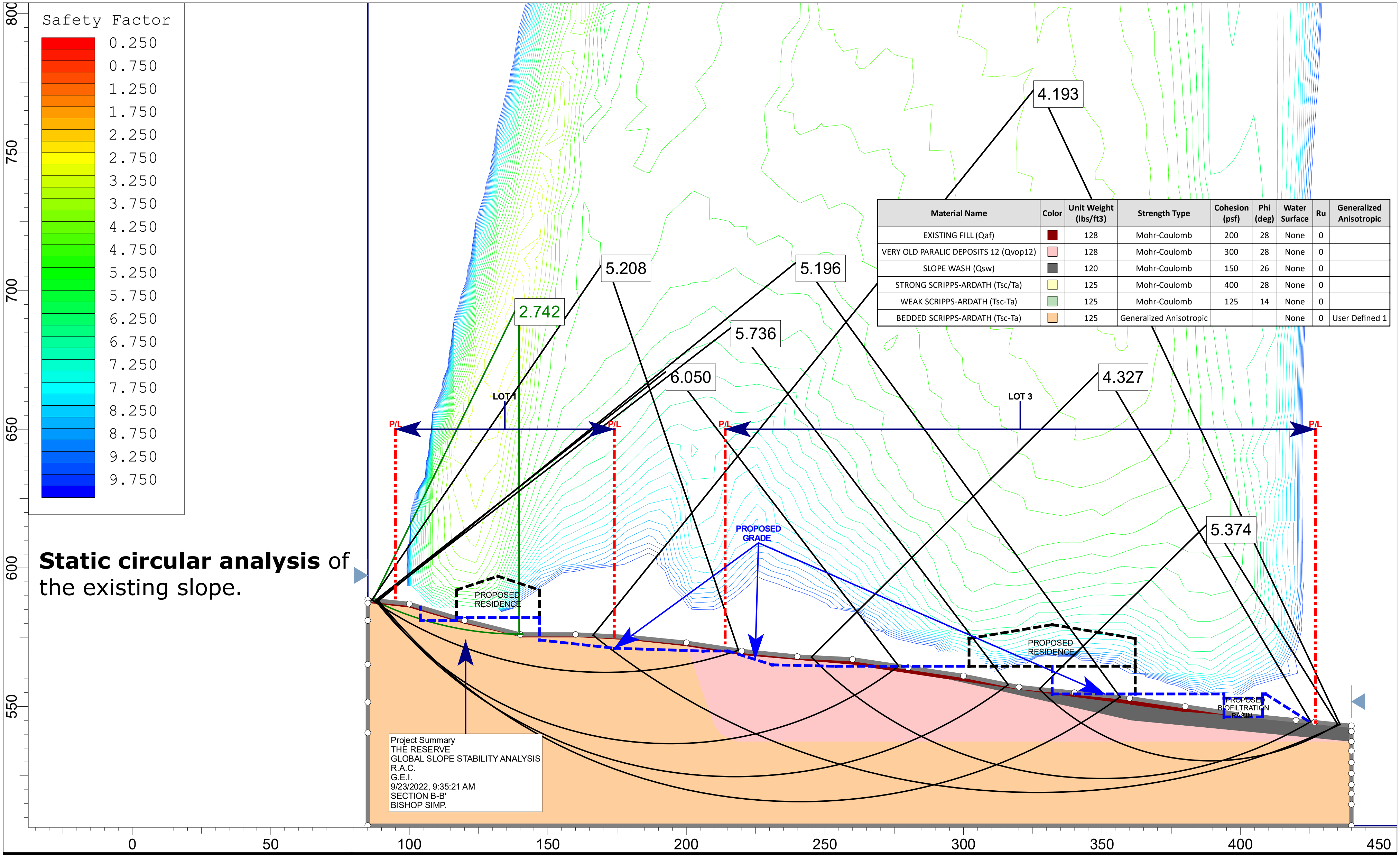

Static circular analysis of the existing slope.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	🟥	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS 12 (Qvop12)	■	128	Mohr-Coulomb	300	28	None	0	
SLOPE WASH (Qsw)	■	120	Mohr-Coulomb	150	26	None	0	
STRONG SCRIPPS-ARDATH (Tsc/Ta)	🟨	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	🟩	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc-Ta)	🟧	125	Generalized Anisotropic			None	0	User Defined 1

Project Summary
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GLOBAL SLOPE STABILITY ANALYSIS
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G.E.I.
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SECTION B-B'
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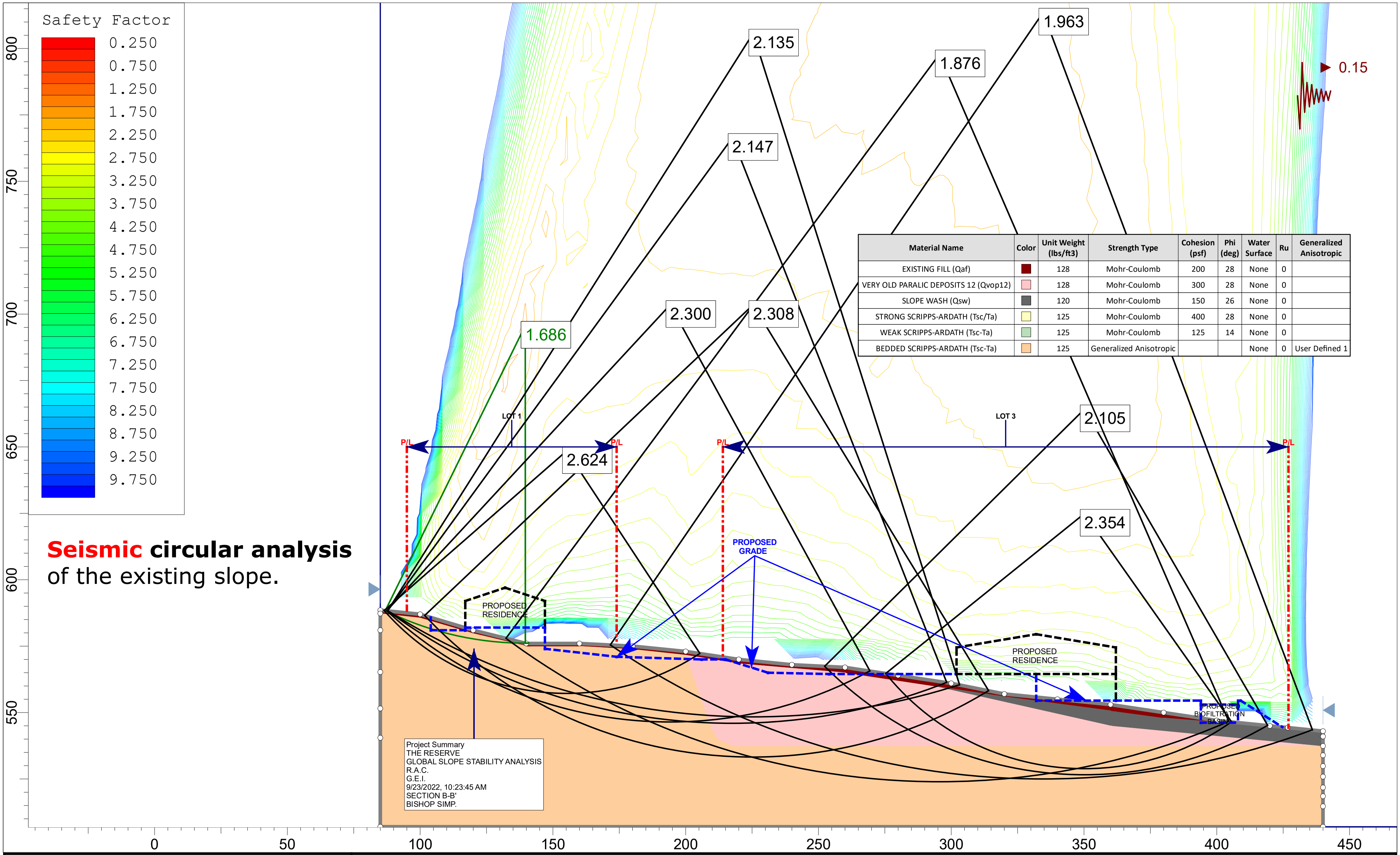

Seismic circular analysis
of the existing slope.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	🟥	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS 12 (Qvop12)	■	128	Mohr-Coulomb	300	28	None	0	
SLOPE WASH (Qsw)	■	120	Mohr-Coulomb	150	26	None	0	
STRONG SCRIPPS-ARDATH (Tsc/Ta)	■	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	🟩	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc-Ta)	🟧	125	Generalized Anisotropic			None	0	User Defined 1

Project Summary
THE RESERVE
GLOBAL SLOPE STABILITY ANALYSIS
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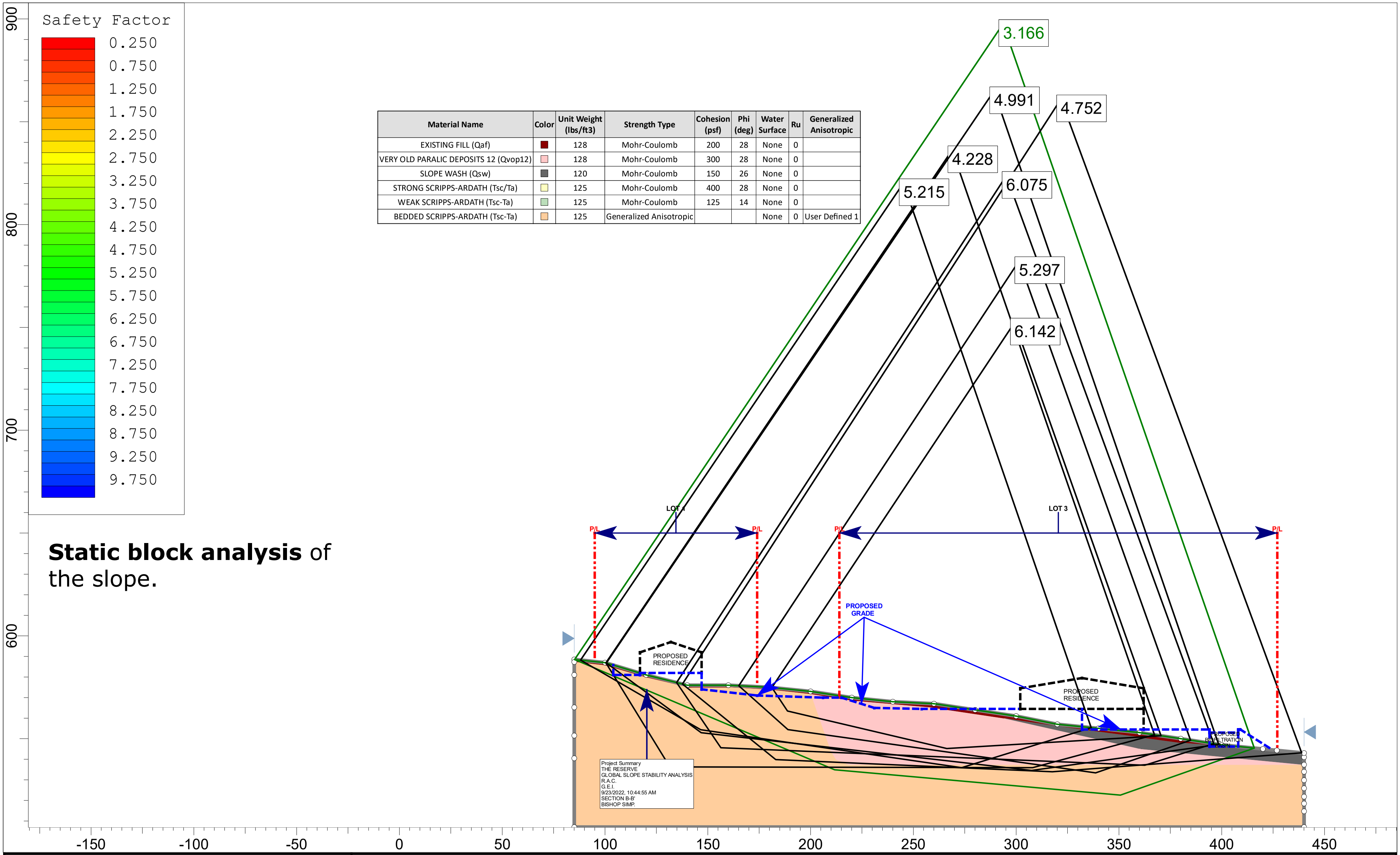

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	■	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS 12 (Qvop12)	□	128	Mohr-Coulomb	300	28	None	0	
SLOPE WASH (Qsw)	■	120	Mohr-Coulomb	150	26	None	0	
STRONG SCRIPPS-ARDATH (Tsc/Ta)	□	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	□	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc-Ta)	□	125	Generalized Anisotropic			None	0	User Defined 1

Static block analysis of the slope.

Project Summary
THE RESERVE
GLOBAL SLOPE STABILITY ANALYSIS
R.A.C.
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9/23/2022, 10:44:55 AM
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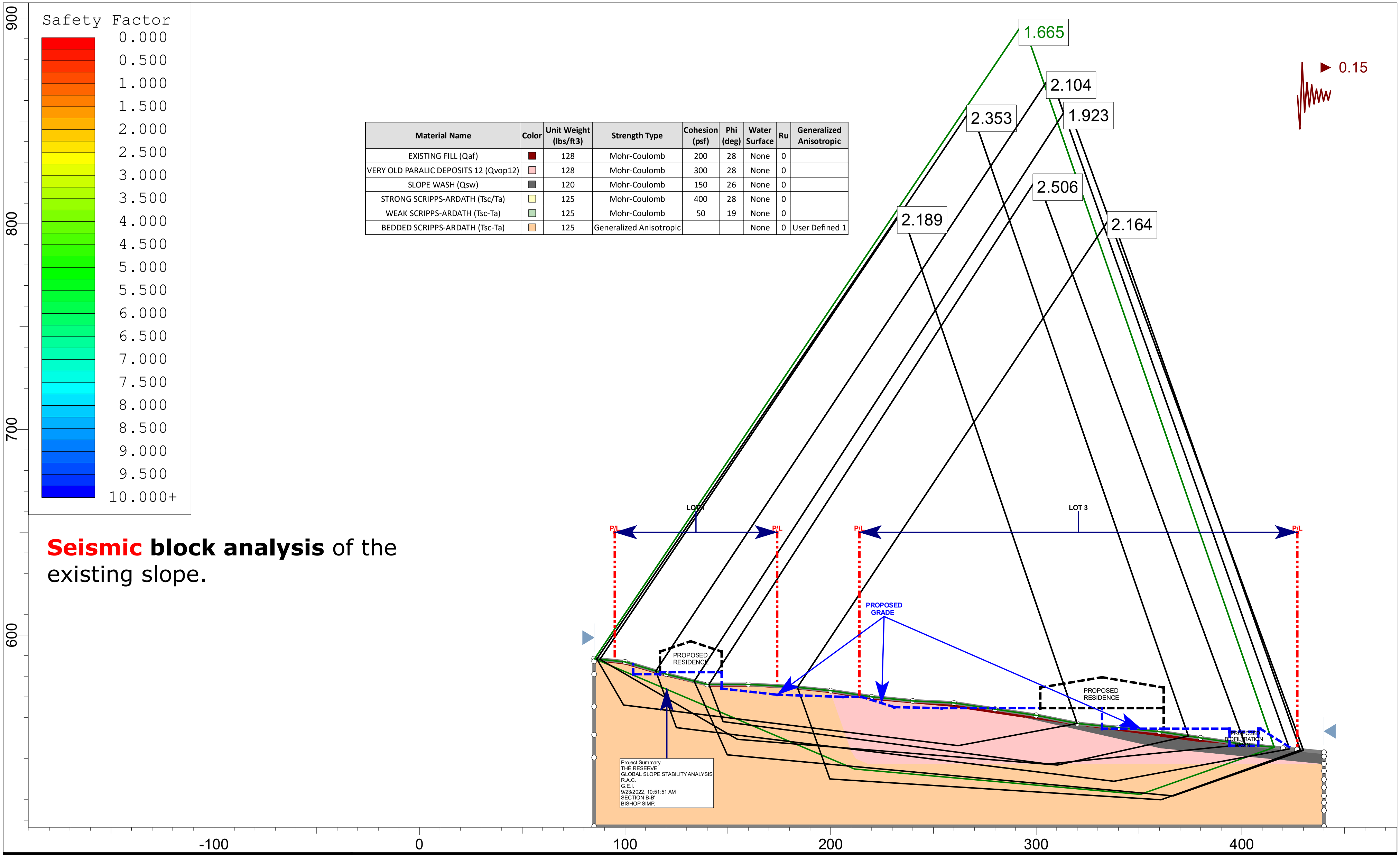
Project THE RESERVE **SECTION B-B'**

Analysis Description GLOBAL SLOPE STABILITY ANALYSIS

Drawn By R.A.C. *Scale* 1:775 *Company* G.E.I.

Date 9/23/2022, 10:44:55 AM *File Name* JOB NO. 10-9977_S(B)_02.slim



Seismic block analysis of the existing slope.

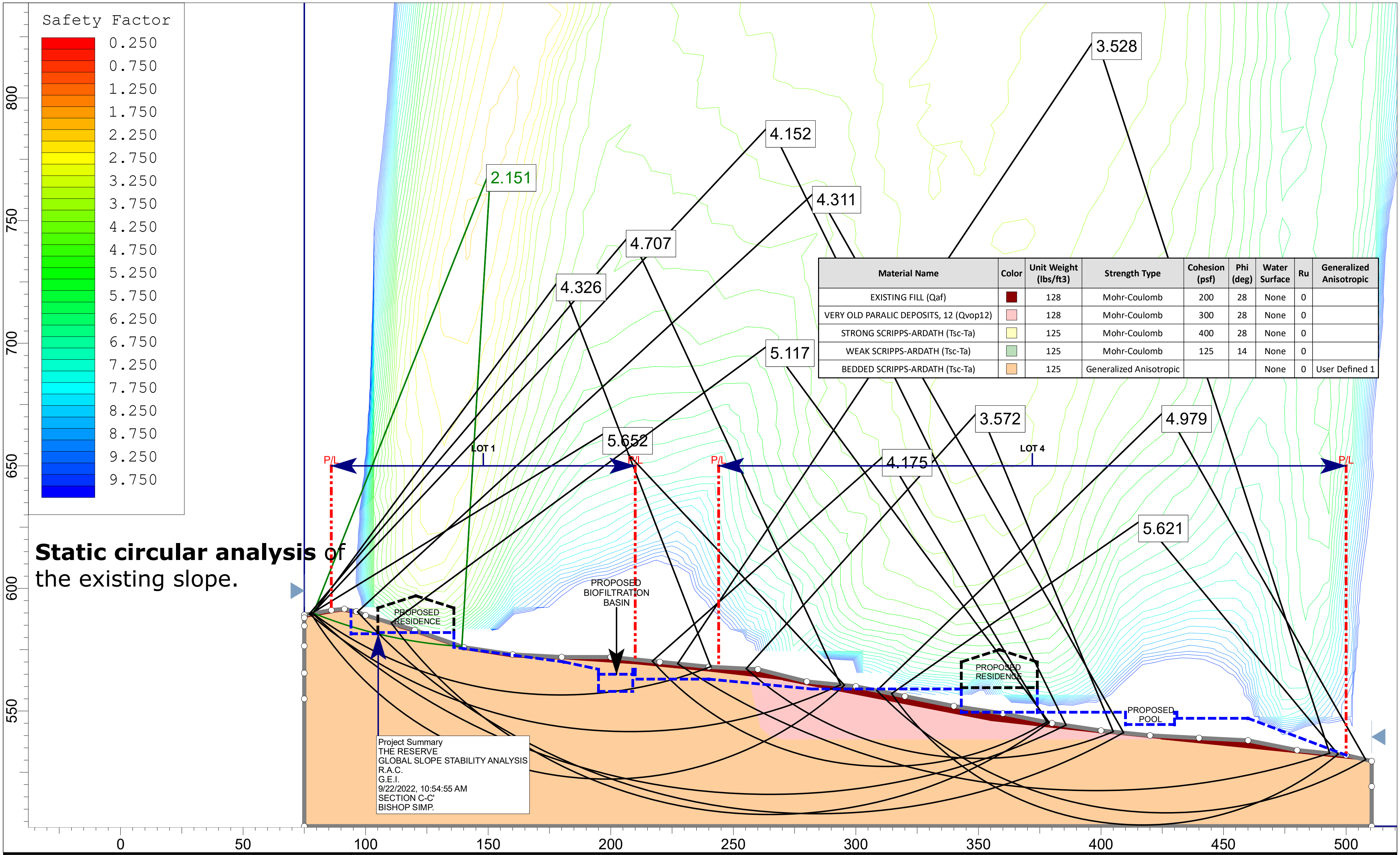
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	■	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS 12 (Qvop12)	■	128	Mohr-Coulomb	300	28	None	0	
SLOPE WASH (Qsw)	■	120	Mohr-Coulomb	150	26	None	0	
STRONG SCRIPPS-ARDATH (Tsc/Ta)	■	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	■	125	Mohr-Coulomb	50	19	None	0	
BEDDED SCRIPPS-ARDATH (Tsc-Ta)	■	125	Generalized Anisotropic			None	0	User Defined 1

Project Summary
THE RESERVE
GLOBAL SLOPE STABILITY ANALYSIS
R.A.C.
G.E.I.
9/23/2022, 10:51:51 AM
SECTION B-B'
BISHOP SIMP.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	THE RESERVE		**SECTION B-B'**
Analysis Description	GLOBAL SLOPE STABILITY ANALYSIS		
Drawn By R.A.C.	*Scale* 1:775	*Company*	G.E.I.
Date	9/23/2022, 10:51:51 AM	*File Name*	JOB NO. 10-9977_S(B)_02w_0.15gSHAKE.slim



Static circular analysis of the existing slope.

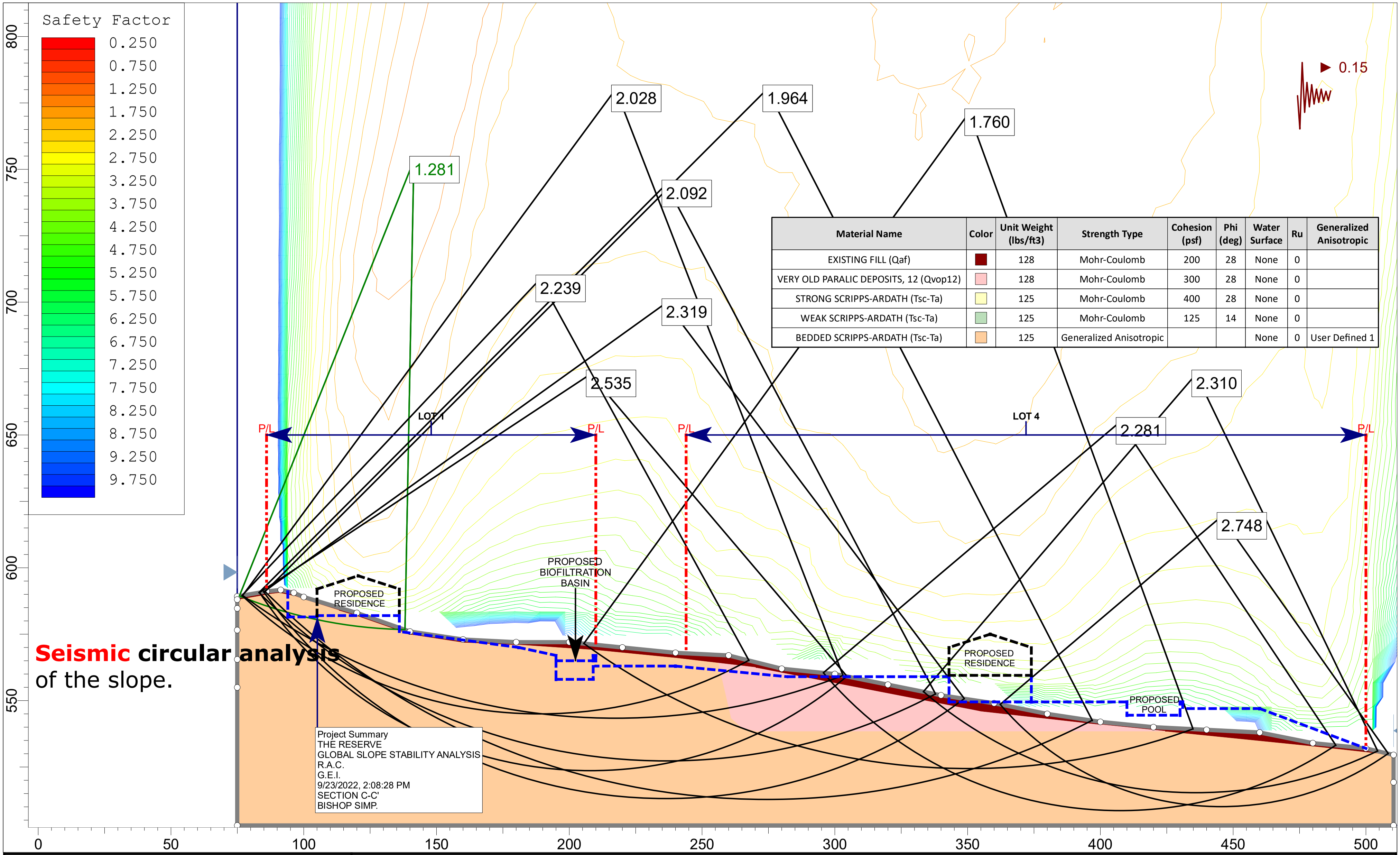
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	🟥	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS, 12 (Qvop12)	🟥	128	Mohr-Coulomb	300	28	None	0	
STRONG SCRIPPS-ARDATH (Tsc-Ta)	🟨	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	🟩	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc-Ta)	🟧	125	Generalized Anisotropic			None	0	User Defined 1

Project Summary
THE RESERVE
GLOBAL SLOPE STABILITY ANALYSIS
R.A.C.
G.E.I.
9/22/2022, 10:54:55 AM
SECTION C-C'
BISHOP SIMP.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project THE RESERVE		**SECTION C-C'**	
Analysis Description GLOBAL SLOPE STABILITY ANALYSIS			
Drawn By R.A.C.	*Scale* 1:650	*Company* G.E.I.	
Date 9/22/2022, 10:54:55 AM		*File Name* JOB NO. 10-9977_S(C)_01.slim	



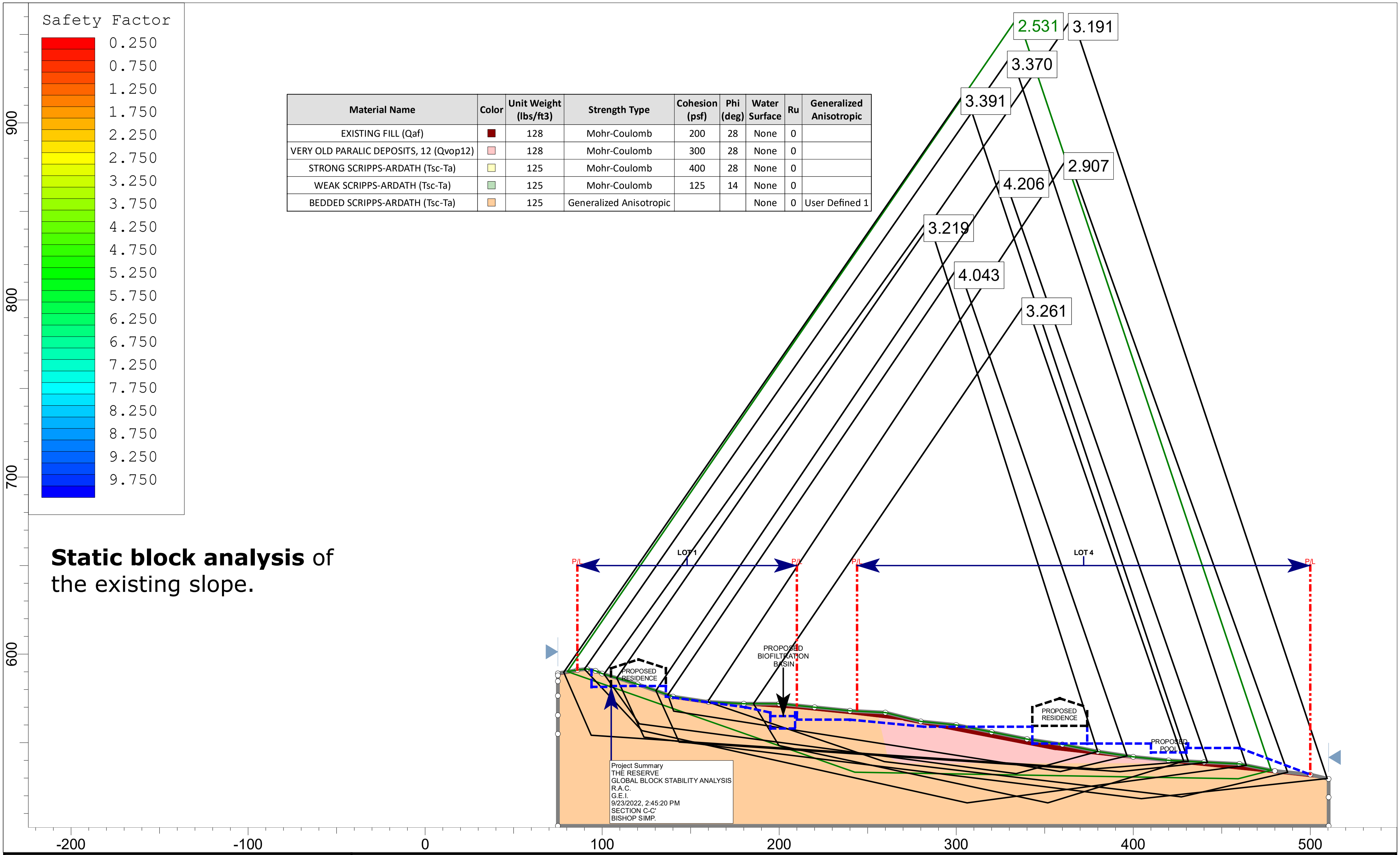
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	🟥	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS, 12 (Qvop12)	🟥	128	Mohr-Coulomb	300	28	None	0	
STRONG SCRIPPS-ARDATH (Tsc-Ta)	🟨	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	🟩	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc-Ta)	🟧	125	Generalized Anisotropic			None	0	User Defined 1

Seismic **circular analysis** of the slope.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	THE RESERVE	**SECTION C-C'**	
Analysis Description	GLOBAL SLOPE STABILITY ANALYSIS		
Drawn By	R.A.C.	*Scale* 1:600	*Company* G.E.I.
Date	9/23/2022, 2:08:28 PM	*File Name*	JOB NO. 10-9977_S(C)_01w_0.15gSHAKE.slim



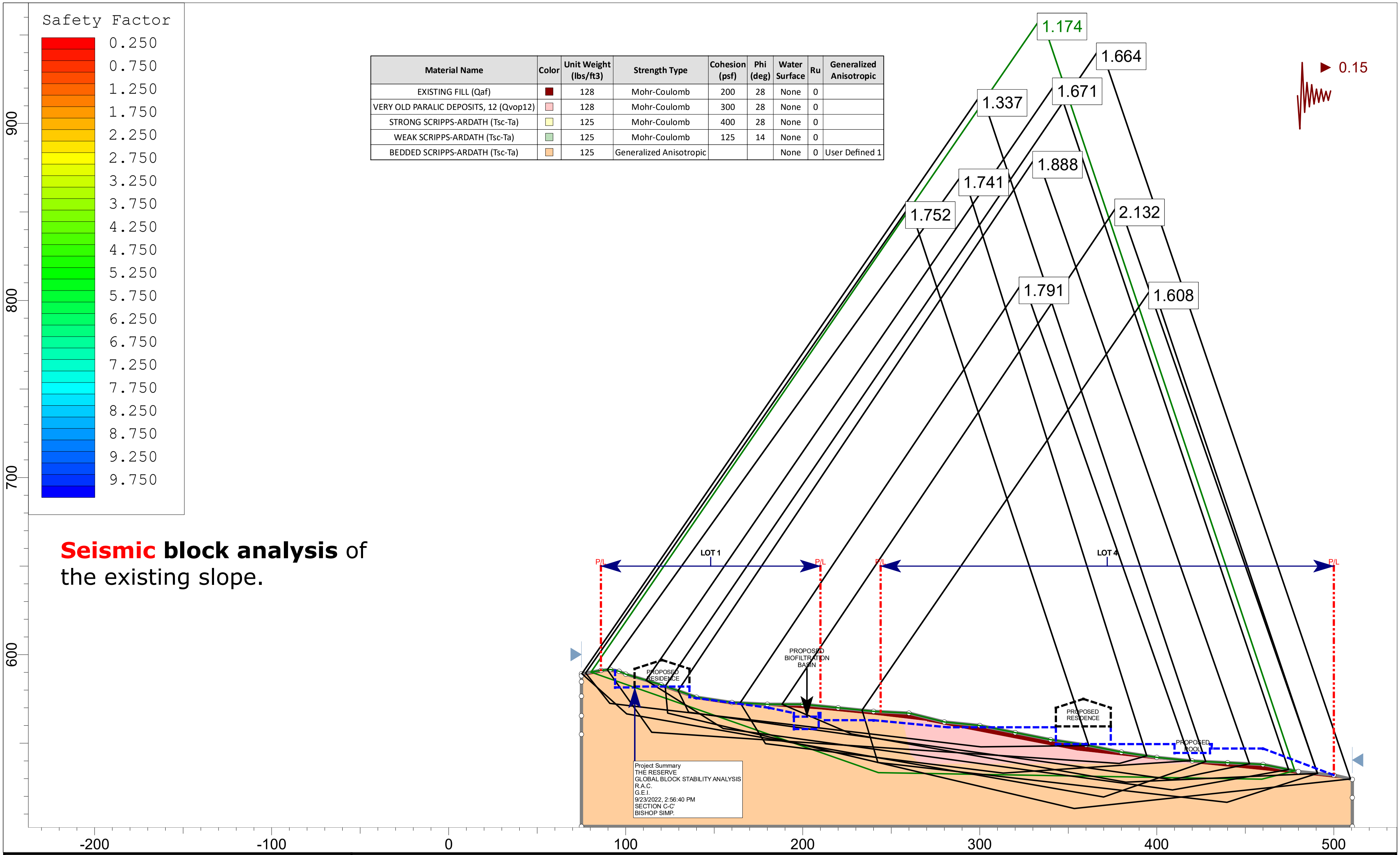
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	■	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS, 12 (Qvop12)	■	128	Mohr-Coulomb	300	28	None	0	
STRONG SCRIPPS-ARDATH (Tsc-Ta)	■	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	■	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc-Ta)	■	125	Generalized Anisotropic			None	0	User Defined 1

Static block analysis of the existing slope.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	THE RESERVE	**SECTION C-C'**	
Analysis Description	GLOBAL BLOCK STABILITY ANALYSIS		
Drawn By	R.A.C.	*Scale* 1:900	*Company* G.E.I.
Date	9/23/2022, 2:45:20 PM	*File Name*	JOB NO. 10-9977_S(C)_02.slim



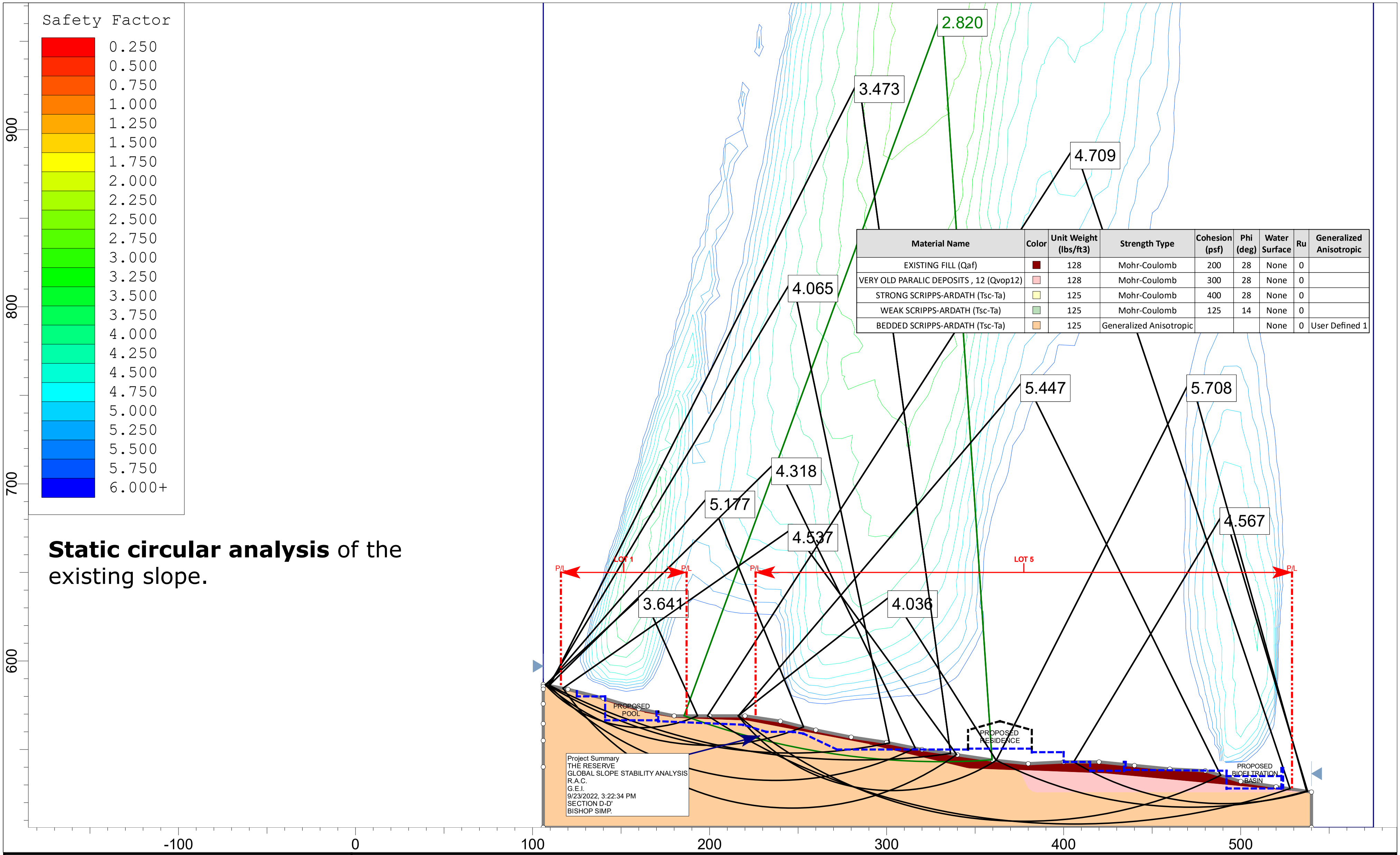
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	■	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS, 12 (Qvop12)	■	128	Mohr-Coulomb	300	28	None	0	
STRONG SCRIPPS-ARDATH (Tsc-Ta)	■	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	■	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc-Ta)	■	125	Generalized Anisotropic			None	0	User Defined 1

Seismic block analysis of the existing slope.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	THE RESERVE		**SECTION C-C'**
Analysis Description	GLOBAL BLOCK STABILITY ANALYSIS		
Drawn By	R.A.C.	*Scale* 1:900	*Company* G.E.I.
Date	9/23/2022, 2:56:40 PM	*File Name*	JOB NO. 10-9977_S(C)_02w_0.15gSHAKE.slim



Project Summary
THE RESERVE
GLOBAL SLOPE STABILITY ANALYSIS
R.A.C.
G.E.I.
9/23/2022, 3:22:34 PM
SECTION D-D'
BISHOP SIMP.

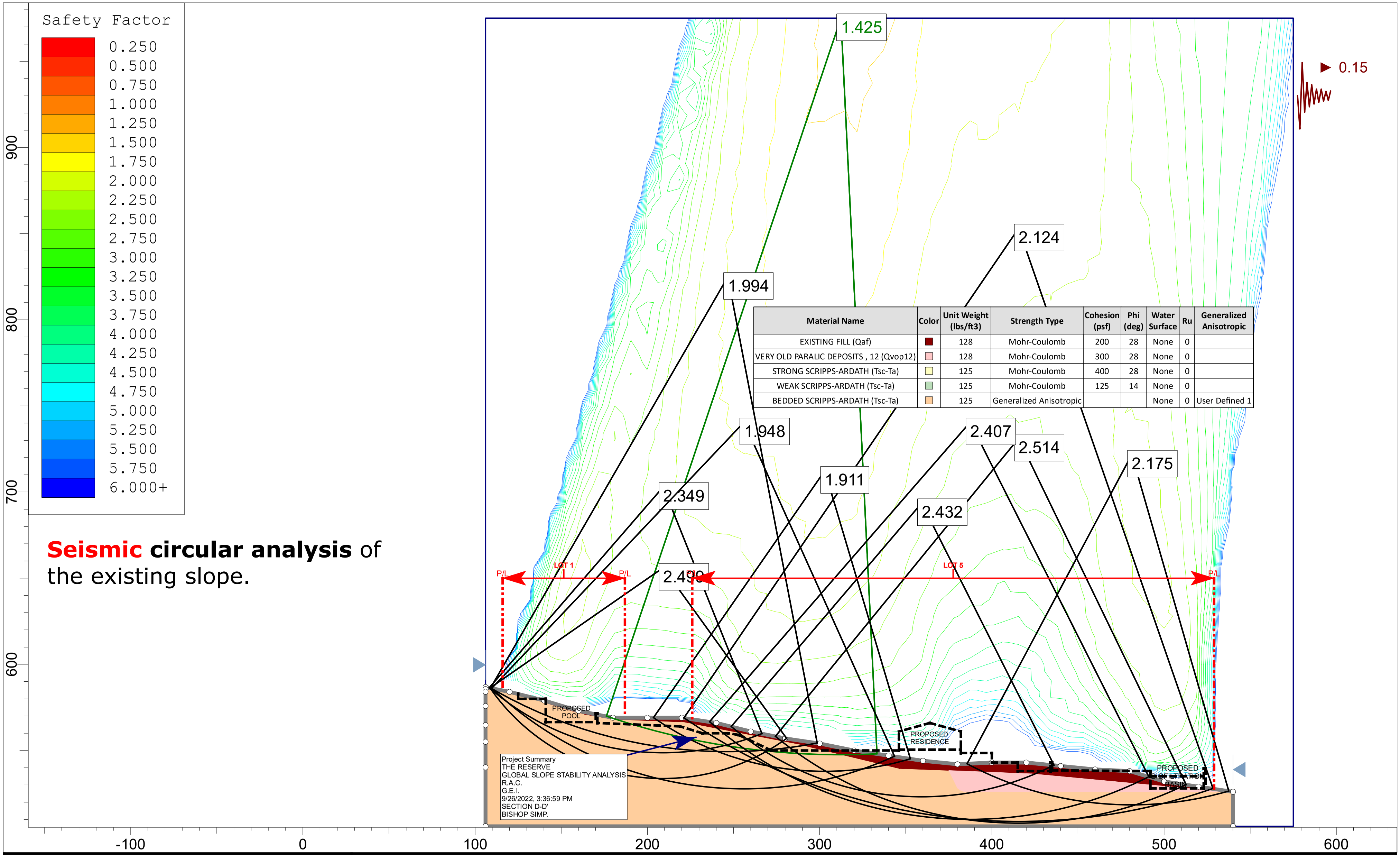
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	■	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS , 12 (Qvop12)	■	128	Mohr-Coulomb	300	28	None	0	
STRONG SCRIPPS-ARDATH (Tsc-Ta)	■	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	■	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc-Ta)	■	125	Generalized Anisotropic			None	0	User Defined 1

Static circular analysis of the existing slope.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	THE RESERVE	**SECTION D-D'**	
Analysis Description	GLOBAL SLOPE STABILITY ANALYSIS		
Drawn By	R.A.C.	*Scale*	1:900
Company	G.E.I.		
Date	9/23/2022, 3:22:34 PM	*File Name*	JOB NO. 10-9977_S(D)_01.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	🟥	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS , 12 (Qvop12)	🟥	128	Mohr-Coulomb	300	28	None	0	
STRONG SCRIPPS-ARDATH (Tsc-Ta)	🟨	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	🟩	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc-Ta)	🟧	125	Generalized Anisotropic			None	0	User Defined 1

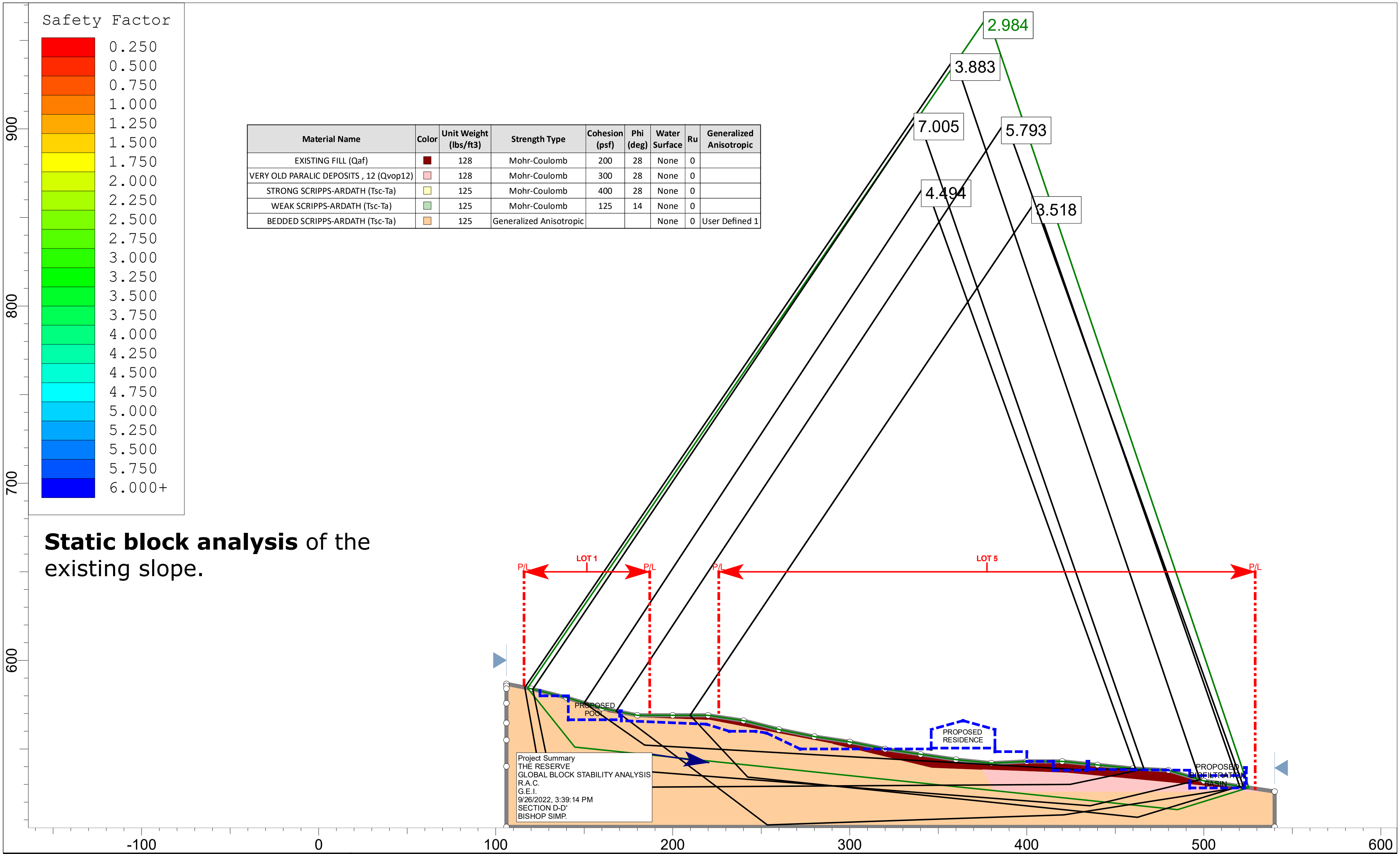
Project Summary
THE RESERVE
GLOBAL SLOPE STABILITY ANALYSIS
R.A.C.
G.E.I.
9/26/2022, 3:36:59 PM
SECTION D-D'
BISHOP SIMP.

Seismic circular analysis of the existing slope.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	THE RESERVE		**SECTION D-D'**
Analysis Description	GLOBAL SLOPE STABILITY ANALYSIS		
Drawn By	R.A.C.	*Scale* 1:925	*Company* G.E.I.
Date	9/26/2022, 3:36:59 PM	*File Name*	JOB NO. 10-9977_S(D)_01w_0.15gSHAKE.slim



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	🟥	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS , 12 (Qvop12)	🟥	128	Mohr-Coulomb	300	28	None	0	
STRONG SCRIPPS-ARDATH (Tsc-Ta)	🟨	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	🟩	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc-Ta)	🟧	125	Generalized Anisotropic			None	0	User Defined 1

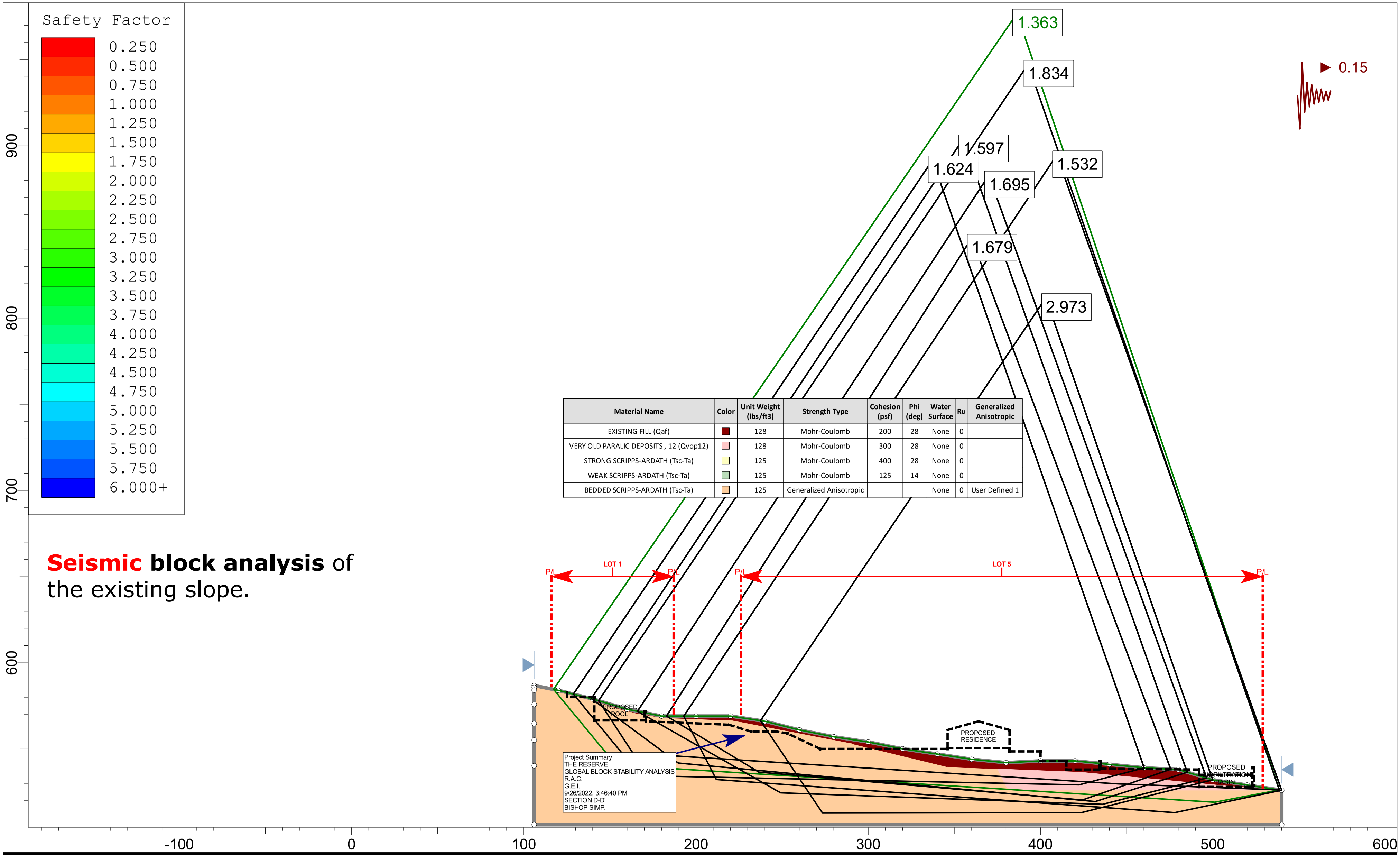
Static block analysis of the existing slope.

Project Summary
THE RESERVE
GLOBAL BLOCK STABILITY ANALYSIS
R.A.C.
G.E.I.
9/26/2022, 3:39:14 PM
SECTION D-D'
BISHOP SIMP.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project THE RESERVE		**SECTION D-D'**	
Analysis Description GLOBAL BLOCK STABILITY ANALYSIS			
Drawn By R.A.C.	*Scale* 1:900	*Company* G.E.I.	
Date 9/26/2022, 3:39:14 PM		*File Name* JOB NO. 10-9977_S(D)_02.slim	



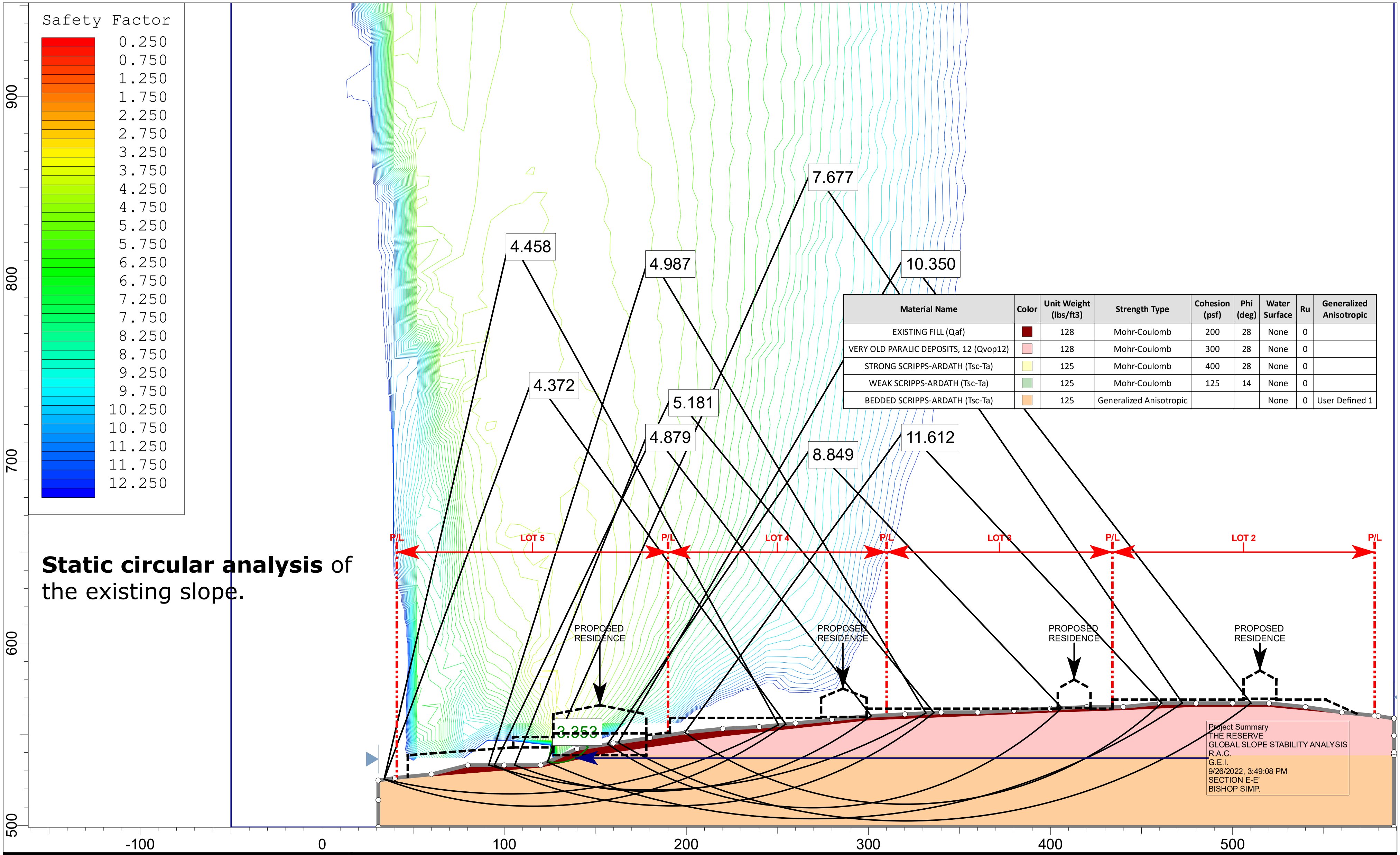
Seismic block analysis of the existing slope.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	🟥	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS , 12 (Qvop12)	🟥	128	Mohr-Coulomb	300	28	None	0	
STRONG SCRIPPS-ARDATH (Tsc-Ta)	🟨	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	🟩	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc-Ta)	🟧	125	Generalized Anisotropic			None	0	User Defined 1

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project THE RESERVE		**SECTION D-D'**	
Analysis Description GLOBAL BLOCK STABILITY ANALYSIS			
Drawn By R.A.C.	*Scale* 1:925	*Company* G.E.I.	
Date 9/26/2022, 3:46:40 PM		*File Name* JOB NO. 10-9977_S(D)_02w_0.15gSHAKE.slim	



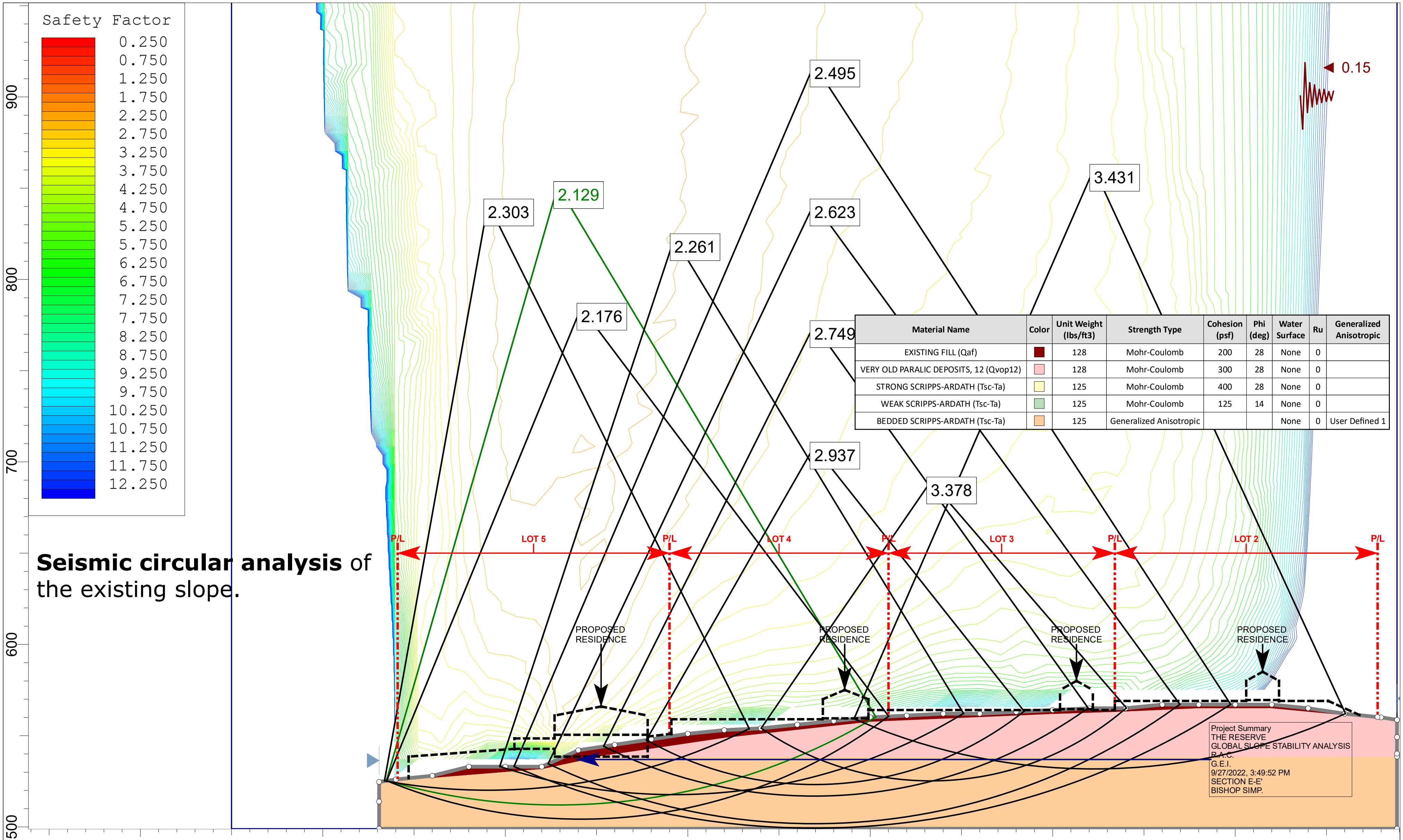
Static circular analysis of the existing slope.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	🟥	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS, 12 (Qvop12)	🟥	128	Mohr-Coulomb	300	28	None	0	
STRONG SCRIPPS-ARDATH (Tsc-Ta)	🟨	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	🟩	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc-Ta)	🟧	125	Generalized Anisotropic			None	0	User Defined 1

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	THE RESERVE	**SECTION E-E'**	
Analysis Description	GLOBAL SLOPE STABILITY ANALYSIS		
Drawn By	R.A.C.	*Scale*	1:875
Company	G.E.I.		
Date	9/26/2022, 3:49:08 PM	*File Name*	JOB NO. 10-9977_S(E)_01.slim



Seismic circular analysis of the existing slope.

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	🟥	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS, 12 (Qvop12)	🟥	128	Mohr-Coulomb	300	28	None	0	
STRONG SCRIPPS-ARDATH (Tsc-Ta)	🟨	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	🟩	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc-Ta)	🟧	125	Generalized Anisotropic			None	0	User Defined 1

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	THE RESERVE **SECTION E-E'**
Analysis Description	GLOBAL SLOPE STABILITY ANALYSIS
Drawn By	R.A.C.
Scale	1:875
Company	G.E.I.
Date	9/27/2022, 3:49:52 PM
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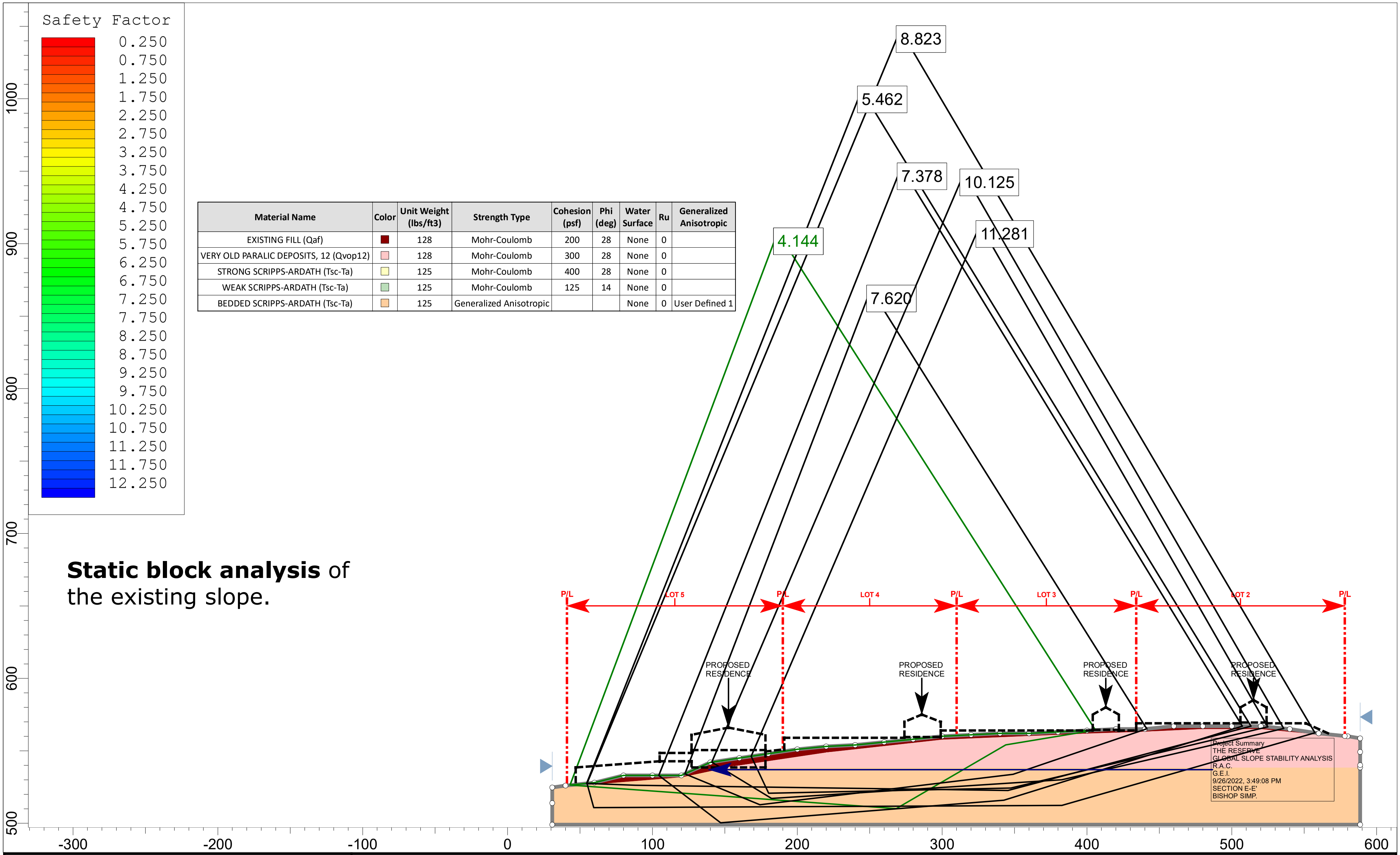


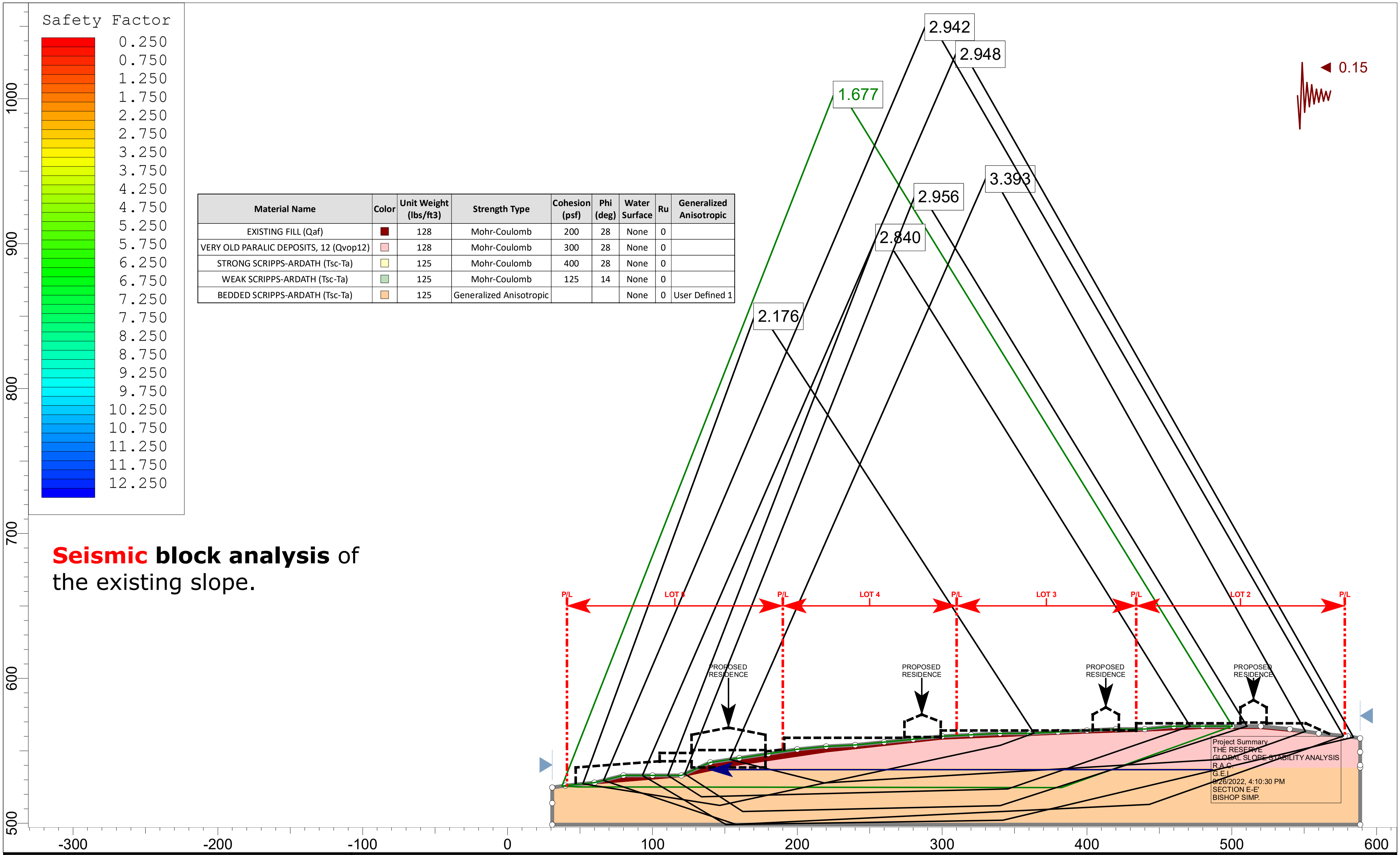
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	🟫	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS, 12 (Qvop12)	🟥	128	Mohr-Coulomb	300	28	None	0	
STRONG SCRIPPS-ARDATH (Tsc-Ta)	🟨	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	🟩	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc-Ta)	🟧	125	Generalized Anisotropic			None	0	User Defined 1

Static block analysis of the existing slope.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project THE RESERVE			**SECTION E-E'**
Analysis Description GLOBAL SLOPE STABILITY ANALYSIS			
Drawn By R.A.C.	*Scale* 1:1100	*Company* G.E.I.	
Date 9/26/2022, 3:49:08 PM		*File Name* JOB NO. 10-9977_S(E)_02.slim	



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Ru	Generalized Anisotropic
EXISTING FILL (Qaf)	🟥	128	Mohr-Coulomb	200	28	None	0	
VERY OLD PARALIC DEPOSITS, 12 (Qvop12)	■	128	Mohr-Coulomb	300	28	None	0	
STRONG SCRIPPS-ARDATH (Tsc-Ta)	■	125	Mohr-Coulomb	400	28	None	0	
WEAK SCRIPPS-ARDATH (Tsc-Ta)	🟩	125	Mohr-Coulomb	125	14	None	0	
BEDDED SCRIPPS-ARDATH (Tsc-Ta)	🟧	125	Generalized Anisotropic			None	0	User Defined 1

Seismic block analysis of the existing slope.

Geotechnical Exploration, Inc.

SLIDEINTERPRET 6.039

Project	THE RESERVE **SECTION E-E'**
Analysis Description	GLOBAL SLOPE STABILITY ANALYSIS

Drawn By	R.A.C.	*Scale*	1:1100	*Company*	G.E.I.
Date	9/26/2022, 4:10:30 PM			*File Name*	JOB NO. 10-9977_S(E)_02w_0.15gSHAKE.slim