UPDATE REPORT OF PRELIMINARY GEOTECHNICAL INVESTIGATION

Proposed Balazs Residential Project
A.P.N. 267-147-06
Artesian Breeze Way
San Diego, California

JOB NO. 21-13629
24 September 2024
Revised 23 January 2025

Prepared for:

Alex and Zsuzsi Balazs

Geotechnical Exploration, Inc.

SOIL AND FOUNDATION ENGINEERING ● GROUNDWATER ● ENGINEERING GEOLOGY

24 September 2024
Revised 23 January 2025

Alex and Zsuzsi Balazs
17306 Eagle Canyon Way
San Diego, CA 92127

Job No. 21-13629

Subject: **Update Report of Preliminary Geotechnical Investigation**
Proposed Balazs Residential Project
A.P.N. 267-147-06
Artesian Breeze Way
San Diego, California

Dear Mr. and Mrs. Balazs:

In accordance with your request, and our proposal of December 17, 2021, ***Geotechnical Exploration, Inc.*** has performed a preliminary geotechnical investigation for the proposed residential project in San Diego, California. The field work was performed on January 13, 2022.

If the conclusions and recommendations presented in this revised report are incorporated into the design and construction of the proposed two-story, single-family residence with a basement level, an attached garage, swimming pool and associated improvements, it is our opinion that the site is suitable for the proposed 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. 21-13629** will expedite a response to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

Jaime A. Cerros, P.E.
Senior Geotechnical Engineer
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Geotechnical Exploration, Inc. presents the following revised report findings and updated soil recommendations in regards to expansive soils, soluble sulfates and chloride testing, peak ground acceleration, and updated building code for the subject project.

I. PROJECT SUMMARY

It is our understanding, based on our review of the available conceptual site plan prepared by Lars Remodeling and Design, dated September 24, 2021, that the proposed project will consist of the construction of a two-story structure with a basement, an attached garage, swimming pool, retaining walls and associated improvements. For the project we utilized the topographic contour lines presented in the site plan. Currently, the property is a vacant undeveloped lot consisting of a northeasterly moderately descending slope. The construction will require cutting into the natural slope and using retaining walls to create split-level building pads. Refer to Figure No. I, Vicinity Map, for property location.

The proposed area of construction is generally located at the southwestern portion of the property with a proposed driveway that descends from the current northern terminus of Artesian Breeze Way. Foundation loads are expected to be typical for this type of construction. At the time of preparation of this report, a grading plan was not available. When final plans are completed, they should be made available for our review. Additional or modified recommendations will be provided at that time if warranted.

Based on the available information at this stage, it is our opinion that the proposed site development would not destabilize neighboring properties or induce the settlement of adjacent structures or improvements if designed and constructed in accordance with our recommendations.

II. SCOPE OF WORK

The scope of work performed for this investigation included a site reconnaissance and subsurface exploration program under the direction of our geologist with placement, logging and sampling of seven (7) backhoe excavated exploratory trenches, review of available published information pertaining to the site geology, laboratory testing, geotechnical engineering analysis of the field and laboratory data, and the preparation of this report. The data obtained and the analyses performed were for the purpose of providing design and construction criteria for the project earthwork, building foundations, retaining walls, slab on-grade floors and associated improvements.

III. SITE DESCRIPTION

The lot is known as Assessor's Parcel No. 267-147-06-00, Parcel 4 of Parcel Map 15646, in the County of San Diego, State of California. The square-shaped property is reported by *SANDAG-SANGIS* as a 2.41-acre lot. The property is located at the northern terminus of Artesian Breeze Way and is bordered on all sides by similar developed and under construction residential properties. Refer to the Plot Plan and Site-Specific Geologic Map, Figure No. II.

The existing property descends from the northern terminus of Artesian Breeze Way at an approximate elevation of +457 feet above Mean Sea Level (MSL) at the southwest corner of the property to +387 feet above MSL near the northeast corner of the property. Vegetation on the site primarily consists of native vegetation and seasonal wild grasses. Rock outcrops occur at several locations on the property. Survey information concerning elevations across the site was obtained from a topographic survey provided on the Concept Site Plan prepared by Lars Remodeling and Design, dated September 24, 2021, and a Topographic Survey prepared by Ciremele Surveying, Inc., dated June 14, 2021.

IV. FIELD INVESTIGATION

The field investigation was conducted on January 13, 2022, and consisted of surface reconnaissance and a subsurface exploration program utilizing a backhoe to investigate and sample the subsurface soils. Seven exploratory trenches (T-1 through T-7) were excavated to depths ranging from 3.25 feet to 6.5 feet in the areas of the proposed construction and associated improvements. The trenches were continuously logged in the field by our geologist and described in accordance with the Unified Soil Classification System (refer to Appendix A). The approximate locations of the trench excavations are shown on the Plot Plan, Figure No. II.

Representative samples were obtained from the exploratory trenches at selected depths appropriate to the investigation. Relatively undisturbed chunk samples and disturbed bulk samples were collected from the exploratory trenches to aid in classification and for appropriate laboratory testing. The samples were returned to our laboratory for evaluation and testing. Exploratory trench logs have been prepared on the basis of our observations and laboratory test results, and are attached as Figure Nos. IIIa-g. The trench logs and related information depict subsurface conditions only at the specific locations shown on the plot plan and on the particular date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at the locations. Also, the passage of time may result in changes in subsurface conditions due to environmental changes.

V. LABORATORY TESTING & SOIL INFORMATION

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-g and IVa-d. The following tests were conducted on representative soil samples:

1. *Moisture Content (ASTM D2216-19)*
2. *Standard Test Method for Bulk Specific Gravity and Density of Compacted Bituminous Mixtures using Coated Samples (ASTM D1188-07)*
3. *Laboratory Compaction Characteristics (ASTM D1557-12e1)*
4. *Determination of Percentage of Particles Smaller than #200 Sieve (ASTM D1140-17)*
5. *Standard Test Method for Expansion Index of Soils (ASTM 4829-11*
6. *Standard Test Method for Direct Shear Test of Soils Under Consolidated Drained Conditions (ASTM D3080-11)*
7. *Water Soluble Sulfate (Department of Transportation California Test 417)*
8. *Water Soluble Chloride (Department of Transportation California Test 422)*

Moisture content and density measurements were performed by ASTM methods D1188-07, the bulk specific gravity utilizing paraffin-coated specimens, to establish the in-situ moisture and density of samples retrieved from the exploratory trenches. This method helps to establish the in-situ density of chunk samples retrieved from the exploratory excavations.

Laboratory compaction values (ASTM D1557-12r21) establish the optimum moisture content and the laboratory maximum dry density of the tested soils. The relationship between the moisture and density of remolded soil samples helps to establish the

relative compaction of the existing site soils and the soil compaction conditions to be anticipated during any future grading operation.
The particle size smaller than a No. 200 sieve analysis (ASTM D1140-17) aids in classifying the tested soils in accordance with the Unified Soil Classification System and provides qualitative information related to engineering characteristics such as expansion potential, permeability, and shear strength.

The expansion potential of soils is determined, when necessary, utilizing the Standard Test Method for Expansion Index of Soils (ASTM D4829-19). In accordance with the Standard (Table 5.3), potentially expansive soils are classified as follows:

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

Expansion index testing of representative samples of the clayey topsoil and weathered formational soil resulted in an expansion index of 133. The topsoils and weathered formational soil are generally considered to have high to very high expansion potential. Expansion index testing of representative samples of the clayey sand bedrock resulted in an expansion index of 31. The more sandy bedrock materials are generally considered to have low expansion potential. The unweathered bedrock materials at depth have very low expansion potential.

A direct shear test (ASTM D3080) was performed on a remolded bedrock/formation sample considered representative of the anticipated bedrock/formation soil conditions in order to evaluate strength characteristics of the existing site soils. The shear test was performed with a constant strain rate direct shear machine. The

specimen tested was saturated and then sheared under various normal loads (refer to Figure No. IVd for the shear test results).
To assess soil corrosivity of the 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 near surface soils most likely to be in contact with concrete and ferrous metals. The tested soils yielded a low resistivity of 8,600 to Ohm-cm, indicating that the soils are mildly corrosive to ferrous metals based on generally adopted corrosion severity ratings.

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 is 5.5 or less. Results of the laboratory testing yielded a pH value of 6.6, indicating that the tested soils are slightly acidic and not a significant factor in soil corrosivity to metals. A corrosion expert should be consulted for recommendations based on the results obtained.

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 10 ppm or 0.001 percent indicating that, at this site, chloride is not a major factor in corrosion to ferrous metals.

The results of water-soluble sulfate testing performed on a representative sample of the near surface soils in the general area of the proposed structure yielded a soluble sulfate content of less than 40 ppm or 0.004 percent, indicating that the proposed cement-concrete structures that are in contact with the underlying soils are anticipated to be affected with a ***negligible*** sulfate exposure.

Results of resistivity, pH, chloride and soluble sulfate tests are presented in the attached Appendix C. It should be noted that ***Geotechnical Exploration Inc***., does not practice corrosion engineering and our test results here should be construed as an aid to the owner or owner's representative. Test results should be evaluated by an engineer specializing in soil corrosivity for any specific design requirement.

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 utilized values for the angle of internal friction and cohesion for those soils that will provide significant lateral support or bearing functions on the project. These values have been utilized in assigning the recommended bearing values as well as active and passive earth pressure design criteria for proposed foundations, retaining walls.

VI. 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, metasedimentary 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). For an extended discussion of regional geology, refer to Appendix D.

VII. SITE-SPECIFIC SOIL & GEOLOGIC DESCRIPTION

Our field investigation, reconnaissance and review of the geologic map by Kennedy and Tan, 2007, *Geologic Map of the Oceanside 30'x60' Quadrangle, California* indicate that the site is underlain by bedrock consisting of Mesozoic, undivided metasedimentary and metavolcanic rocks with a wide variety of unmetamorphosed and low- to high-metamorphic grade volcanic and sedimentary rocks. They include prebatholithic (metamorphosed) and synbatholithic (un-metamorphosed) rocks including metavolcanic rocks, metasedimentary rocks, volcanic, metavolcanic, sedimentary and metasedimentary rocks.

An excerpt of the geological map is included as Figure No. V. Our exploratory trenches indicate the bedrock materials are overlain across the site by up to 3 feet of topsoil and weathered bedrock materials. Site-specific geology is mapped on Figure No. II, Plot Plan and Site-Specific Geologic Map.

Topsoil (Qts): Our site investigation indicates that the areas in the general vicinity of our exploratory trenches are overlain by 1 to 1.5 feet of loose to medium dense topsoil consisting of moist, gray-brown sandy clay, with variable amounts of roots and rock fragments. About 2 feet of loose fill soil was placed on the topsoil at the location of T-7 near the southwest corner of the lot. The topsoils are generally soft. In our opinion, due to the variable density and poor condition of the topsoil, it is not considered suitable in its current condition to support loads from structures or additional fill. Refer to Figure Nos. IIIa-g for details.

Weathered Bedrock/Bedrock (Mzu): The encountered natural soils were observed to be clayey sand bedrock materials. These bedrock materials were observed to be weathered and more clayey in the upper 1 to 2 feet. These materials are described in the literature as Mesozoic, undivided metasedimentary and metavolcanic rocks. The bedrock materials were encountered in all trenches underlying the topsoil at depths ranging from 6 inches to 1.5 feet. The encountered bedrock materials consist of dense to very dense (hard), fractured, dry yellow brown and gray-brown metamorphic rock. The encountered bedrock was observed to be weathered in the upper 1 to 2 feet. The weathered bedrock material consists of very moist, firm to stiff, red-brown sandy clay with rock fragments, gravel and cobbles. In our opinion, the dense to very dense (hard) nature of the less weathered bedrock material below an approximate depth of 1 to 2.5 feet, makes it suitable in its current condition to support loads from structures or additional fill. The upper weathered bedrock is not considered suitable without regrading and blending due to its highly expansive nature. Refer to Figure Nos. IIIa-g for details.

VIII. GEOLOGIC HAZARDS

The following is a discussion of the geologic conditions and hazards common to this area of San Diego, as well as project-specific geologic information relating to development of the subject property.

A. Local and Regional Faults

Reference to the Geologic Map and Legend, Figure No. V (Kennedy and Tan, 2007), indicates that no faults are shown to cross the site. Furthermore, our site reconnaissance presented no indications of faulting across the site. In our professional opinion, neither an active fault nor a potentially active fault underlies the site. A brief description of nearby active faults, including distances from the mapped fault to the subject site at the closest point (based on the USGS Earthquake Hazards-Interactive Fault Map), is presented below:

- *Newport-Inglewood-Rose Canyon Fault Zone System*: The Rose Canyon portion of the Newport-Inglewood-Rose Canyon Fault Zone is mapped approximately 10 miles west-southwest of the site and the offshore portion (Newport-Inglewood) is mapped approximately 19 miles west-northwest of the site. This fault is estimated to be capable of generating an earthquake of M6.9 (EERI, 2021).

- *Coronado Bank Fault*: The Coronado Bank Fault is mapped approximately 23 miles southwest of the site estimated to be capable of generating a M7.6 earthquake.
- *Elsinore Fault*: The Temecula and Julian sections of the Elsinore Fault Zone are located approximately 26 miles northeast and east of the site respectively, and this fault is capable of generating an earthquake of M6.5 to M7.5 (SCEDC, 2022).
- *San Diego Trough Fault Zone*: The San Diego Trough Fault Zone is mapped approximately 30 miles west-southwest of the site at its closest point. The most recent surface rupture is of Holocene age (SCEDC, 2022).
- *San Clemente Fault Zone*: The San Clemente Fault Zone is mapped approximately 55 miles southwest of the site at its closest point. The most recent surface rupture is of Holocene age (SCEDC, 2022).
- *San Jacinto Fault*: The San Jacinto Fault is mapped approximately 47 to 69 miles northeast of the site and is capable of generating an earthquake of M6.5 to M7.5 (SCEDC, 2022).

The potential for strong ground shaking from earthquakes on active southern California faults and active faults in northwestern Mexico should be anticipated at the site. Design of building structures in accordance with the current building codes would reduce the potential for injury or loss of human life. Buildings constructed in accordance with current building codes may suffer significant damage but should not undergo total collapse.

B. 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). Our investigation indicates that the subject site is not directly on a known active fault trace and, therefore, the risk of ground rupture is remote.

<u>Ground Shaking</u>: 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 significant damage. It is our opinion that the most serious damage to the site would be caused by a large earthquake originating on a nearby strand of the Rose Canyon, Coronado Bank or Newport-Inglewood Faults. Although the chance of such an event is remote, it could occur within the useful life of the structure.

<u>Landslides</u>: Our investigation indicates that the subject site is not directly on a known recent or ancient landslide. Review of the *"Geologic Map of the Oceanside 30'x60' Quadrangle, California"* by Kennedy and Tan (2007) and the *USGS Landslide Hazard Program Site (US* Landslide Inventory), indicate that there are no known or suspected ancient landslides located on the site.

<u>Slope Stability:</u> Our site reconnaissance and exploratory excavations indicate that the site is underlain by stable and dense bedrock material at shallow depths of 6 inches to 1.5 feet. In our opinion, there is not a slope stability issue with the site.

<u>Liquefaction</u>: 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, cohesionless saturated silt, sand, and fine-grained gravel deposits of Holocene to late Pleistocene age and in areas where the groundwater is shallower

than about 50 feet (DMG Special Publication 117) when they are sufficiently shaken by an earthquake. On this site, the risk of liquefaction of formational materials due to seismic shaking does not exist due to the dense to very dense (hard) nature of the underlying bedrock materials and the lack of shallow static groundwater. In our opinion, the site does not have a potential for soil strength loss to occur due to a seismic event.

Tsunamis and Seiches: A tsunami is a series of long waves generated in the ocean by a sudden displacement of a large volume of water. Underwater earthquakes, landslides, volcanic eruptions, meteor impacts, or onshore slope failures can cause this displacement. Tsunami waves can travel at speeds averaging 450 to 600 miles per hour. As a tsunami nears the coastline, its speed diminishes, its wave length decreases, and its height increases greatly. After a major earthquake or other tsunami-inducing activity occurs, a tsunami could reach the shore within a few minutes. One coastal community may experience no damaging waves while another may experience very destructive waves. Some low-lying areas could experience severe inland inundation of water and deposition of debris more than 3,000 feet inland. The site is located at several miles from the exposed coastline and at an elevation of approximately 450 feet above MSL. There is no risk of tsunami inundation at the site.

A seiche is a run-up of water within a lake or embayment triggered by fault- or landslide-induced ground displacement. The site is not located near any lakes or coastal lagoons that could be considered capable of producing a seiche and inundating the subject site.

Flood Hazard: Review of the FEMA flood maps number 06073C0764H, effective on 12/20/2019, the project site is located within the Special Flood Hazard Area X. Zone

X is described as minimal flood hazard. The civil engineer should verify this statement with the County of San Diego (FEMA, 2019).

***Geologic Hazards Summary**:* It is our opinion, based upon a review of the available maps, our research and our site investigation, that the site is underlain at shallow depth by dense to very dense (hard) bedrock materials and is suited for the proposed residential structure, retaining walls, swimming pool and associated improvements provided the recommendations herein are implemented. Furthermore, based on the available information at this stage, it appears the proposed site development will not destabilize or result in settlement of adjacent property or improvements if the recommendations presented in this report are implemented.

No significant geologic hazards are known to exist on the site that would prohibit the construction of the proposed residential structure, retaining walls and associated improvements. Ground shaking from earthquakes on active southern California faults and active faults in northwestern Mexico is the greatest geologic hazard at the property. Design of building structures in accordance with the current building codes would reduce the potential for injury or loss of human life. Buildings constructed in accordance with current building codes may suffer significant damage but should not undergo total collapse.

In our explicit professional opinion, no active or potentially active faults underlie the project site.

IX. GROUNDWATER

Groundwater was not encountered in our exploratory trench excavations to the maximum depths explored. We do not anticipate significant groundwater problems to develop in the future, *if the property is developed as proposed and proper drainage* *is implemented and maintained*. It should be kept in mind that any required construction operations will change surface drainage patterns and/or 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 should also be recognized that minor groundwater seepage problems might occur after development of a site even where none were present before development. These are usually minor phenomena and are often the result of an alteration in drainage patterns and/or an increase in irrigation water. Based on the permeability characteristics of the soil and the anticipated usage and development, it is our opinion that any seepage problems, which may occur, will be minor in extent. It is further our opinion that these problems can be most effectively corrected on an individual basis if and when they occur.

On properties such as the subject site where dense, low permeability soils exist at shallow depths, even normal landscape irrigation practices on the property or neighboring properties, 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.

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 used, should be provided with the proper cross-ventilation to help reduce the potential for moisture-related problems. Additional recommendations may be required at the time of construction.

It must be understood that unless discovered during site exploration or encountered during site construction 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 construction, should be evaluated and remedied by the project civil and geotechnical consultants. The project developer and property owner, however, must realize that post-construction appearances of groundwater may have to be dealt with on a site-specific basis. Proper functional surface drainage should be implemented and maintained at the property.

X. CONCLUSIONS & RECOMMENDATIONS

The following recommendations are based upon the practical field investigations conducted by our firm, and resulting laboratory tests, in conjunction with our knowledge and experience with similar soils in the San Diego County area. 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 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 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 recommencement of grading and/or foundation installation work and comply with the governing agency's requirements for a change to the Geotechnical Consultant of Record for the project.*

We recommend that the planned residential structure, garage, retaining walls, swimming pool and improvements be supported by conventional, individual-spread and/or continuous footing foundations founded on medium dense to dense (hard) bedrock/formational soils and minimum 90 percent compacted structural fill soils. Individual structures may bear on dense (hard) bedrock/formational or fill soils depending on their locations, final grading elevations and exposure of formational materials.

Removal and recompaction of existing topsoils and upper weathered bedrock across the proposed building pad area will be required to support the proposed structures and associated improvements. Fill soils across the site will be required to be compacted to at least 90 percent relative compaction. The existing highly expansive topsoil should not be used as fill or backfill. The weathered formational materials are considered suitable for use as recompacted fill soils. Any buried trash and roots encountered during site demolition and grading should be removed and exported off site.

Construction plans have not been provided to us for the preparation of this report, however, when completed they should be made available for our review. Additional or modified recommendations for foundation design and construction may be provided as warranted.

From a geotechnical engineering standpoint, it is our opinion that the site is suitable for construction of the proposed residences and associated improvements provided the conclusions and recommendations presented in this report are incorporated into its design and construction.

A. Preparation of Soils for Site Development

1. *General:* Grading should conform to the guidelines presented in the 2022 California Building Code (CBC) as well as the requirements of the County of San Diego.

   During earthwork construction, all highly expansive topsoil should be removed and only the low-expansive soils should be reprocessed along with the weathered bedrock materials. General grading procedures of the contractor, should be observed, and the fill placed 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 area of construction should be completely cleared and stripped of surficial and subsurface obstructions. This includes the complete removal of all near-surface rocks and miscellaneous debris (if encountered). After clearing, the site should be stripped of existing vegetation within the areas of proposed construction. *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 suitable compacted fill soils. Prior to any filling operations, the cleared and stripped vegetation and debris should be disposed of off-site.

3. *Excavation:* After the pertinent areas of the site have been cleared and stripped, the existing surficial topsoils and the upper weathered bedrock materials should be removed and recompacted. *The highly expansive topsoils should not be used as fill or backfill material.* The depth of removal across the site will vary depending on the thickness of unsuitable soils overlying the bedrock materials. It is anticipated that the depth of removal will be approximately 2 to 3 feet below existing grade in the areas of the proposed residence and improvements. It should be mentioned that the depths of removal described above are based on the results of our exploratory trench locations. Deeper or shallower removal may be necessary in areas outside our exploratory excavations.

   Based on our experience with similar materials in the area, it is our opinion that the existing topsoils materials and weathered bedrock can be excavated utilizing ordinary medium to heavyweight earthmoving equipment. Less weathered bedrock may require heavy equipment to excavate. 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 along with the excavation 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. Deeper excavation areas, such as the basement, may require the use of a rock breaker or similar equipment.

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 proposed building pad areas, we recommend that the cut portion of the building pad be undercut to a minimum of 24 inches 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 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 structural fill soil thickness differential of 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. 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. *Highly expansive soils should be removed from building and improvement areas*.

6. *Material for Fill:* Existing on-site low-expansion potential (Expansion Index of 50 or less per ASTM D4829-19) soils with an organic content of less than 3 percent by volume are, in general, suitable for use as fill. Medium expansive soils will require blending with low expansive soils during grading and should not be used as structural fill under building areas. 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 D1557-12r21. 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 within 3 percent of optimum. Highly expansive soils placed outside building and improvement areas should be compacted to at least 5 percent over optimum.

Soil compaction testing by nuclear method ASTM D6938-17a should be performed every 2 feet or less of fill placement by a representative of ***Geotechnical Exploration, Inc.*** Furthermore, our representative should perform necessary observation of fill placement during grading operations throughout the project.

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*: The test results discussed earlier in this report should be evaluated by an engineer specializing in corrosivity and type of cement recommendations should be provided by the structural engineer based on the current edition of the CBC (2022) or the American Concrete Institute and the soluble sulfate and chloride test results.

9. *Trench and Retaining Wall Backfill:* All utility trenches and retaining walls should be backfilled with properly compacted fill. Backfill material should be placed in lift thicknesses appropriate to the type of compaction equipment utilized and compacted to a minimum degree of compaction of 90 percent based upon ASTM D1557-12r21 by mechanical means. Any portion of the trench backfill in public street areas within pavement sections should conform to the material and compaction requirements of the adjacent pavement section.

   Backfill soils placed behind retaining or pool walls should be installed as early as the retaining walls are capable of supporting lateral loads. Backfill soils behind retaining or pool walls should be low expansive (Expansion Index less than 50 per ASTM D4829-19).

   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.

10. *Observations and Testing*: As stated in CBC 2022, *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 indicate 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.

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 or via email ***at least*** 24 hours in advance of the needed service visit.

B. Slopes

11. *Temporary Slopes*: Based on our subsurface investigation work, laboratory test results, and engineering analysis, temporary cut slopes up to 12 feet in height in the formational materials should be stable from mass instability at an inclination 0.75:1.0 (horizontal to vertical). Temporary cut slopes up to 12 feet in height in loose fill soils should be stable against mass instability at an inclination of 1.5:1.0. In properly compacted fill soils, temporary slopes should be stable at a slope ratio of 1:1 up to 12 feet in height.

Some localized sloughing or raveling of the soils exposed on the slopes may occur. Since the stability of temporary construction slopes will depend largely on the contractor's activities and safety precautions (storage and equipment loadings near the tops of cut slopes, surface drainage provisions, etc.), it should be the contractor's responsibility to establish and maintain all temporary construction slopes at a safe inclination appropriate to the methods of operation. No soil stockpiles or surcharge may be placed within a horizontal distance of 10 feet or the depth of the excavation, whichever is larger, from the excavation top.

If these recommendations are not feasible due to space constraints, temporary shoring may be required for safety and to protect adjacent property improvements. Similarly, footings near temporary cuts should be underpinned or protected with shoring.

12. *Slope Observations:* 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, any required slope design changes would be presented at that time.

13. *Permanent Slopes*: Any new or existing cut or fill slopes up to 15 feet in height should be constructed at an inclination of 2.0:1.0 (horizontal to vertical), be provided with a keyway at least 2 feet in depth and at least 10 feet in width for the entire length of the slope. Permanent slopes at a 2.0:1.0 slope ratio should possess a factor of safety of 1.5 against deep and shallow failure.

    Slope construction should adhere to Appendix J "Grading" of the 2022-CBC Standards for slope construction, benching, setbacks, drainage and terracing, slope protection and erosion control and in the County of San Diego grading ordinances.

C. Seismic Design Criteria

14. *Seismic Data Bases:* The estimation of the peak ground acceleration (PGA) 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.

15. *Seismic Design Criteria:* The proposed structure should be designed in accordance with the 2022 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 33.0209 degrees and a longitude of -117.1586 degrees, utilizing a program titled "*Seismic 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. See Appendix B.

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

17. *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 2022 CBC. The summarized seismic soil parameters are presented in Table I below, have been calculated with the SEAOC Seismic Design Map Tool. The complete values are included in Appendix B. The 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
0.867	0.319	1.0	0.631	0.667	0.421	1.153	1.979	0.375	1.225	D

D. Foundation Recommendations

18. *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 medium dense to dense (hard) bedrock/formational materials or on properly compacted fill soils over bedrock/formational soils. No footings should be underlain by undocumented fill soils. All building footings should be built on formational soils or properly compacted fill prepared as recommended in this report. 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 so as to provide at least 8 feet of horizontal cover between the slope face and outside edge of the footing at the footing bearing level.

19. *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 medium dense to dense (hard) bedrock/formational soils or properly compacted fill soils.

20. *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.

21. *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 0.40 is considered applicable. An additional allowable passive resistance equal to an equivalent fluid weight of 275 pounds per cubic foot (pcf) acting against the foundations may be used in design provided the footings are poured neat against the medium dense to 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.

22. *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.

E. ***Retaining Wall Design***

23. *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 to be 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.

24. *Design Parameters – Restrained:* Temporary or permanent site restrained retaining walls or restrained building retaining walls supporting low expansive 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-expansive soils 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.

25. *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 13 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.

26. *Retaining Wall Drainage:* The preceding design pressures assume that the walls are backfilled with properly compacted low expansion potential materials 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. See Figure No. VI, Retaining Wall Subdrain Schematic.

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.

27. Drainage Quality Control: 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.

F. Concrete Slab On-Grade Criteria

Slabs on-grade may only be used on new, properly compacted fill or when bearing on dense formational soils.

28. *Minimum Floor Slab Thickness and Reinforcement:* 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 minimum 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 4 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 placement under the new concrete slab. The vapor barrier or waterproofing should be installed following the manufacturer's guidelines.

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.

29. *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 are providing as Appendix D a discussion regarding minimum protection for slabs. 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.

30. *Exterior Slab Thickness and Reinforcement:* Exterior slab reinforcement and control joints should be designed by the project Structural Engineer. 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 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.

G. ***Pavements***

31. *Concrete Pavement:* We recommend that driveways subject only to automobile and light truck traffic be 5.5 inches thick and be supported directly on properly prepared/compacted on-site subgrade soils. The upper 6 inches of the subgrade below the slab should be compacted to a minimum degree of compaction of 95 percent just prior to paving. The concrete should conform to Section 201 of The Standard Specifications for Public Works Construction, 2024 Edition, for Class 560-C-3250.

In order to control shrinkage cracking, we recommend that saw-cut, weakened-plane joints be provided at about 12-foot centers both ways and at reentrant corners. The pavement slabs should be saw-cut as soon as practical but no more than 24 hours after the placement of the concrete. The depth of the joint should be one-quarter of the slab thickness and its width should not exceed 0.02-foot. Reinforcing steel is not necessary unless it is desired to increase the joint spacing recommended above. Pavement joints should be sealed with an approved pavement joint sealer.

32. *Interlocking Permeable Pavers*: If desired, we recommend that permeable pavement pavers for the driveway, subject only to automobile and light truck traffic, and other hardscape areas be supported on a 1.5 inches of bedding sand No. 8, on 6 inches of crushed miscellaneous base conforming to Section 219 of the Standard Specifications for Public Works Construction, 2024 Edition (or 6 inches of No. 57 crushed rock gravel per ASTM D448 gradation). The upper 12 inches of the pavement subgrade soil as well as the aggregate base layer should be compacted to a minimum degree of compaction of 95 percent. Preparation of the subgrade and placement of the base materials should be performed under the observation of our representative.

H. Swimming Pool Recommendations

Preliminary or final pool plans have not been provided to us during the preparation of this report. Based on a conceptual plan provided, we understand that a swimming pool is proposed in the area northwest of the residence and near the northeastern slope. The existing topsoil and weathered bedrock in this area was observed to be approximately 3 feet thick. This information could be helpful in pool design considerations, i.e., pool depth, location, grades, etc. Final pool plans should be made available for our review.

33. *Swimming Pool Design:* The bearing surfaces of the proposed pool foundations should be founded entirely in dense formational soils. Any existing fill should be removed and recompacted to support a pool deck or portion of pool walls. The new pool excavation should be verified by our firm within 48 hours prior to steel and concrete placement. Should the pool shell not be entirely founded in formational soils across the entire bottom, areas of fill should be removed to formational soils and backfilled with sand cement slurry including 4 bags of cement per cubic yard and have a minimum thickness of 1 foot.

The swimming pool shell should be designed for a soil pressure of at least 50 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 capable of supporting 62.4 pcf. The outer edge of the pool toward the descending slope (or spa) should be provided with a foundation setback of at least 10 feet from a descending slope face or retaining walls. Any portion of the pool shell within 10 feet of a slope face should be designed to support at least the water pressure of 62.4 pcf. If the pool is over 6 feet in depth, the soil seismic increment 15 pcf recommended for retaining walls should also be included.

34. *Swimming Pool Deck:* As a minimum, we recommend that the pool deck concrete slabs be 4 inches thick and be founded on properly compacted and tested fill, with No. 3 bars at 15-inch centers, both ways, at the center of the slab, and contain adequate isolation and control joints. Actual pool deck reinforcement and control joint spacing should be designed by the project's structural engineer or pool design engineer. The shrinkage joints should be spaced no farther than 12 inches and at reentrant corners. All joints should be sealed with elastomeric joint sealant that should be properly maintained by the property owner.

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 pool deck subgrade should be properly moisture conditioned and compacted, and should be verified by our firm within 48 hours prior to steel and concrete. The swimming pool deck and surrounding patio areas should be provided with adequate surface drainage including positive surface drainage and/or functional area drains. The surface water should be directed toward area drains and away from the slope face.

I. ***<u>Site Drainage Considerations</u>***

35. *<u>Erosion Control</u>*: 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.

36. *<u>Surface Drainage:</u>* Adequate measures should be taken to properly finish-grade the lot after the residential structures 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 residences, 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 CBC requires a minimum 1-percent surface gradient for proper drainage of building pads unless waived by the building official. Concrete pavement may have a minimum gradient of 0.5-percent.

37. *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.

J. General Recommendations

38. *Project Start Up Notification:* In order to reduce work delays during site development, this firm should be contacted 48 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 re-designing 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.).

39. *Cal-OSHA*: 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.

40. *Erosion Control*: 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.

41. *Construction Best Management Practices (BMPs):* Construction BMPs must be implemented in accordance with the requirements of the controlling jurisdiction. 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 work day 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 a period greater than 7 days 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.

XI. GRADING NOTES

Geotechnical Exploration, Inc. recommends that we be retained to verify the actual soil conditions revealed during site grading work and footing excavation to be as anticipated in this "*Report of Preliminary Geotechnical Investigation*" for the project. In addition, the placement and compaction of any fill or backfill soils during site grading work must be observed and tested by the soil engineer.

It is the responsibility of the grading contractor and general contractor to comply with the requirements on the grading plans as well as 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.

XII. LIMITATIONS

Our conclusions and recommendations have been based on available data obtained from our field investigation and laboratory analysis, as well as our experience with similar soils and formational materials located in this area of the County 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 grading operations begin or when footing excavations are placed. 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.

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 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.

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 building plans, especially with respect to the height and location of any proposed structures, this report must be presented to us for immediate review and possible revision.

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 project plans once they are available, to verify that our recommendations are adequately incorporated in the plans. Additional or modified recommendations may be issued if warranted after plan 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 on the site; 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. 21-13629** will expedite a reply to your inquiries.

Respectfully submitted,

GEOTECHNICAL EXPLORATION, INC.

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September 2024

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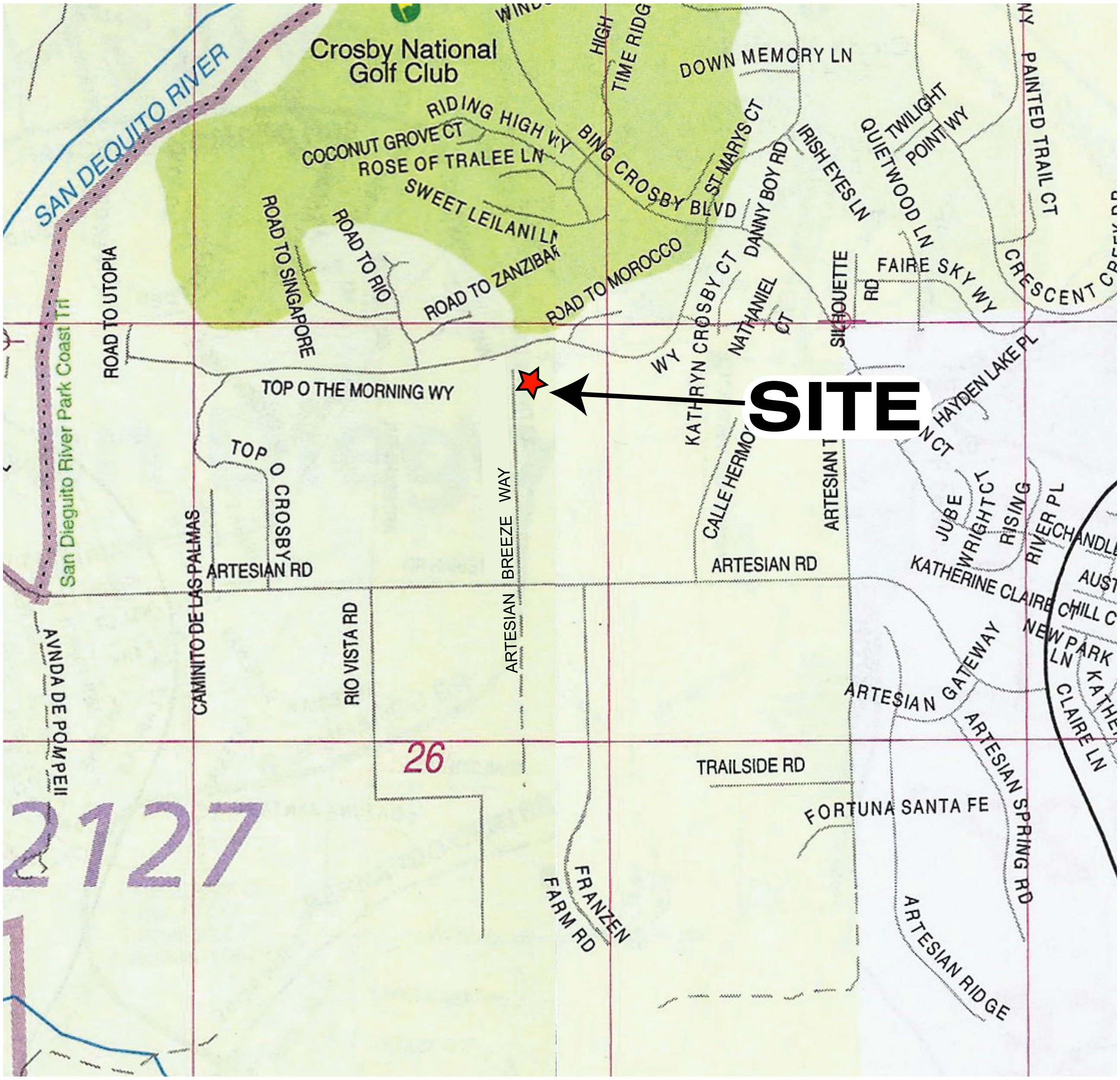
VICINITY MAP



Thomas Bros. Guide San Diego County, pg 1169-A3

Balazs Residential Project
A.P.N. 267-147-06
Artesian Breeze Way
San Diego, CA.

Figure No. I
Job No. 21-13629

Geotechnical Exploration, Inc.

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.

N

40 0 20 40 60

GRAPHIC SCALE
(1" = 40')

LEGEND

T-7	Approximate Location of Exploratory Trench
🟧	Proposed BMP Area

GEOLOGIC LEGEND

Qaf	Artificial Fill
Ts	Topsoil
Mzu	Metasedimentary and Metavolcanic Rocks
- -?- -	Approximate Geologic Contact

REFERENCE: *This Plot Plan was prepared from and existing undated PLAN by AP CONSULTING and from on-site field reconnaissance performed by GEI.*

PLOT PLAN AND SITE SPECIFIC GEOLOGIC MAP

Balazs Residential Project
A.P.N. 267-147-06
Artesian Breeze Way
San Diego, CA.
Figure No. II
Job No. 21-13629

Geotechnical Exploration, Inc.

September 2024

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Rubber-tire Backhoe	**14' X 2.5' X 4.25' Trench**	**1-13-22**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 432' Mean Sea Level	**Not Encountered**	**SO**

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)
		1	**SANDY CLAY** , with gravel. Soft. Very moist. Dark gray-brown. **TOPSOIL** -- minor roots.	CL									
1		2	**SANDY CLAY** , with gravel and cobbles. Firm to stiff. Very moist. Dark red-brown. **WEATHERED METASEDIMENTARY AND METAVOLCANIC BEDROCK (Mzu)**	CL	28.6	92.6							
		☒	-- some random boulders.										
2			-- 74% passing #200 sieve.		31.0		16.8	110.9			133		
			-- becomes less weathered.										
		3	**SANDY CLAY/ CLAYEY SAND** , with angular gravel and cobbles. Stiff to very stiff (dense to very dense). Moist. Light brown, yellow-brown and gray-brown. **METASEDIMENTARY AND METAVOLCANIC BEDROCK (Mzu)**	CL/ SC									
3													
4			-- 30% passing #200 sieve.										
		4	-- becomes fine- to coarse-grained **CLAYEY SAND**; increasing hardness with depth to unweathered bedrock.	CL/ SC	14.5								
5			Bottom @ 4.25'										

▼ PERCHED WATER TABLE

☒ BULK BAG SAMPLE

1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

S NUCLEAR FIELD DENSITY TEST

▨ STANDARD PENETRATION TEST

JOB NAME
Balazs Residential Project - APN 267-147-06

SITE LOCATION
Artesian Breeze Road, San Diego, CA

JOB NUMBER
21-13629

REVIEWED BY
LDR/JAC

Geotechnical Exploration, Inc.

LOG No.
T-1

FIGURE NUMBER
IIIa

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Rubber-tire Backhoe	**10' X 2.5' X 3.5' Trench**	**1-13-22**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 440' Mean Sea Level	**Not Encountered**	**SO**

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. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
			SANDY CLAY , with gravel. Soft. Very moist. Dark gray-brown. **TOPSOIL** -- minor roots.	CL								
1		1	**SANDY CLAY** , with gravel and cobbles. Firm to stiff. Very moist. Dark red-brown. **WEATHERED METASEDIMENTARY AND METAVOLCANIC BEDROCK (Mzu)** -- some random boulders. -- becomes less weathered.	CL	30.5	92.7						
2		2	**SANDY CLAY/ CLAYEY SAND** , with angular gravel and cobbles. Stiff to very stiff (dense to very dense). Moist. Light brown, yellow-brown and gray-brown. **METASEDIMENTARY AND METAVOLCANIC BEDROCK (Mzu)**	CL/ SC								
3			-- becomes fine- to coarse-grained **CLAYEY SAND**; increasing hardness with depth to unweathered bedrock.	CL/ SC								
4			Bottom @ 3.5'									
5												

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

JOB NAME
Balazs Residential Project - APN 267-147-06

SITE LOCATION
Artesian Breeze Road, San Diego, CA

JOB NUMBER	REVIEWED BY	LOG No.
21-13629	**LDR/JAC**	**T-2**
FIGURE NUMBER	**Geotechnical Exploration, Inc.**	
IIIb		

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Rubber-tire Backhoe	**10' X 2.5' X 6.5' Trench**	**1-13-22**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 435' Mean Sea Level	**Not Encountered**	**SO**

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)
1	▨	☒	**SANDY CLAY** , with gravel. Soft. Very moist. Dark gray-brown. **TOPSOIL** -- minor roots.	CL			11.4	122.8					
2	▨	1	**SANDY CLAY** , with gravel and cobbles. Firm to stiff. Moist to very moist. Dark red-brown. **WEATHERED METASEDIMENTARY AND METAVOLCANIC BEDROCK (Mzu)** -- some random boulders. -- becomes less weathered.	CL	19.6	104.0							
3 4 5 6	▨	☒	**SANDY CLAY/ CLAYEY SAND** , with angular gravel and cobbles. Stiff to very stiff (dense to very dense). Moist. Light brown, yellow-brown and gray-brown. **METASEDIMENTARY AND METAVOLCANIC BEDROCK (Mzu)** -- becomes fine- to coarse-grained **CLAYEY SAND**; increasing hardness with depth to unweathered bedrock.	CL/ SC CL/ SC			16.6	110.5			31		
7			Bottom @ 6.5'										

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

JOB NAME
Balazs Residential Project - APN 267-147-06

SITE LOCATION
Artesian Breeze Road, San Diego, CA

JOB NUMBER	REVIEWED BY	LOG No.
21-13629	**LDR/JAC**	**T-3**
FIGURE NUMBER	Geotechnical Exploration, Inc.	
IIIc		

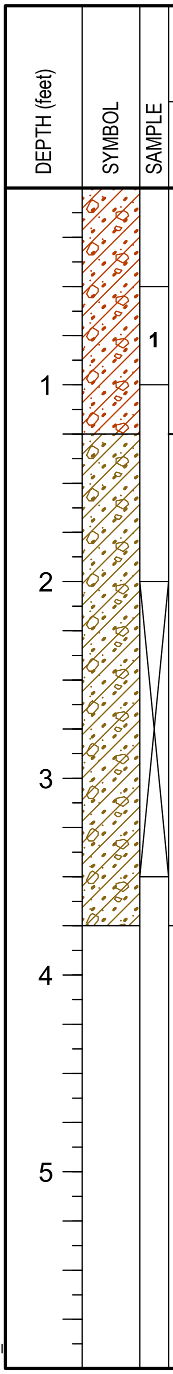
EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Rubber-tire Backhoe	**10' X 2.5' X 3.25' Trench**	**1-13-22**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 420' Mean Sea Level	**Not Encountered**	**SO**

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. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
			DESCRIPTION AND REMARKS (Grain size, Density, Moisture, Color)	U.S.C.S.								
	▨	1	**SANDY CLAY** , with gravel. Soft. Very moist. Dark gray-brown. **TOPSOIL** -- minor roots. -- 67% passing #200 sieve.	CL	29.1							
1	▨		**SANDY CLAY** , with gravel and cobbles. Firm to stiff. Very moist. Dark red-brown.	CL								
	▨		**WEATHERED METASEDIMENTARY AND METAVOLCANIC BEDROCK (Mzu)** -- becomes less weathered @ 1.5'.	CL/ SC								
2	▨	2	**SANDY CLAY/ CLAYEY SAND** , with angular gravel and cobbles. Stiff to very stiff (dense to very dense). Moist. Light brown, yellow-brown and gray-brown. **METASEDIMENTARY AND METAVOLCANIC BEDROCK (Mzu)**		12.7	116.7						
3	▨		-- becomes fine- to coarse-grained **CLAYEY SAND**; increasing hardness with depth to unweathered bedrock.	CL/ SC								
4			Bottom @ 3.25'									
5												

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

JOB NAME		
Balazs Residential Project - APN 267-147-06		
SITE LOCATION		
Artesian Breeze Road, San Diego, CA		
JOB NUMBER **21-13629**	REVIEWED BY **LDR/JAC**	LOG No. **T-4**
FIGURE NUMBER **IIId**	**Geotechnical Exploration, Inc.**	

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Rubber-tire Backhoe	**8.5' X 2.5' X 3.75' Trench**	**1-13-22**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 445' Mean Sea Level	**Not Encountered**	**SO**



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. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
SANDY CLAY , with gravel and cobbles. Soft to firm. Very moist. Dark red-brown. **WEATHERED METASEDIMENTARY AND METAVOVANIC BEDROCK (Mzu)** -- minor roots. -- some boulders. -- becomes less weathered @ 1.25'.	CL								
SANDY CLAY/ CLAYEY SAND , with angular gravel and cobbles. Stiff to very stiff (dense to very dense). Moist. Light brown, yellow-brown and gray-brown. **METASEDIMENTARY AND METAVOLCANIC BEDROCK (Mzu)**	CL/ SC								
-- becomes fine- to coarse-grained **CLAYEY SAND**; increasing hardness with depth to unweathered bedrock.	CL/ SC								
Bottom @ 3.75'									

▼ PERCHED WATER TABLE

☒ BULK BAG SAMPLE

1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

S NUCLEAR FIELD DENSITY TEST

▨ STANDARD PENETRATION TEST

JOB NAME **Balazs Residential Project - APN 267-147-06**		
SITE LOCATION **Artesian Breeze Road, San Diego, CA**		
JOB NUMBER **21-13629**	REVIEWED BY **LDR/JAC**	LOG No. **T-5**
FIGURE NUMBER **IIIe**	Geotechnical Exploration, Inc.	

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Rubber-tire Backhoe	**10' X 2.5' X 5' Trench**	**1-13-22**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 451' Mean Sea Level	**Not Encountered**	**SO**

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. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
	▨		**SANDY CLAY** , with gravel. Soft. Very moist. Dark gray-brown. **TOPSOIL**	CL								
		1	-- some roots.		17.2	97.3						
1			-- becomes less weathered @ 1.5'.									
2	▨		**SANDY CLAY** , with gravel and cobbles. Firm to stiff. Moist to very moist. Dark red-brown. **WEATHERED METASEDIMENTARY AND METAVOLCANIC BEDROCK (Mzu)**	CL								
		2	-- some boulders.									
3	▨		**SANDY CLAY/ CLAYEY SAND** , with angular gravel and cobbles. Stiff to very stiff (dense to very dense). Moist. Light brown, yellow-brown and gray-brown. **METASEDIMENTARY AND METAVOLCANIC BEDROCK (Mzu)**	CL/ SC CL/ SC								
4		3	-- becomes fine- to coarse-grained **CLAYEY SAND**; increasing hardness with depth to unweathered bedrock.		14.7	116.8						
5												
6			Bottom @ 5'									

▼ PERCHED WATER TABLE

☒ BULK BAG SAMPLE

1 IN-PLACE SAMPLE

■ MODIFIED CALIFORNIA SAMPLE

S NUCLEAR FIELD DENSITY TEST

▨ STANDARD PENETRATION TEST

JOB NAME
Balazs Residential Project - APN 267-147-06

SITE LOCATION
Artesian Breeze Road, San Diego, CA

JOB NUMBER
21-13629

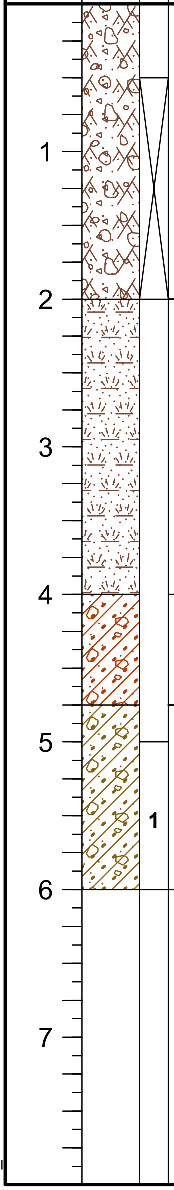
REVIEWED BY
LDR/JAC

Geotechnical Exploration, Inc.

FIGURE NUMBER
IIIf

LOG No.
T-6

EQUIPMENT	DIMENSION & TYPE OF EXCAVATION	DATE LOGGED
Rubber-tire Backhoe	**15' X 2.5' X 6' Trench**	**1-13-22**
SURFACE ELEVATION	GROUNDWATER/ SEEPAGE DEPTH	LOGGED BY
± 449' Mean Sea Level	**Not Encountered**	**SO**



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. - (%)	BLOW COUNTS/FT.	SAMPLE O.D. (INCHES)
		☒	**SANDY CLAY/ CLAYEY SAND**, with gravel. Soft/ loose. Very moist. Dark brown. **FILL/ TOPSOIL** -- some roots and plastic fragments. -- approximately 2 feet of soil has been dumped and spread near the southwest corner of the lot.	CL/ SC								
			SANDY CLAY, with gravel. Soft. Very moist. Dark gray-brown. **TOPSOIL**	CL								
			SANDY CLAY, with gravel and cobbles. Firm to stiff. Moist. Dark red-brown. **WEATHERED METASEDIMENTARY AND METAVOLCANIC BEDROCK (Mzu)** -- becomes less weathered @ 4.75'.	CL								
		1	**SANDY CLAY/ CLAYEY SAND**, with angular gravel and cobbles. Stiff to very stiff (dense to very dense). Moist. Light brown, red-brown and gray-brown. **METASEDIMENTARY AND METAVOLCANIC BEDROCK (Mzu)**	CL/ SC								
			Bottom @ 6'									

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

JOB NAME
Balazs Residential Project - APN 267-147-06

SITE LOCATION
Artesian Breeze Road, San Diego, CA

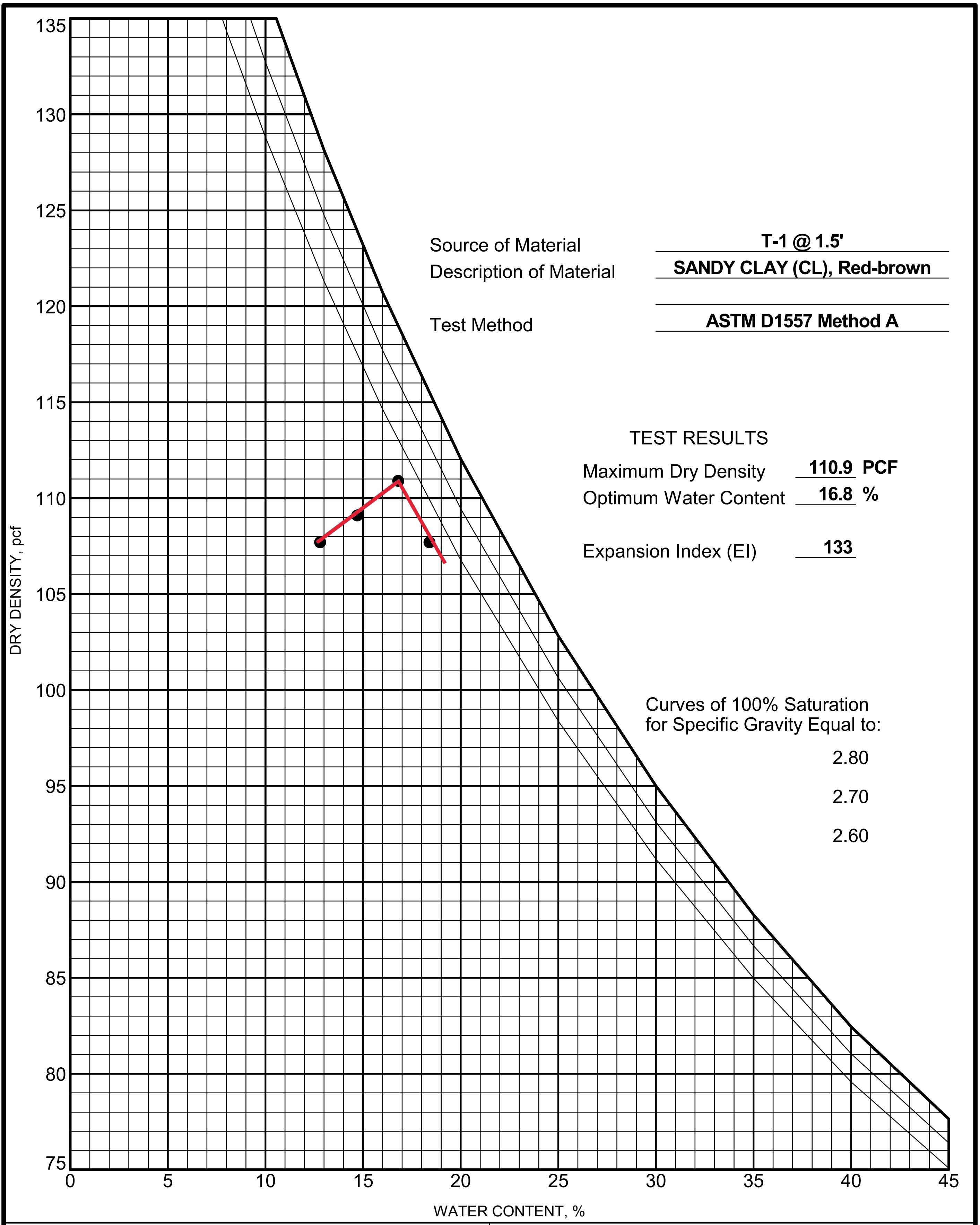
JOB NUMBER
21-13629

REVIEWED BY
LDR/JAC

Geotechnical Exploration, Inc.

FIGURE NUMBER
IIIg

LOG No.
T-7



Source of Material **T-1 @ 1.5'**

Description of Material **SANDY CLAY (CL), Red-brown**

Test Method **ASTM D1557 Method A**

TEST RESULTS

Maximum Dry Density **110.9 PCF**

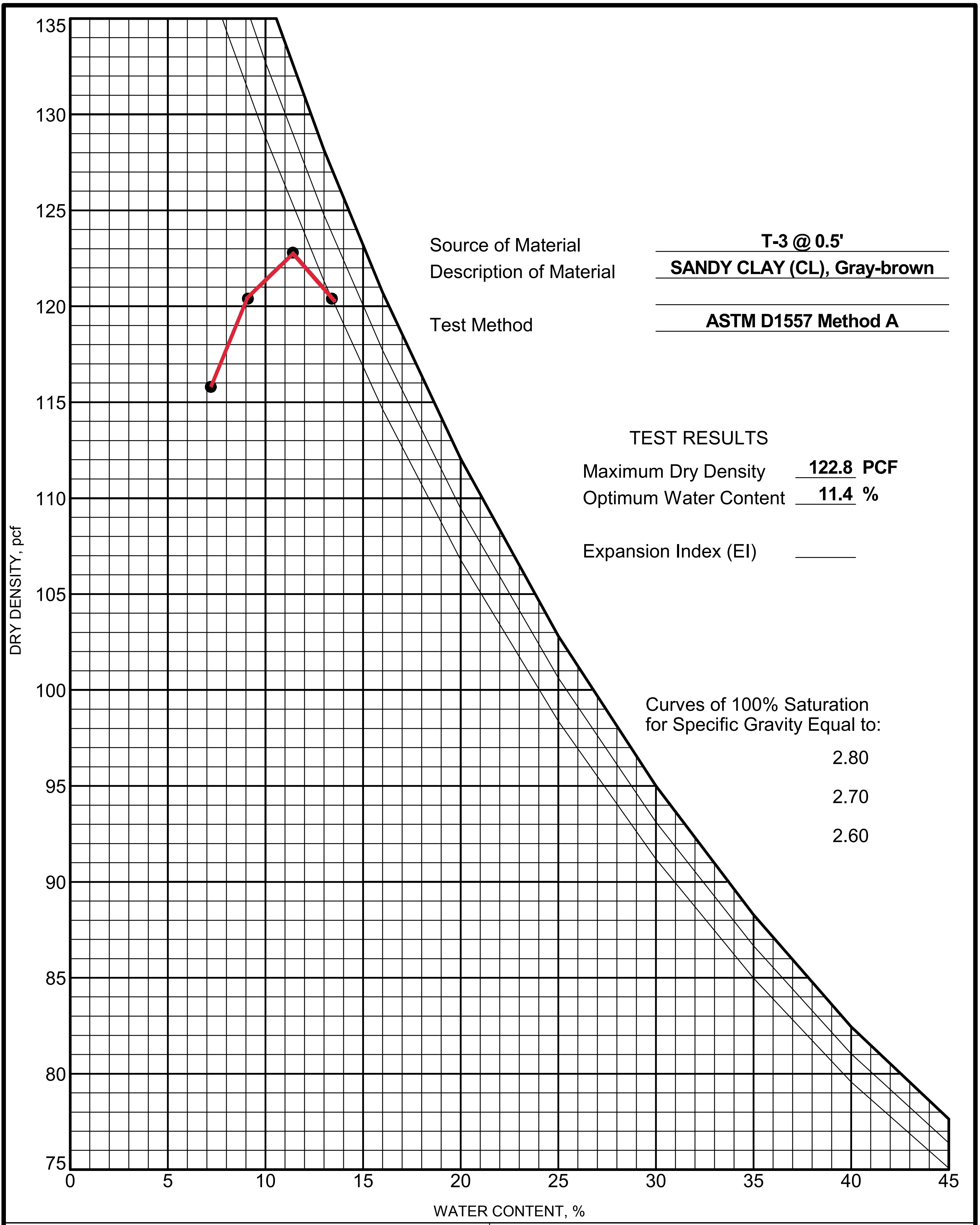
Optimum Water Content **16.8 %**

Expansion Index (EI) **133**

Geotechnical Exploration, Inc.

MOISTURE-DENSITY RELATIONSHIP

Figure Number: IVa
Job Name: Balazs Residential Project - APN 267-147-06
Site Location: Artesian Breeze Road, San Diego, CA
Job Number: 21-13629



Source of Material **T-3 @ 0.5'**

Description of Material **SANDY CLAY (CL), Gray-brown**

Test Method **ASTM D1557 Method A**

TEST RESULTS

Maximum Dry Density **122.8** **PCF**

Optimum Water Content **11.4** **%**

Expansion Index (EI) ______

Curves of 100% Saturation
for Specific Gravity Equal to:

2.80

2.70

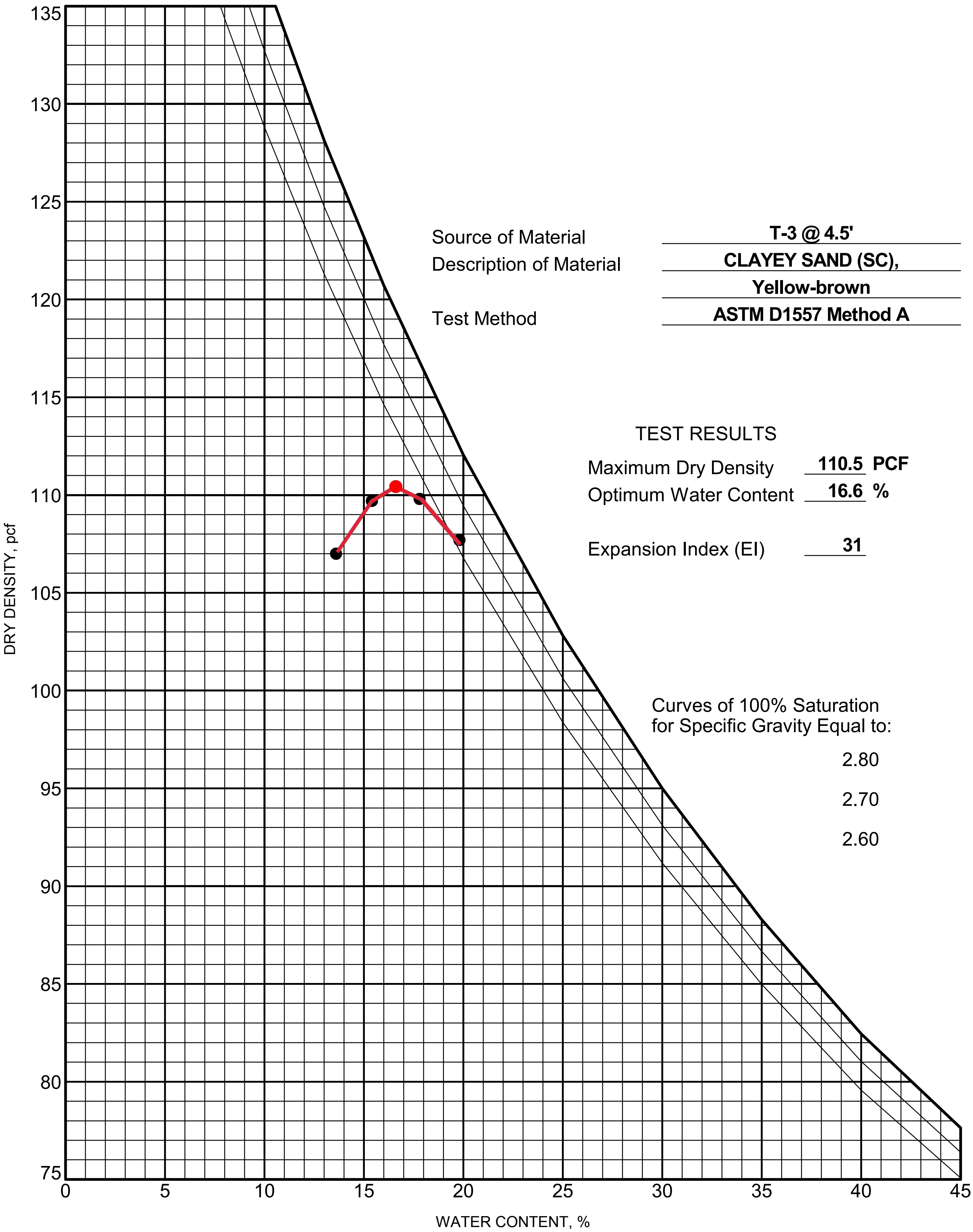
2.60

Geotechnical Exploration, Inc.

MOISTURE-DENSITY RELATIONSHIP

Figure Number: IVb
Job Name: Balazs Residential Project - APN 267-147-06
Site Location: Artesian Breeze Road, San Diego, CA
Job Number: 21-13629

COMPACTION + EI DARK GRID 13629 BALAZS.GPJ GEI FEB06.GDT 2/2/22



Source of Material **T-3 @ 4.5'**

Description of Material **CLAYEY SAND (SC),**
Yellow-brown

Test Method **ASTM D1557 Method A**

TEST RESULTS

Maximum Dry Density **110.5 PCF**

Optimum Water Content **16.6 %**

Expansion Index (EI) **31**

Curves of 100% Saturation
for Specific Gravity Equal to:

2.80

2.70

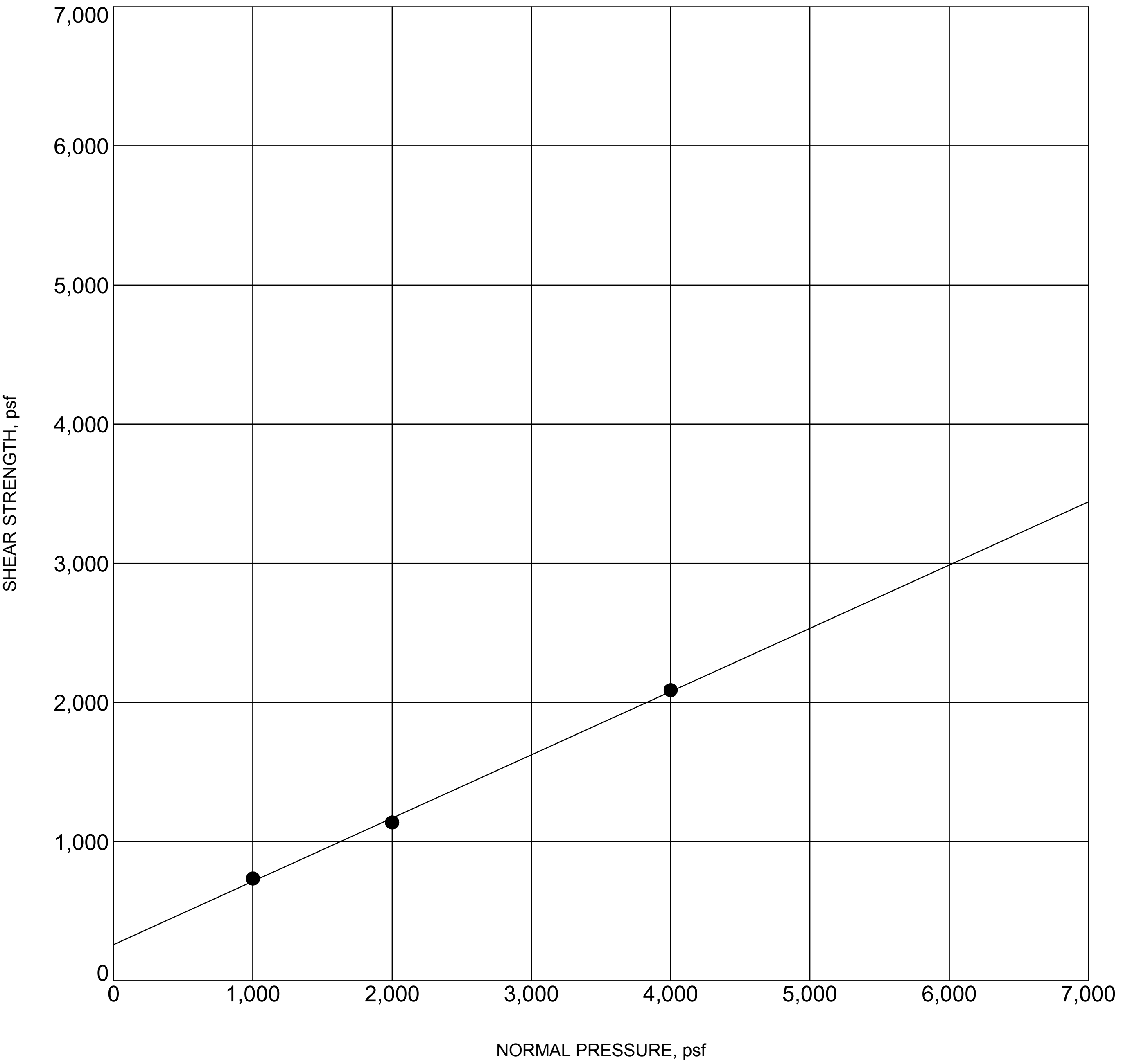
2.60

Geotechnical Exploration, Inc.

MOISTURE-DENSITY RELATIONSHIP

Figure Number: IVc
Job Name: Balazs Residential Project - APN 267-147-06
Site Location: Artesian Breeze Road, San Diego, CA
Job Number: 21-13629

COMPACTION + EI DARK GRID 13629 BALAZS.GPJ GEI FEB06.GDT 2/2/22

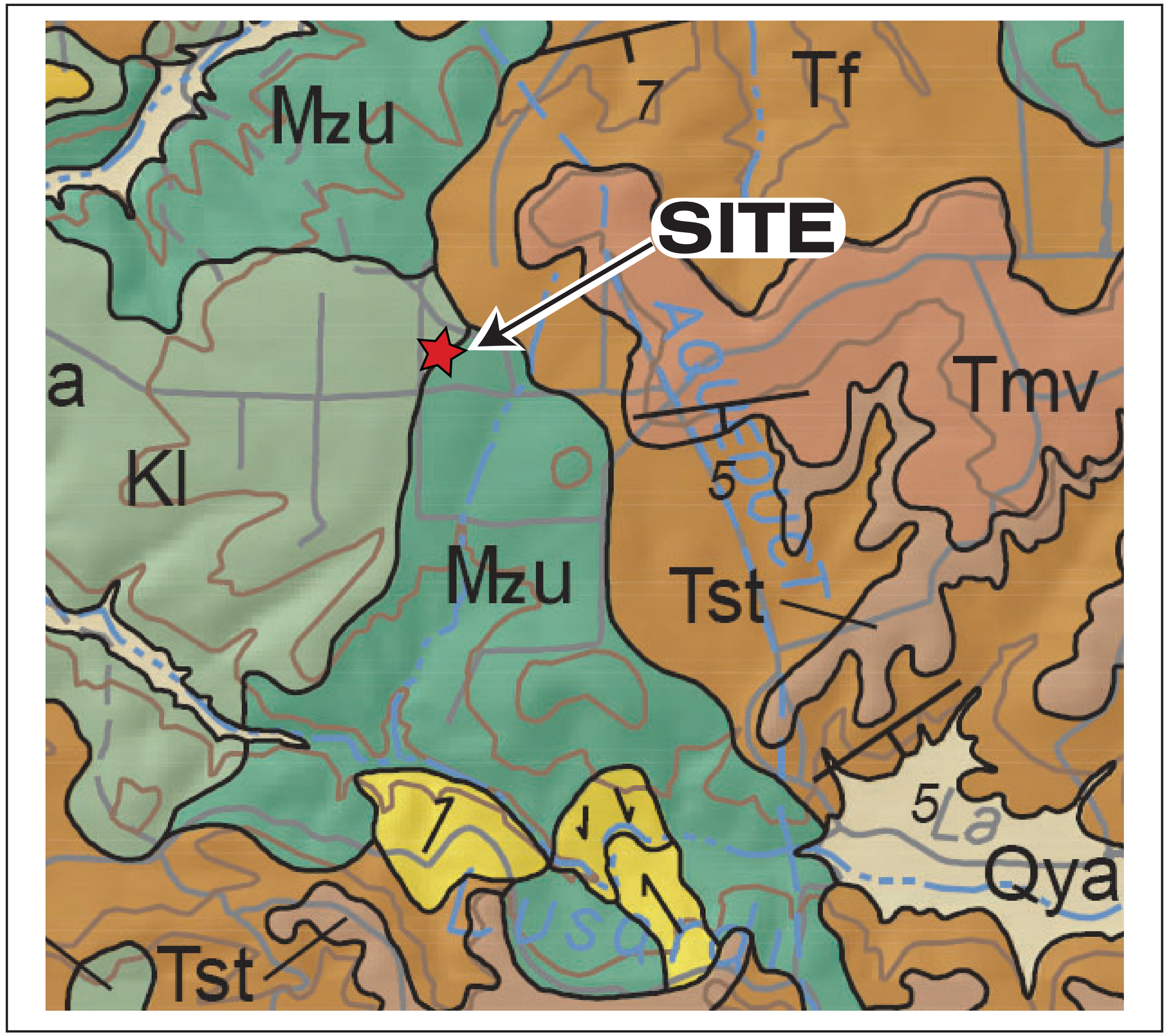


	Specimen Identification	Classification	γ_d	MC%	c	ϕ
●	**T-3 @ 4.5'**	**CLAYEY SAND (SC), Yellow-brown**			**259**	**24**

Geotechnical Exploration, Inc.

DIRECT SHEAR TEST

Figure Number: IVd
Job Name: Balazs Residential Project - APN 267-147-06
Site Location: Artesian Breeze Road, San Diego, CA
Job Number: 21-13629



Balazs Residential Project
A.P.N. 267-147-06
Artesian Breeze Way
San Diego, CA.

EXCERPT FROM
GEOLOGIC MAP OF THE OCEANSIDE 30' × 60' QUADRANGLE, CALIFORNIA

Compiled by
Michael P. Kennedy[1] and Siang S. Tan[1]
2007

Digital preparation by
Kelly R. Bovard[2], Rachel M. Alvarez[2], Michael J. Watson[2], and Carlos I. Gutiérrez[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

MzU	Metasedimentary and Metavolcanic Rocks
Kl	Lusardi Formation

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.

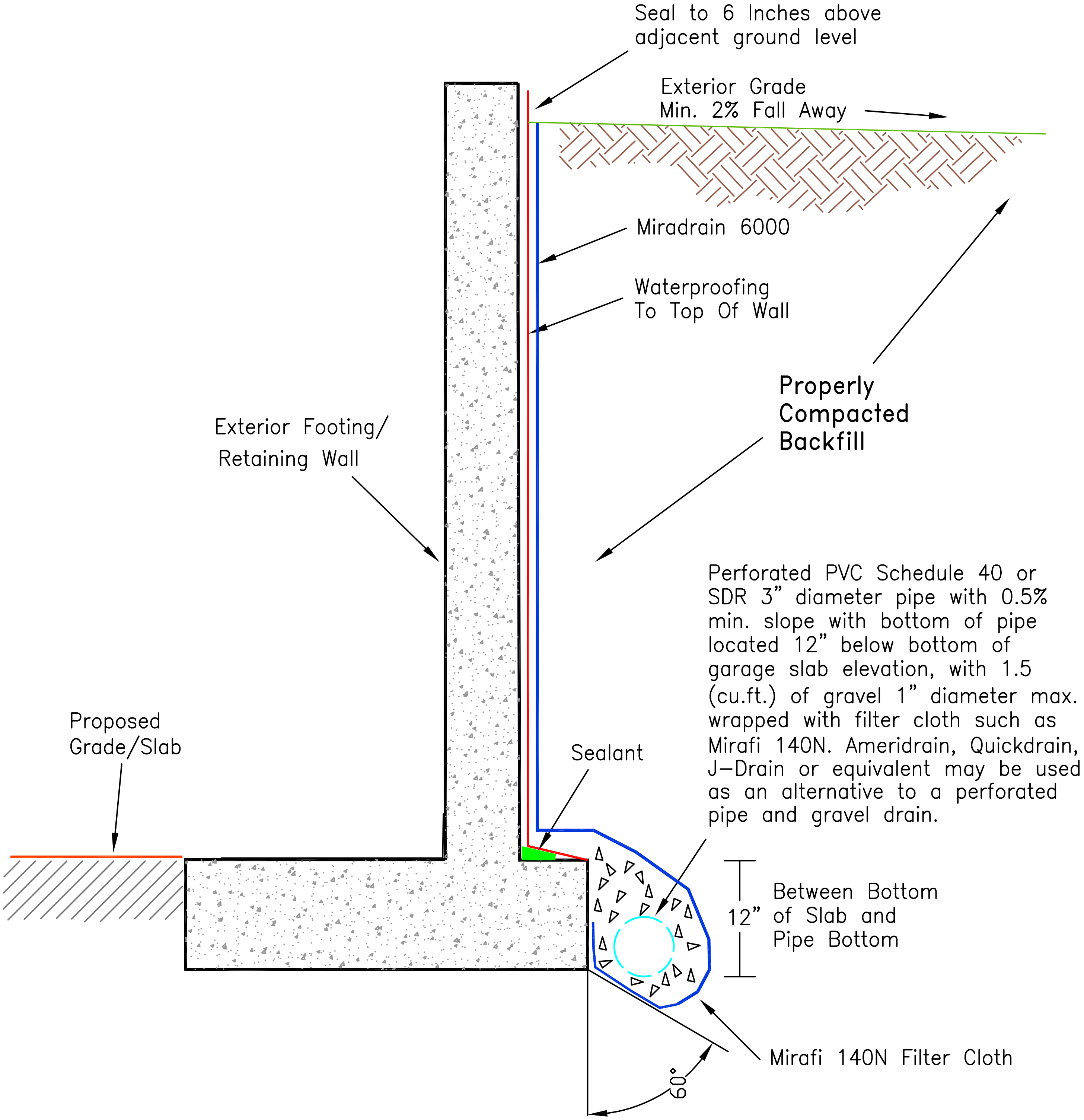
This map was funded in part by the U.S. Geological Survey National Cooperative Geologic Mapping Program, STATEMAP Award no. 98HQAG2049.

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

Figure No. V
Job No. 21-13629

Geotechnical Exploration, Inc.
February 2022

SUBGRADE RETAINING WALL DRAINAGE RECOMMENDATIONS



Perforated PVC Schedule 40 or SDR 3" diameter pipe with 0.5% min. slope with bottom of pipe located 12" below bottom of garage slab elevation, with 1.5 (cu.ft.) of gravel 1" diameter max. wrapped with filter cloth such as Mirafi 140N. Ameridrain, Quickdrain, J–Drain or equivalent may be used as an alternative to a perforated pipe and gravel drain.

NOT TO SCALE

Figure No.VI
Job No. 21-13629

Geotechnical Exploration, Inc.

February 2022

VI-221-13629-RW.dwg

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.

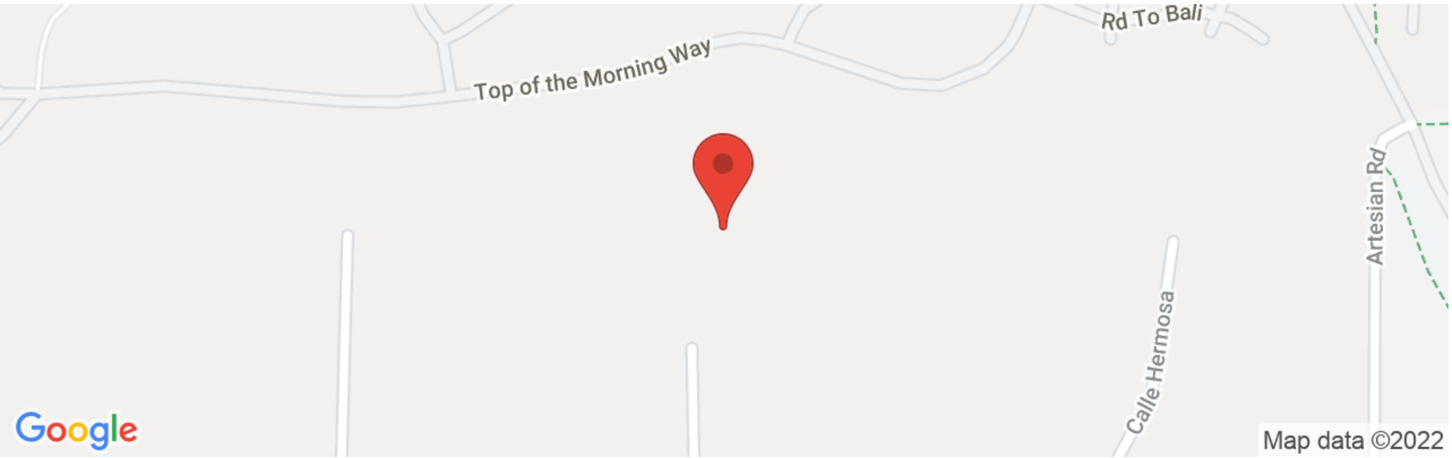
Fine-grained (More than half of material is smaller than a No. 200 sieve)

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

Artesian Breeze Way, APN 267-147-06

Latitude, Longitude: 33.0209, -117.1586



Date	2/1/2022, 1:12:27 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	0.867	MCE_R ground motion. (for 0.2 second period)
S_1	0.319	MCE_R ground motion. (for 1.0s period)
S_{MS}	1	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8 0.631	Site-modified spectral acceleration value
S_{DS}	0.667	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8 0.421	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	1.153	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8 1.979	Site amplification factor at 1.0 second
PGA	0.375	MCE_G peak ground acceleration
F_{PGA}	1.225	Site amplification factor at PGA
PGA_M	0.459	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	0.867	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	0.948	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.319	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.347	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.914	Mapped value of the risk coefficient at short periods
C_{R1}	0.92	Mapped value of the risk coefficient at a period of 1 s

APPENDIX C

WATER SOLUBLE SULFATE AND CHLORIDE LAB RESULTS

LABORATORY REPORT

Telephone (619) 425-1993 Fax 425-7917 Established 1928

CLARKSON LABORATORY AND SUPPLY INC.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
ANALYTICAL AND CONSULTING CHEMISTS

Date: September 12, 2024
Purchase Order Number: 21-13629
Sales Order Number: 64940
Account Number: GEOE

To:

Geotechnical Exploration, INC.
7420 Trade Street
San Diego, Ca 92121

Laboratory Number: SO1396-1 Customers Phone: 858-549-7222

Sample Designation:

One soil sample received on 09/04/24 at 12:00pm,
taken from Balazs 21-13629 marked as Sample 1.

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 7.4

Water Added (ml)	Resistivity (ohm-cm)
10	24000
5	9600
5	4700
5	2900
5	1900
5	1700
5	1800
5	1900

38 years to perforation for a 16 gauge metal culvert.
49 years to perforation for a 14 gauge metal culvert.
68 years to perforation for a 12 gauge metal culvert.
87 years to perforation for a 10 gauge metal culvert.
106 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate Calif. Test 417	0.005%
Water Soluble Chloride Calif. Test 422	0.003%

Rosa Bernal
RMB/js

LABORATORY REPORT

Telephone (619) 425-1993 Fax 425-7917 Established 1928

CLARKSON LABORATORY AND SUPPLY INC.
350 Trousdale Dr. Chula Vista, Ca. 91910 www.clarksonlab.com
ANALYTICAL AND CONSULTING CHEMISTS

Date: September 12, 2024
Purchase Order Number: 21-13629
Sales Order Number: 64940
Account Number: GEOE

To:
--
Geotechnical Exploration, INC.
7420 Trade Street
San Diego, Ca 92121

Laboratory Number: SO1396-2 Customers Phone: 858-549-7222

Sample Designation:
--
One soil sample received on 09/04/24 at 12:00pm,
taken from Balazs 21-13629 marked as Sample 2.

Analysis By California Test 643, 1999, Department of Transportation Division of Construction, Method for Estimating the Service Life of Steel Culverts.

pH 5.8

Water Added (ml)	Resistivity (ohm-cm)
10	35000
5	11000
5	6700
5	3700
5	2500
5	1800
5	1700
5	1900
5	2000

14 years to perforation for a 16 gauge metal culvert.
18 years to perforation for a 14 gauge metal culvert.
25 years to perforation for a 12 gauge metal culvert.
32 years to perforation for a 10 gauge metal culvert.
39 years to perforation for a 8 gauge metal culvert.

Water Soluble Sulfate	Calif. Test 417	0.004%
Water Soluble Chloride	Calif. Test 422	0.003%

Rosa Bernal
RMB/js

APPENDIX D

REGIONAL GEOLOGIC DESCRIPTION

In the Coastal Plain region, 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. 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 Nación 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 Range forms 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).

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, now the California Geological Survey (CGS), an "active" fault, described by CGS (2018) as a Holocene-Active fault, is one that has had (ground) surface displacement within Holocene time, the last 11,700. In addition, "potentially active fault" has been amended to Pre-Holocene fault: a fault whose recency of past movement is older than 11,700 years, and thus does not meet the criteria of Holocene-Active fault as defined in the State Mining and Geology Board regulations.

For the City of San Diego, the lead agency for this project, a three-tier fault classification is used as follows:

- *Active Faults*: Faults that have demonstrable surface displacement during Holocene time.
- *Potentially Active Faults*: Faults with Quaternary displacement but Holocene surface displacement is indeterminate.
- *Inactive Faults*: Pre-Quaternary faults.

During recent history, prior to April 2010, the San Diego County area has been relatively quiet seismically. The youngest paleoearthquake that cuts the early historical living surface is likely the 1862 San Diego earthquake that had an estimated magnitude of M6 (Legg and Agnew, 1979; Singleton et al., 2019). Paleoseismic trenches at the Presidio Hills Golf Course on the main trace of the Rose Canyon Fault contained evidence for historical ground rupturing earthquakes as recently as 1862 and the mid-1700s. Results of the study also suggest the Rose Canyon Fault has a ~700-800-year recurrence interval (Singleton et al., 2019).

On June 15, 2004, a M5.3 earthquake occurred approximately 45 miles southwest of downtown San Diego (26 miles west of Rosarito, Mexico). 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. 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.

On April 04, 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, although this is the largest event to strike in this area since 1892. The April 04, 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.

This event's aftershock zone extends significantly to the northwest, overlapping with the portion of the fault system that is thought to have ruptured in 1892. Ground motions for the April 04, 2010, main event, recorded at stations in San Diego and reported by the California Strong Motion Instrumentation Program (CSMIP), ranged up to 0.058g. During the 2010 earthquake, the PGA was measured as ranging between 0.013g to 0.074g for rock sites. The PGA for sediment sites varied between 0.148g to 0.815g based on a publication from the American Geophysical Union, Fall Meeting 2010 (refer to Appendix F, attached). The project site is founded on marine sedimentary formational materials and for design purposes, we recommend a PGA value of 0.375 for the project site based on the seismic design maps by SEAOC OSHPD.

On July 07, 2010, a M5.4 earthquake occurred in Southern California at 4:53 pm (Pacific Time) about 30 miles south of Palm Springs, 25 miles southwest of Indio, and 13 miles north-northwest of Borrego Springs. The earthquake occurred near the Coyote Creek segment of the San Jacinto Fault. The earthquake exhibited right lateral slip to the northwest, consistent with the direction of movement on the San Jacinto Fault. It was followed by more than 60 aftershocks of M1.3 and greater during the first hour.

In the last 50 years, there have been four other earthquakes in the magnitude M5.0 range within 20 kilometers of the Coyote Creek segment: M5.8 in 1968, M5.3 on 2/25/1980, M5.0 on 10/31/2001, and M5.2 on 6/12/2005. The biggest earthquake near this location was the M6.0 Buck Ridge earthquake on 3/25/1937.

APPENDIX E

SLAB MOISTURE INFORMATION

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 E1745-09 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 302.2R-06 Guide to Concrete Floor and Slab Construction.

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 E1745-09 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.

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.

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.

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.

APPENDIX F

American Geophysical Union, Fall Meeting 2010 Excerpt on Strong Motion Data of the El Mayor-Cucupah Earthquake of April 4, 2010

The EL Mayor-Cucapah Earthquake of April 4, 2010 (mw 7.2): Main Shock and Aftershocks Relocation and Relevant Aspects of the Strong Motion Data Recorded in the Epicenter Region. (Invited)

Munguia, L.

The El Mayor-Cucapah earthquake (Mw = 7.2) was the largest earthquake to occur in northeastern Baja California since February 1892, when an earthquake of slightly lower magnitude occurred at the northwestern end of the Laguna Salada fault. This event, with epicenter located ~ 40 km south of the city of Mexicali, occurred on April 4, 2010 at 15:40 local time or 22:40 UTC. Thirteen strong motion stations of a network operated by CICESE in Baja California were triggered by the main shock. Those stations are located at distances of 12 to 140 km from the epicenter. Six of them are located on sediments of the Mexicali Valley, at less than 40 km from the epicenter; the other seven stations recorded at larger distances, on granitic rocks of the peninsular ranges of Baja California. In this study, we analyze the digital accelerograms produced by the main shock and by most of the larger aftershocks. At first, the P-wave arrival times measured on the accelerograms were combined with time readings from the closer-to-the-source weak-motion stations to relocate the hypocenters. With this, we attempted to improve the hypocenter locations obtained on the basis of more regional data sets. Due to lack of station coverage to the south of the main-shock's epicenter, this task was particularly important for those aftershocks occurring along the SE extension of the ruptured area. Our located hypocenters had smaller location errors and provide a better insight about the extent of the main-shock rupture to southeast. Concerning the strong motion data, we noted that on sediments the recorded peak ground accelerations (PGA) varied from 0.148 to 0.815 g, while on the rock sites the PGA were in the range 0.013 to 0.074 g. The largest peak acceleration recorded was observed on the vertical component of the MDO station, sited on sediments. For ground velocity and displacement, the peak values from sedimentary sites are between 14 and 61 cm/sec and 9 and 52 cm, respectively. In such instances the larger values were always observed on the horizontal components of ground motion. From these comparisons, an immediate conclusion is that peak strong motion parameters from sediment sites were, on average, up to ten times higher than those from rock sites. Finally, and in addition to a general description of the recorded ground motions, other aspects of the earthquake, like PGA attenuation with distance, spectral composition of the ground motions, nonlinearity features and preliminary estimations of the radiated seismic energy and stress drop will also be briefly addressed in our technical presentation.

Publication:	American Geophysical Union, Fall Meeting 2010, abstract id. T51E-04
Pub Date:	December 2010
Bibcode:	2010AGUFM.T51E..04M
Keywords:	7212 SEISMOLOGY / Earthquake ground motions and engineering seismology; 7215 SEISMOLOGY / Earthquake source observations; 7230 SEISMOLOGY / Seismicity and tectonics; 7294 SEISMOLOGY / Seismic instruments and networks