PRELIMINARY GEOTECHNICAL EVALUATION
COUNTY OF SAN DIEGO TRACT NO. 5573,
VISTA AREA, SAN DIEGO COUNTY, CALIFORNIA
APNS 181-180-56, -84 AND -86

FOR

MS. MARGARET TOMLINSON AND MS. HOLLY MARSHALL
c/o BHA, INC.
5115 AVENIDA ENCINAS, SUITE L
CARLSBAD, CALIFORNIA 92008-4387

W.O. 6814-A-SC FEBRUARY 4, 2015

GeoSoils, Inc.

Geotechnical • Geologic • Coastal • Environmental

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February 4, 2015

W.O. 6814-A-SC

Ms. Margaret Tomlinson and Ms. Holly Marshall
c/o **bHA, Inc.**
5115 Avenida Encinas, Suite L
Carlsbad, California 92660

Attention: Mr. Rod Bradley

Subject: Preliminary Geotechnical Evaluation, County of San Diego Tract No. 5573, Vista Area, San Diego County, California, APNs 181-180 -56, -84 and -86

Dear Mr. Bradley:

In accordance with your request and authorization, GeoSoils, Inc. (GSI) has performed a preliminary geotechnical evaluation of the subject site with respect to the proposed residential development currently shown on the "Tentative Subdivision Map" prepared by bHA, Inc. ([bHA], 2014). The primary purpose of the study was to evaluate the onsite soils and geologic conditions, and their effects on the proposed site development from a geotechnical perspective. A secondary purpose of our evaluation was to reduce the data acquired from our field studies and laboratory testing so that preliminary recommendations for earthwork, foundations, slab-on-grade floors, walls, hardscape and pavements could be provided.

EXECUTIVE SUMMARY

Based on our review of the available data (see Appendix A), field exploration, laboratory testing, and geologic and engineering analysis, the proposed development of the property appears to be feasible from a geotechnical viewpoint, provided the recommendations presented in the text of this report are properly incorporated into the design and construction of the project. The most significant elements of this study are summarized below:

- Based on a review of bHA (2014), it is our understanding that proposed site development consists of preparing the site to receive 15 residential lots with two (2) private roads. The project also involves the installation of underground utilities and improvements to the existing Hollyberry Drive. Cut and fill grading will be necessary to achieve the design grades.

- Earth materials considered unsuitable for the support of settlement-sensitive improvements (i.e., residential structures, underground utilities, walls, hardscape, pavements, etc.) and/or planned engineered fills consist of surficial soil units including; existing undocumented artificial fill, Quaternary-age colluvium, and the near-surface, weathered portions of the Quaternary-age older alluvium as well as weathered portions of the Cretaceous-age granitic and Mesozoic metasedimentary bedrock that underlie the aforementioned surficial soil units. Unweathered granitic and metasedimentary bedrock are considered suitable for the support of settlement-sensitive improvements and/or planned fill in their existing state. Based on the available data, the thickness of unsuitable earth materials across the site is anticipated to range between approximately 1½ and 6 feet. However, localized areas of thicker unsuitable soils cannot be precluded and should be anticipated. Unsuitable soils should be removed to expose unweathered granitic and metasedimentary bedrock, and then be reused as properly engineered fill.

- In general, the seismic refraction data indicates that in the vicinity of the seismic refraction lines shown on Plates 1 and 2, planned excavations for grading should be feasible with a Caterpillar D9 bulldozer or equivalent. However, localized areas requiring blasting and/or rock breaking equipment cannot be precluded and should be anticipated. Non-trenchable conditions using a Caterpillar 235 trackhoe are anticipated to be encountered at depths ranging between approximately 3 to 6 feet below the existing grade. Thus, in order to ease the effort of excavating for underground utilities, overexcavation of street areas during mass grading to 1 foot below the depth of the lowest utility invert should be considered. This is not a geotechnical requirement in streets; however, it may be mandated by the governing agency. Based on all of the above, the need for overexcavation, blasting and/or line shooting, or the use of rock breaking equipment would be anticipated for underground utility excavations within the site.

- It should be noted, that the 2013 California Building Code ([2013 CBC], California Building Standards Commission [CBSC], 2013) indicates that the mitigation of unsuitable soils be performed across all areas of the site covered under a grading permit and not just within the influence of the proposed residential structures. Relatively deep removals may necessitate a special zone of consideration, on perimeter/confining areas. This zone would be approximately equal to the depth of removals, if removals cannot be performed onsite and offsite. For this site, the width of this zone is anticipated to be 1½ to 5 feet, based on the available data. Any settlement-sensitive improvement constructed within this zone, may require deepened foundations, reinforcements, etc., or will retain some potential for settlement and associated distress. This will require proper disclosure to all interested/affected parties, should this condition exist at the conclusion of grading.

- On a preliminary basis, temporary excavations greater than 4 feet, but less than 20 feet in overall height, performed into the onsite earth materials, should conform to CAL-OSHA and/or OSHA requirements for Type "B" soils provided that groundwater and/or running sands are not present. All temporary excavations should be observed by a licensed engineering geologist or geotechnical engineer prior to worker entry. Although not anticipated, based on the available data, if temporary slopes conflict with property boundaries, shoring or alternating slot excavations may be necessary. The need for shoring or alternating slot excavations should be further evaluated during the grading plan review stage.

- Laboratory testing performed on representative samples of the onsite soils indicates the expansion index (E.I.) of the tested sample is 29. This correlates to low expansion potential. Atterberg Limits testing performed on a representative fine-grained soil sample indicates a plasticity index (P.I.) of 9. Thus, the onsite soils are considered non-detrimentally expansive and do not warrant specific structural design or earthwork to mitigate expansive soil-related deformations (i.e., shrinking and swelling) on a preliminary basis. As such, conventional foundation and slab-on-grade floor systems may be utilized for the proposed residential structures. However, based on our experience with nearby sites underlain by relatively similar earth materials, detrimentally expansive soils (i.e., soils possessing an E.I. greater than 20 and a P.I. greater than 14 could encountered during grading. As such, GSI is providing recommendations for post-tensioned foundations systems and earthwork mitigation of detrimentally expansive soils in this document Slab subgrade pre-soaking/pre-moistening is recommended for low expansive soil conditions (E.I. = 21 to 50) with a P.I. up to and including 14. A combination of slab subgrade pre-soaking/pre-moistening and post-tension foundations would be recommended if soils exposed near finish grade of the lots have an E.I. greater than 20 and a P.I. greater than 14.

- Soil pH, saturated resistivity, and soluble sulfate, and chloride testing was performed on a representative sample of the onsite soils. Testing indicates that the analyzed soils are neutral with respect to soil acidity/alkalinity; are moderately corrosive to exposed, buried metals when saturated; possess negligible ("not applicable" per ACI 318-11) sulfate exposure to concrete; and although not negligible; are below action levels for chloride exposure (per State of California Department of Transportation, 2003). Reinforced concrete mix design for foundations, slab-on-grade floors, and pavements should minimally conform to "Exposure Class C1" in Table 4.3.1 of ACI 318-11, as concrete would likely be exposed to moisture. GSI does not consult in corrosion engineering. Therefore, additional comments and recommendations may be obtained from a qualified corrosion engineer based on the level of corrosion protection desired or required for the project, as determined by the project architect, civil engineer, and/or structural engineer.

- GSI did not encountered the regional groundwater table nor evidence of perched water within our subsurface explorations. Thus, regional groundwater is not anticipated to significantly affect the proposed site development. There is potential for perched groundwater to occur both during and following development along boundaries of contrasting permeability and density (i.e., older alluvium/bedrock contacts, sandy/clayey fill lifts, fill/bedrock contacts, etc.), and along geologic discontinuities (i.e., fractures, joints, etc.). The potential for perched water to manifest should be anticipated and disclosed to all interested/affected parties. Perched water manifestation is based on numerous factors including but not limited to site geologic conditions, rainfall, irrigation, broken or damaged wet utilities, etc.

- Our evaluation indicates there are no known active faults crossing the site and the natural slope upon which the site is located has very low susceptibility to deep-seated landslides. Owing to the depth to groundwater and the dense nature of the bedrock materials and older alluvium, the potential for the site to be adversely affected by liquefaction/lateral spreading is considered very low. Site soils are considered erosive. Thus, properly designed site drainage is necessary in reducing erosion damage to the planned improvements.

- The seismic acceleration values and design parameters provided herein should be considered during the design of the proposed development. The adverse effects of seismic shaking on the structure(s) will likely be wall cracks, some foundation/slab distress, and some seismic settlement. However, it is anticipated that the structure will be repairable in the event of the design seismic event. This potential should be disclosed to all interested/affected parties.

- Graded slopes constructed at gradients of 2:1 (horizontal:vertical [h:v]) or flatter to the currently proposed heights are considered grossly and surficially stable assuming proper construction and site drainage, normal conditions of care, maintenance, and average rainfall.

- Excavations into the bedrock materials may generate oversized rock that will require special handling and placement.

- Based on our review, the onsite soils are not classified as prime farmland.

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

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

The opportunity to be of service is greatly appreciated. If you have any questions concerning this report, or if we may be of further assistance, please do not hesitate to contact any of the undersigned.

Respectfully submitted,

GeoSoils, Inc.

Ryan Boehmer
Project Geologist

David W. Skelly
Civil Engineer, RCE 47857

John P. Franklin
Engineering Geologist, CEG 1340

RB/JPF/DWS/jh

Distribution: (4) Addressee (via US Mail and email)

TABLE OF CONTENTS

PRELIMINARY GEOTECHNICAL EVALUATION
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SCOPE OF SERVICES

The scope of our services has included the following:

1. Review of available geologic literature, regional geologic maps, and aerial photographs of the site and near vicinity (see Appendix A).

2. Geologic site reconnaissance, mapping, and subsurface exploration with 10 exploratory test excavations (see Appendix B).

3. Rock hardness evaluation, utilizing three (3) seismic refraction lines (see Appendix C).

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

5. Review of soil survey mapping to evaluate the presence/absence of onsite soils consistent with prime farmland (Appendix E).

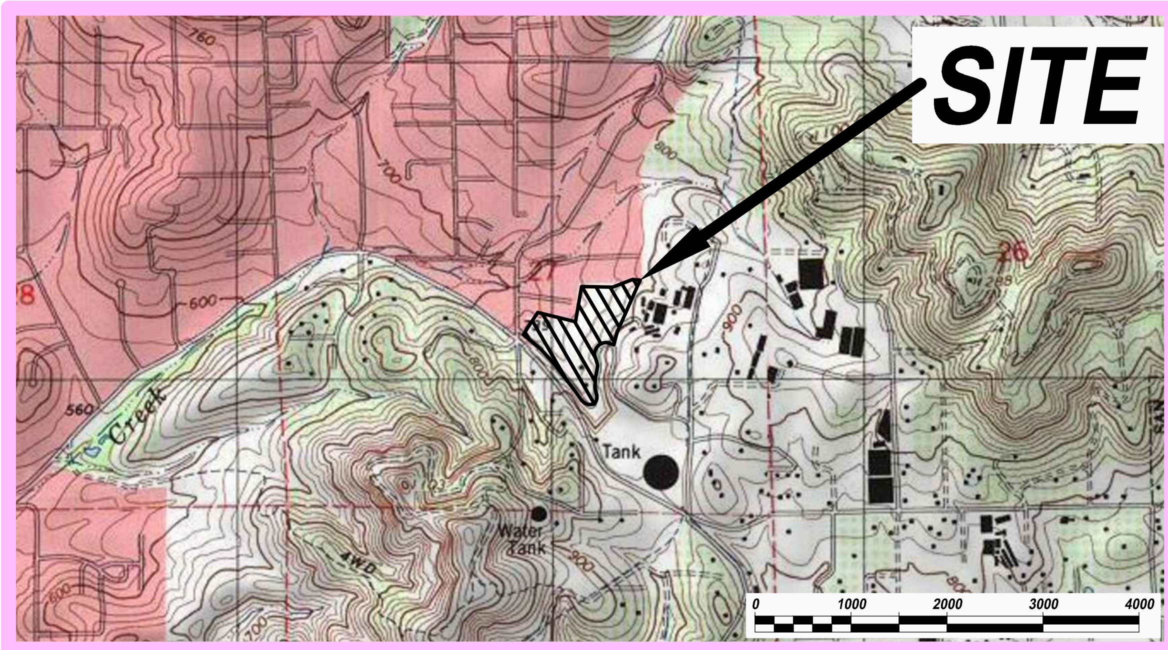
6. Appropriate laboratory testing of representative soil samples (Appendix F).

7. Engineering and geologic analysis of data collected.

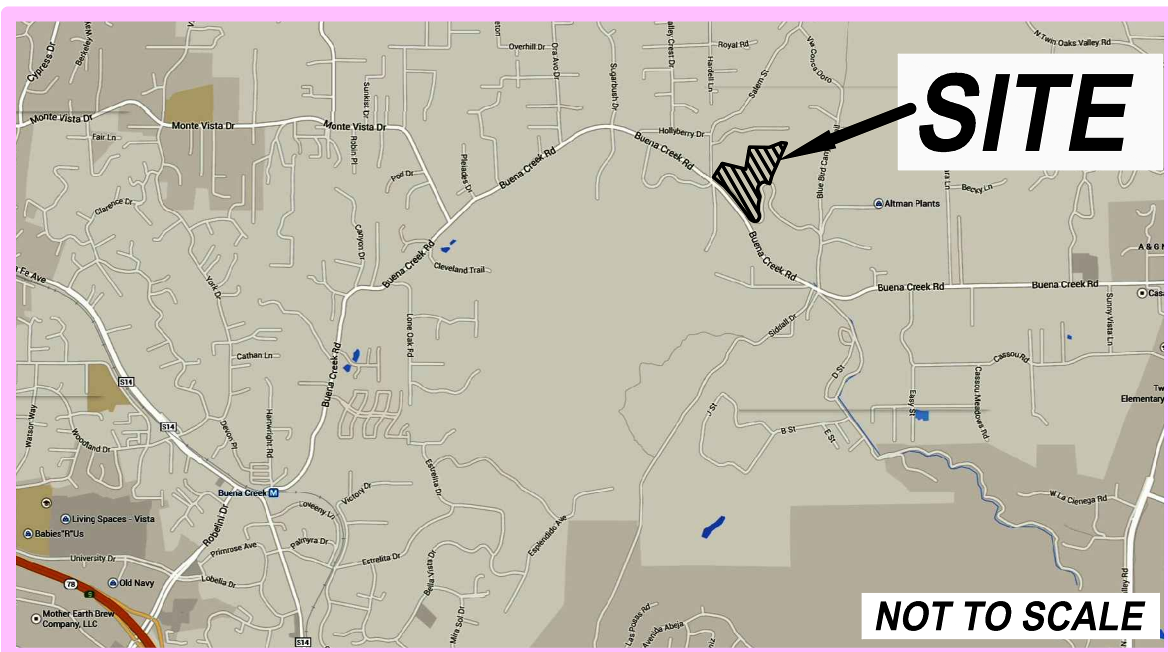
8. Preparation of this summary report.

SITE DESCRIPTION AND PROPOSED DEVELOPMENT

The subject site consists of three (3) irregularly shaped parcels of land located easterly of the Hollyberry Drive and Bluebird Canyon Road intersection, in the Vista area of San Diego County, California (see Figure 1, Site Location Map). The geographic coordinates for the approximate centroid of the subject site is 33.1878˚N, 117.1812˚W. The site is bounded by existing residential development and Hollyberry Drive to the west, by Bluebird Canyon Road to the south, by Bluebird Canyon Road and existing residential development to the east, and by existing residential development to the remaining quadrant. Based on bHA (2014), site elevations range between approximately 709 and 813 feet (unknown datum) for an overall relief of approximately 104 feet. A majority of the site is situated within a northwesterly trending natural drainage. Whereas, the easterly portion of the site is located along a westerly facing slope. In general, the site slopes to the northwest and locally to the southwest and west at moderate to relatively steep gradients. Existing slope gradients are approximately 1.4:1 (horizontal:vertical [h:v]) or



Base Map: TOPO!® ©2003 National Geographic, U.S.G.S. San Marcos Quadrangle, California -- San Diego Co., 7.5 Minute, dated 1996, current, 1999.



Base Map: Google Maps, Copyright 2014 Google, Map Data Copyright 2014 Google

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SITE LOCATION MAP

Figure 1

flatter in the upland portions of the site and approximately 2.5:1 (h:v) or flatter along the lower elevations of the site. Site drainage is controlled by sheet flow runoff predominately directed toward a northwesterly trending natural drainage course that transects the site near its southwesterly boundary. Based on a conversation with the client's representative, the aforementioned natural drainage course also accepts overflow waters from the water tank southeast of the site. Existing site improvements include two (2) abandoned residential structures and associated outbuildings. Site vegetation consists of grasses, weeds, and sparse to locally dense trees. Based on site observations, it appears that a majority of the site was formerly utilized as an orchard.

Based on a review of bHA (2014), GSI understands that proposed site development would include razing the existing structures and preparing the site to receive 15 residential lots with two (2) private cul-de-sac streets, along with a designated detention basin, and five smaller onsite detention/bio-basins on individual lots. Proposed site development would also include the installation of underground utilities, hardscape improvements, and improving the easterly side of Hollyberry Drive where it fronts the site. bHA (2014) indicates that cut and fill grading techniques will be necessary to achieve the design grades, with maximum plan cuts and fills on the order of 15 and 12 feet, respectively. Grade differentials will be transitioned by the construction of graded cut, fill, and fill-over-cut slopes with slope gradients on the order of 2:1 (h:v) or flatter. The maximum height graded cut, fill, and fill-over-cut slopes will be on the order of 26, 7, and 17 feet, respectively.

SITE EXPLORATION

Surface observations and subsurface explorations were performed on November 26, 2014. Seismic refraction surveys were performed on December 8, 2014. All field work was performed by representatives of this office. A survey of line and grade for the subject site was not conducted by this firm at the time of our site reconnaissance. Near-surface soil and geologic conditions were explored with 10 exploratory test pit excavations and three (3) seismic refraction traverses. A rubber-tire backhoe was used to complete the test pit excavations. The approximate locations of the exploratory test pits and seismic refraction lines are shown on the Geotechnical Map (see Plates 1 through 3) which uses bHA (2014) as a base. Logs of the test pits are presented in Appendix B. Limitations to our field investigation included the inability to access the lowest elevation of the site, within the existing residential property located north of the Hollyberry Drive and Buena Creek Road intersection, with the backhoe.

REGIONAL GEOLOGY

The site is located within the central mountain-valley physiographic region of San Diego County. This region consists of ridges and intermontane basins. The basins or valleys range between 500 and 5,000 feet in elevation and are likely due to multiple erosion cycles. The encompassing Peninsular Ranges Geomorphic Province is characterized as elongated mountain ranges and valleys that trend northwesterly (Norris and Webb, 1990). This geomorphic province extends from the base of the east-west aligned Santa Monica - San Gabriel Mountains, and continues south into Baja California, Mexico. The mountain ranges within this province are underlain by basement rocks consisting of pre-Cretaceous metasedimentary rocks, Jurassic metavolcanic rocks, and Cretaceous plutonic (granitic) rocks.

The San Diego region is underlain by a thick sequence of forearc and forearc-basin Jurassic and Cretaceous andesitic flows, sedimentary, volcaniclastic breccias, and marine and metasedimentary rocks partly intruded by Peninsular Ranges batholithic rocks of Cretaceous age in their oldest part (Kennedy and Tan, 2005). Deposition of sedimentary rocks began occurring during the Cretaceous Period and Cenozoic Era in the continental margin of the forearc basin. Sediments, derived from Cretaceous-age plutonic rocks and Mesozoic-era metasedimentary and metavolcanic rocks, were deposited during the Tertiary Period (Eocene-age) into the narrow, steep, coastal plain and continental margin of the basin. These rocks have since been uplifted, eroded, and deeply incised. During early Pleistocene time, a broad coastal plain was created near the coast from the deposition of marine terrace deposits (currently termed “paralic deposits”). During mid- to late-Pleistocene time, this plain was uplifted, eroded and incised. Alluvium derived from the weathered rocks have since been deposited along the lower slopes and stream channels, and young marine sediments are currently being deposited/eroded within coastal and beach areas.

Regional geologic mapping by Kennedy and Tan (2005) indicates that the site is underlain by Mesozoic-age undivided metasedimentary and metavolcanic bedrock. Kennedy and Tan (2005) describe this bedrock unit as a wide variety of low- to high-metamorphic grade metavolcanic and metasedimentary rocks that are mostly volcaniclastic breccia and metaandesitic flows, tuffs, and tuff-breccia.

SITE GEOLOGIC UNITS

The site geologic units encountered during our subsurface investigation and site reconnaissance included small, localized areas of undocumented artificial fill, Quaternary-age colluvium (topsoil), Quaternary-age older alluvium, Cretaceous-age granitic bedrock, and Mesozoic-age undivided metavolcanic/metasedimentary bedrock (weathered and unweathered). The earth materials are generally described below from the youngest to the oldest. The distribution of these materials across the site is shown on Plates 1 through 3.

Artificial Fill - Undocumented (Map Symbol - Afu)

Undocumented artificial fill was encountered at the surface in Test Pits TP-1 through TP-3 to depths ranging between approximately ½-foot to 1 foot below the existing grade. It was also observed at the surface along the westerly embankment for Bluebird Canyon Road. The fill encountered in the test pits consisted of brown silty sand with gravel and brown and dark brown clayey sand with gravel. The fill was generally dry and loose, and locally contained organic matter (roots and rootlets) as well as pore space. The fill is considered unsuitable for the support of settlement-sensitive improvements and/or new planned fills in its existing state.

Quaternary-age Colluvium (Not Mapped)

Quaternary-age colluvium was encountered at the surface in Test Pits TP-4 through TP-10, and generally consisted of brown, strong brown, and dark brown clayey sand; dark brown sandy clay; and brown silty sand. The colluvium was dry, but locally became damp with depth, and loose, and in some areas became stiff with depth. The colluvium was generally porous and locally desiccated. The thickness of the colluvium, encountered in our test pits, was on the order of 1½ to4 feet. The colluvium is considered potentially compressible in its existing state and therefore should not be used to support settlement-sensitive improvements and/or new planned fills without mitigation.

Quaternary-age Older Alluvium (Map Symbol - Qoal)

Quaternary-age older alluvium was encountered beneath the fill and colluvium in Test Pits TP-1 through TP-6. The older alluvium generally consisted of dark brown, gray brown, and dark yellowish brown clayey sand with local trace amounts of gravel; and dark yellowish brown, brown, olive brown, and dark brown silty sand with local trace amounts of clay and gravel. The older alluvium also locally contained subangular fragments of metasedimentary rocks. The older alluvium was generally dry near its upper limits but became damp to moist with depth. The older alluvium was also loose to medium dense with visible pore space in its upper portions but was void of porosity, slightly cemented, and locally became dense with depth. In Test Pits TP-1, TP-2, and TP-4, GSI encountered difficulty when excavating into the older alluvium with a rubber-tire backhoe. Where weathered and porous in the upper approximately 1½ to 3 feet, the older alluvium is considered potentially compressible in its existing state and therefore should not be used for the support of settlement-sensitive improvements and/or new planned fills, without treatment. Unweathered older alluvium is considered suitable for the support of settlement-sensitive improvements and new planned fills in its existing state. Based on the available subsurface data, suitable older alluvium occurs at approximate depths of 3 to 6 feet below the existing grades. A review of our Seismic Line SL-1 suggests that the older alluvium may extend to depths of approximately 16½ to 20½ feet below the existing grade.

Cretaceous-age Granitic Bedrock (Map Symbol - Kgr)

Cretaceous-age granitic bedrock was encountered in Test Pits TP-7, TP-8, and TP-10. Based on our understanding of the regional geologic setting, the granitic bedrock encountered in these test pits is likely a batholithic intrusion of the nearly coeval Mesozoic-age metasedimentary bedrock that also underlies the site. As observed in the aforementioned test pits, the granitic bedrock was generally weathered in its upper ½- to 1 foot, and generally consisted of olive gray, olive brown, and brown clayey sand that was damp to moist and medium dense to dense. Unweathered granitic bedrock was encountered at approximately 1½ to 5 feet below the existing grade, and disintegrated into gray brown and brown silty sand and gray brown poorly graded sand upon excavation. Unweathered granitic bedrock was generally dry and medium dense to dense. Weathered portions of the granitic bedrock are considered unsuitable for the support of settlement-sensitive improvements and new planned fills in its existing state without remedial treatment. Whereas, the unweathered portions of the granitic bedrock are considered suitable bearing materials for settlement-sensitive improvements and new planned fills.

Mesozoic-age Metasedimentary Bedrock (Map Symbol - Mzu)

Mesozoic-age metasedimentary bedrock was observed to underlie the surficial earth materials in Test Pit TP-9, and with the exception of the granitic bedrock intrusion, encountered along the northerly site margins is believed to underlie the soil overburden within the remainder of the site. As directly observed in Test Pit TP-9, the metasedimentary bedrock disintegrated to a dark gray silty sand with angular, platy rock fragments. The metasedimentary bedrock was dry and dense. Unweathered metasedimentary bedrock is considered suitable for the support of settlement-sensitive improvements and/or planned fill in its existing state. Although, not encountered in our subsurface explorations, it is possible for weathered metasedimentary bedrock to exist onsite. If weathered metasedimentary bedrock is encountered during grading, it should be evaluated by a GSI representative and if found unsuitable for the support of the proposed settlement-sensitive improvements and/or new planned fills, it should be removed and reused as properly engineered fill.

GEOLOGIC STRUCTURE

The metasedimentary bedrock encountered in Test Pit TP-10 exhibited planar, schistose foliation generally oriented N 50° W/48° NE. The metasedimentary bedrock exposed in an outcrop displayed foliation oriented N 75° W/50° NE and a joint oriented N 18° E/80° NW. The granitic bedrock encountered in Test Pits TP-7, TP-8, and TP-10 was generally massive. The older alluvium was generally thickly bedded. Based on the available data, geologic structure is not considered adverse with respect to the development currently shown on bHA (2014).

ROCK HARDNESS EVALUATION

A seismic refraction survey, consisting of three (3) seismic refraction lines or traverses, was conducted on the site using a Geometrics SmartSeis 12-channel exploration seismograph, with a hammer and plate energy source. The approximate locations of the seismic traverses or lines (SLs) are shown on the enclosed Plates 1 and 2, and the velocity and depth interval results are summarized in Appendix C. An example of the raw seismic data is included as Plate C-1 in Appendix C, and illustrates a forward and split spread shot from the same line.

The first arrival information, shot point locations, geophone locations, and line geometry from each survey are utilized in the computer program SIPwin (Rimrock Geophysics, 2004) which produces time-distance plots for each of the survey lines (see example Plate C-2 in Appendix C). The graphic curves reflect the actual time-distance plots generated by the program, showing the shot points and phone locations. The first curve, from left to right shows the forward spread from the first shot. The second, or split spread shot point creates two curves in opposite directions from the shot in the middle of the spread. The third curve represent the reverse shot from the distant end of the spread. Undulations in time-distance curves can be attributed to a lack of elevation corrections to the raw data, possible minor disturbances from noise (e.g., wind or traffic), decreased energy at distant geophones, and discontinuities in the subsurface.

The velocity-depth models, or cross-sections, generated are included as Plates C-3 through C-5 (see Appendix C) for Seismic Lines SL-1 through SL-3, respectively. As can be seen on these plates, the boundaries between various seismic velocity layers appear to be somewhat undulatory, typical of fractured and weathered metasedimentary rocks where there is an access to the subsurface for air and water, or possibly representative of paleo-drainage features. Fracture density/frequency also contributes to the variation in depth of weathering and therefore differences in seismic velocities.

The data for the surveys performed generally show both three-layer and two-layer cases. Layer boundaries tend to mimic the surface topography, although variations are common depending upon the depth of weathering, fracturing, etc.

Seismic Line SL-1 shows a three-layer case with the uppermost layer being relatively thin (approximately 2½ to 4 feet thick) as would be expected, reflecting the surficial materials (i.e., undocumented fill and colluvium). The average velocity of this layer is 784 feet per second (fps) which is typical for these earth materials. The middle layer in SL-1 is approximately 12½ to 18 feet thick with an average velocity of 3,647 fps, and generally represents older alluvium and possibly moderately to slightly weathered or fractured metasedimentary bedrock. The lowermost layer in SL-1 occurs at depths ranging between approximately 16½ and 20½ feet below the existing ground surface(begs)with an average velocity of 9,264 fps, indicative of generally intact, fresh bedrock.

SL-2 shows a two-layer case. The upper layer in SL-2 ranges between approximately 3 and 5½ feet in thickness with an average velocity of 954 fps, and generally represents surficial materials (i.e., colluvium and highly weathered metasedimentary bedrock). The lower layer in SL-2 occurs at depths ranging between approximately 3 and 5½ feet below the existing grade. The average velocity of Layer 2 is 7,197 fps which likely represents generally intact metasedimentary bedrock with low to moderate fracturing.

SL-3 also shows a two-layer case. The upper layer in SL-3 ranges between approximately 3½ and 6 feet in thickness with an average velocity of 778 fps, and generally represents surficial materials (i.e., colluvium and highly weathered and decomposed granitic bedrock). The lower layer in SL-3 occurs at depths ranging between approximately 3½ and 6 feet below the existing grade. The average velocity of Layer 2 is 4,295 fps which likely represents decomposed granitic bedrock.

An evaluation has been made of the seismic refraction line data to estimate the approximate depth to non-rippable trenching (i.e., underground utility excavation) and to non-rippable bedrock. Approximate cut-off velocities of ±3,800 and ±6,000 fps were used as a basis for non-rippable trenching (assuming a Caterpillar 235 Trackhoe [a large track-mounted excavator], or equivalent), and non-rippable bedrock (assuming a Caterpillar D9L [a large bulldozer], or equivalent), respectively.

Based upon our experience in this area, and the seismic refraction data obtained, the following table reflects our estimates of the trenchability and rippability in the general vicinity of the seismic refraction line locations. Other interpretations are possible.

SEISMIC LINE NO.	GENERAL RIPPABILITY (ASSUMING A CATERPILLAR D9L BULLDOZER OR CATERPILLAR 235 TRACKHOE, OR EQUIVALENT)
SL-1	Rippable with some difficulty to a depth of ±20½ feet on the south end, ±16½ feet in the middle, and ±19 feet on the north end. Blasting and/or the need for rock breaking equipment is likely below those depths. Non-trenchable below a depth of about ±3 feet on the south end, 4 feet in the middle, and ±2½ feet on the north end. Blasting (i.e., line shooting) and/or the need for rock breaking equipment is likely below these depths.
SL-2	Relative easy ripping to a depth of ±3 feet on the north end, ±3½ to ±4 feet in the middle, and ±5½ feet on the south end. Non-rippable bedrock requiring blasting and/or rock breaking equipment is likely below these depths. Non-trenchable below a depth of about ±3 feet on the north end, ±3½ to ±4 feet in the middle, and ±5½ feet on the south end. Blasting (i.e., line shooting) and/or the need for rock breaking equipment is likely below these depths.
SL-3	Easy to difficult ripping the explored depths (i.e., approximately 20 feet begs). Localized hard zones requiring blasting and/or rock breaking equipment cannot be precluded. Non-trenchable below a depth of about ±3½ feet on the north end, ±5 feet in the middle, and ±6 feet on the south end. Blasting (i.e., line shooting) and/or the need for rock breaking equipment is likely below these depths.

Summary

In general, the seismic refraction data indicates that in the vicinity of the seismic refraction lines shown on Plates 1 and 2, planned excavations for grading should be feasible with a Caterpillar D9 bulldozer or equivalent. However, localized areas requiring blasting and/or rock breaking equipment cannot be precluded and should be anticipated. Non-trenchable conditions using a Caterpillar 235 trackhoe are anticipated to be encountered at depths ranging between approximately 3 to 6 feet below the existing grade. Thus, in order to ease the effort of excavating for underground utilities, overexcavation of street areas during mass grading to 1 foot below the depth of the lowest utility invert should be considered. This is not a geotechnical requirement in streets; however, it may be mandated by the governing agency. Based on all of the above, the need for overexcavation, blasting and/or line shooting, or the use of rock breaking equipment would be anticipated for underground utility excavations within the site.

It should be noted that a light-weight rubber-tire backhoe may experience non-trenchable conditions at the even shallower depths than indicated above. However, the rubber-tire backhoe used in our subsurface exploration was able to extend to depths of approximately 4 to 14 feet with some difficulty. Practical refusal in the granitic bedrock was encountered at an approximate depth of 4 feet in Test Pit TP-10.

Bedrock excavations from the surface downward may generate oversize rock. The bulk of the materials derived from the weathered portion of the bedrock (up to and including the ±3,800 to 6,000 fps cut-off) are anticipated to disintegrate to approximately 12 to 24 inches and smaller constituents. Any oversize materials generated would require special handling for use in fills, and should not be placed within 10 feet of finish grade or used as backfill in utility trenches. Governing agencies may require smaller constituents within streets or utility line backfill.

GROUNDWATER

GSI did not encounter the regional groundwater table nor evidence of perched water during our field exploration. The elevation of the groundwater table at the subject site is anticipated to generally be coincident with the flowline of Buena Creek to the northwest of the site. According to the United States Geological Survey, the flowline of this creek in vicinity to the project site is approximately 700 feet NGVD29, or approximately 9 feet below the lowest site elevation. Buena Creek is likely a perched alluvial aquifer and the groundwater elevation within this aquifer likely fluctuates depending on contributions from precipitation and irrigation. The regional groundwater table is likely coincident with sea level, based on the available data.

Groundwater is not expected to be a major factor in site development. However, due to the nature of the site earth materials, seepage and/or perched groundwater conditions may develop throughout the site in the future, both during and subsequent to development, especially along boundaries of contrasting permeabilities and densities (i.e., sandy/clayey fill lifts, fill/bedrock contacts, foliation, fractures, discontinuities, etc.), and should be anticipated. The manifestation of perched water is the result of numerous factors including site geologic conditions, rainfall, irrigation, broken or damaged wet utilities, etc. This potential should be disclosed to all interested/affected parties.

Due to the potential for post-development perched water to manifest near the surface, owing to as-graded permeability contrasts, more onerous slab design is necessary for any new slab-on-grade floor (State of California, 2015). Recommendations for reducing the amount of water and/or water vapor through slab-on-grade floors are provided in the "Soil Moisture Considerations" sections of this report. It should be noted that these recommendations should be implemented if the transmission of water or water vapor through the slab is undesirable. Should these mitigative measures not be implemented, then the potential for water or vapor to pass through the foundations and slabs and resultant distress cannot be precluded, and would need to be disclosed to all interested/affected parties.

MASS WASTING/LANDSLIDE SUSCEPTIBILITY

Mass wasting refers to the various processes by which earth materials are moved down slope in response to the force of gravity. Examples of these processes include slope creep, surficial failures, and deep-seated landslides. Creep is the slowest form of mass wasting and generally involves the outer 5 to 10 feet of a slope surface. During heavy rains, such as those in El Niño years, creep-affected materials may become saturated, resulting in a more rapid form of downslope movement (i.e., landslides and/or surficial failures).

According to regional landslide hazard mapping performed by the State of California Department of Conservation - Division of Mines and Geology (Tan and Giffen, 1995), the subject site is located within Relative Landslide Susceptibility Area 3-1 which is characterized as being generally susceptible to landslides. According to Tan and Giffen (1995), slopes within this area are at or near their stability limits and most slopes posses a factor-of-safety at or below 1.5. This is due to a combination of weak earth materials comprising the slopes and the steepness of slopes. Tan and Giffen (1995) indicate that slopes in this area do not currently contain landslides but can be expected to fail, locally, when adversely modified.

Based on our review of regional geologic maps, there is no evidence of landslides at the subject site nor did we observe any geomorphic expressions indicative of significant on-going or past deep-seated instability or mass wasting events (i.e., scarps, hummocky terrain, landslide debris, etc.). Based on our site specific investigation, the onsite earth materials are generally not susceptible to deep-seated instability, under normal conditions.

The surficial onsite soils are considered erosive. Therefore, slopes comprised of these materials may be subject to rilling, gullying, sloughing, and surficial slope failures depending on rainfall severity and surface drainage. However, such risks can be minimized through properly designed and controlled surface drainage and by following the recommendations contained herein.

FAULTING AND REGIONAL SEISMICITY

Local and Regional Faults

Our review indicates that there are no known active faults crossing this site (Jennings and Bryant, 2010), and the site is not within an Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). However, the site is situated in a region of active faulting. These faults include, but are not limited to: the San Andreas fault; the San Jacinto fault; the Elsinore fault; the Coronado Bank fault zone; and the Newport-Inglewood - Rose Canyon fault zone (NIRCFZ). Portions of the nearby NIRCFZ are located in an Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). According to Blake (2000a), the closest known active fault to the subject site is the Rose Canyon fault, located at an approximate distance of 13.5 miles (21.7 kilometers). Portions of the Rose Canyon fault have demonstrated movement in the Holocene Epoch (i.e., last 11,000 years) and therefore, are considered active and located in an Alquist-Priolo Earthquake Fault Zone (Bryant and Hart, 2007). Cao, et al. (2003) indicate that Rose Canyon fault is an "B" fault with a slip rate of 1.5 ($\pm$0.5) millimeters per year, and is capable of producing a maximum magnitude (M_w) 7.2 earthquake.

Surface Fault Rupture

Surface fault rupture is the displacement of the ground surface caused by fault propagation extending to the surface of the earth's crust. Since there are no known faults crossing the site that have exhibited activity in the last 11,000 years (Jennings and Bryant, 2010; Bryant and Hart, 2007), the potential for surface fault rupture to adversely affect the proposed development is considered very low.

Seismicity

Deterministic Maximum Credible Site Acceleration

The acceleration-attenuation relation of Bozorgnia, Campbell, and Niazi (1999) has been incorporated into EQFAULT (Blake, 2000a). EQFAULT is a computer program developed by Thomas F. Blake (2000a), which performs deterministic seismic hazard analyses using digitized California faults as earthquake sources.

The program estimates the closest distance between each fault and a given site. If a fault is found to be within a user-selected radius, the program estimates peak horizontal ground acceleration that may occur at the site from an upper bound ("maximum credible") earthquake on that fault. Site acceleration (g) was computed by one user-selected acceleration-attenuation relation that is contained in EQFAULT.

Based on the EQFAULT program, a peak horizontal ground acceleration from an upper bound event at the site may be on the order of 0.29 g. The computer printouts of pertinent portions of the EQFAULT program are included within Appendix D.

Historical Site Acceleration

Historical site seismicity was evaluated with the acceleration-attenuation relation of Bozorgnia, Campbell, and Niazi (1999), and the computer program EQSEARCH (Blake, 2000b). This program performs a search of the historical earthquake records for magnitude 5.0 to 9.0 seismic events within a 100-kilometer radius, between the years 1800 through July 2013. Based on the selected acceleration-attenuation relationship, a peak horizontal ground acceleration is estimated, which may have effected the site during the specific event listed. Based on the available data and the attenuation relationship used, the estimated maximum (peak) site acceleration during the period 1800 through July 2013 was 0.16 g. A historic earthquake epicenter map and a seismic recurrence curve are also estimated/generated from the historical data. Computer printouts of the EQSEARCH program are presented in Appendix D.

Seismic Shaking Parameters

Based on the site conditions, the following table summarizes the site-specific design criteria obtained from the 2013 CBC (CBSC, 2013), Chapter 16 Structural Design, Section 1613, Earthquake Loads. The computer program "U.S. Seismic Design Maps," provided by the United States Geological Survey ([USGS], 2014) was utilized for design.

It should be noted that the seismic design parameters, below, are provided for the average soil properties in the top 100 feet of the soil profile. The Site Class B parameters are reasonably and conservatively justified for competent rock with moderate fracturing and weathering based on a calculated shear wave velocity (an "S" wave) of greater than 2,500 feet per second (fps) in the top 100 feet of the soil profile. Below those depths, the velocities (both "P" and "S") are much greater. Estimates of shear wave velocities were based on the studies by Das (1993), Hunt (1986), and Griffiths and King (1976), which indicate that the "S" wave velocities are about 0.58 of the primary ("P") wave velocities measured in our seismic refraction study. As shown by our Seismic Lines SL-1 and SL-2, the computed shear wave velocities for most of the upper 100 feet of the soil profile at the site are clearly in excess of 2,500 fps. Thus, it is our opinion that Site Class B parameters are reasonable for the seismic design of the proposed residential structures where there will be less than 10 feet of soil between the bottom of the footings and the top of the rock surface. Based on the available subsurface data and the planned fills shown bHA (2014), Site Class B parameters appear suitable for structural design of the proposed residences on proposed Lots 7 and 8 on a preliminary basis.

Although the shear wave velocities we calculated from the “P” wave velocities obtained in our Seismic Line SL-1 are in excess of 2,500 fps, we anticipate that there will be more than 10 feet of soil (i.e., engineered fill and older alluvium) between the bottom of the residential footings and the top of the bedrock at the conclusion of grading for the proposed lots in the general vicinity of Seismic Line SL-1 (i.e., proposed Lots 1 through 6 and 12 through 14). Thus, in accordance with American Society of Civil Engineers (ASCE) Standard ASCE/SEI 7-10 (ASCE, 2010) requirements, Site Class C parameters are recommended for these lots on a preliminary basis.

Based on the Das (1993), Hunt (1986), and Griffiths and King (1976) studies, “S” wave velocities within the upper 100 feet of the profile in the vicinity of Seismic Line SL-3, are in excess of 1,500 fps. As such, Site Class C parameters are considered reasonable for the structural design of the residential structures on proposed Lots 9 through 11 and Lot 15, underlain by granitic bedrock (see Plates 1 through 3). However, based on the available subsurface data and the planned fills shown on bHA (2014), there may be 10 feet or more soil (i.e., engineered fill) between the bottom of the footings and the top of the bedrock on Lots 10, 11, and 15, depending on the location of the building footprint and the depth of the footings. Thus, in accordance with American Society of Civil Engineers (ASCE) Standard ASCE/SEI 7-10 (ASCE, 2010) requirements, Site Class C parameters are recommended for these lots on a preliminary basis. Our review indicates that Site Class C appears suitable as well, for the structural design of the residential structure on proposed Lot 9 on a preliminary basis.

Seismic design should be re-evaluated at the conclusion of grading and prior to foundation construction. Amendments to the Site Class parameters relative to each lot may be necessary and would be based on the as-graded conditions within each lot.

2013 CBC SEISMIC DESIGN PARAMETERS			
PARAMETER	**VALUE**	**VALUE**	**2013 CBC AND/OR REFERENCE**
Site Class	B (Lots 7 and 8)	C (Lots 1-6 and 9-15)	Section 1613.3.2/ASCE 7 (Chapter 20)
Spectral Response - (0.2 sec), S_s	1.048 g	1.048 g	Figure 1613.3.1(1)
Spectral Response - (1 sec), S_1	0.410 g	0.410 g	Figure 1613.3.1(2)
Site Coefficient, F_a	1.000	1.000	Table 1613.3.3(1)
Site Coefficient, F_v	1.000	1.390	Table1613.3.3(2)
Maximum Considered Earthquake Spectral Response Acceleration (0.2 sec), S_{MS}	1.048 g	1.048 g	Section 1613.3.3 (Eqn 16-37)

2013 CBC SEISMIC DESIGN PARAMETERS			
PARAMETER	VALUE	VALUE	2013 CBC AND/OR REFERENCE
Maximum Considered Earthquake Spectral Response Acceleration (1 sec), S_{M1}	0.410 g	0.570 g	Section 1613.3.3 (Eqn 16-38)
5% Damped Design Spectral Response Acceleration (0.2 sec), S_{DS}	0.698 g	0.698 g	Section 1613.3.4 (Eqn 16-39)
5% Damped Design Spectral Response Acceleration (1 sec), S_{D1}	0.273 g	0.380 g	Section 1613.3.4 (Eqn 16-40)
Seismic Design Category	D	D	Section 1613.3.5/ASCE 7 (Table 11.6-1 or 11.6-2)
PGA_M	0.392 g	0.395 g	ASCE 7-10 (Eqn 11.8.1)

GENERAL SEISMIC DESIGN PARAMETERS	
PARAMETER	VALUE
Distance to Seismic Source (Rose Canyon fault)	13.5 mi (21.7 km)[1]
Upper Bound Earthquake (Rose Canyon)	$M_w = 7.2$[2]
[1] - Blake (2000a) [2] - Cao, et al. (2003)	

Conformance to the criteria above for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur in the event of a large earthquake. The primary goal of seismic design is to protect life, not to eliminate all damage, since such design may be economically prohibitive. Cumulative effects of seismic events are not addressed in the 2013 CBC (CBSC, 2013) and regular maintenance and repair following locally significant seismic events (i.e., M_w5.5) will likely be necessary.

LIQUEFACTION POTENTIAL

Liquefaction

Liquefaction describes a phenomenon in which cyclic stresses, produced by earthquake-induced ground motion, create excess pore pressures in relatively cohesionless soils. These soils may thereby acquire a high degree of mobility, which can lead to vertical deformation, lateral movement, lurching, sliding, and as a result of seismic loading, volumetric strain and manifestation in surface settlement of loose sediments, sand boils and other damaging lateral deformations. This phenomenon occurs only below the water table, but after liquefaction has developed, it can propagate upward into overlying non-saturated soil as excess pore water dissipates.

One of the primary factors controlling the potential for liquefaction is depth to groundwater. Typically, liquefaction has a relatively low potential at depths greater than 50 feet and is unlikely and/or will produce vertical strains well below 1 percent for depths below 60 feet when relative densities are 40 to 60 percent and effective overburden pressures are two or more atmospheres (i.e., 4,232 psf [Seed, 2005]).

The condition of liquefaction has two principal effects. One is the consolidation of loose sediments with resultant settlement of the ground surface. The other effect is lateral sliding. Significant permanent lateral movement generally occurs only when there is significant differential loading, such as fill or natural ground slopes within susceptible materials. No such loading conditions exist at the site.

Liquefaction susceptibility is related to numerous factors and the following five conditions should be concurrently present for liquefaction to occur: 1) sediments must be relatively young in age and not have developed a large amount of cementation; 2) sediments must generally consist of medium- to fine-grained, relatively cohesionless sands; 3) the sediments must have low relative density; 4) free groundwater must be present in the sediment; and 5) the site must experience a seismic event of a sufficient duration and magnitude, to induce straining of soil particles. Only about one to possibly two of these necessary five concurrent conditions have the potential to affect the site.

Seismic Densification

Seismic densification is a phenomenon that typically occurs in low relative density granular soils (i.e., United States Soil Classification System [USCS] soil types SP, SW, SM, and SC) that are above the groundwater table. These unsaturated granular soils are susceptible if left in the original density (unmitigated), and are generally dry of the optimum moisture content (as defined by the ASTM D 1557). During seismic-induced ground shaking, these natural or artificial soils deform under loading and volumetrically strain, potentially resulting in ground surface settlements. Some densification of the adjoining un-mitigated properties may influence improvements at the perimeter of the site. Special setbacks and/or foundations may be utilized if significant structures/improvements are placed close to the perimeter of the site. Our evaluation assumed that the current offsite conditions will not be significantly modified by future grading at the time of the design earthquake, which is a reasonably conservative assumption.

Summary

It is the opinion of GSI that the susceptibility of the site to experience damaging deformations from seismically-induced liquefaction and densification is relatively low owing to the dense, nature of the metasedimentary and granitic bedrock as well as the older alluvium that underlie the site in the near-surface. In addition, the recommendations for remedial earthwork and foundations would further reduce any significant liquefaction/densification potential. Some seismic densification of the adjoining un-mitigated site(s) may adversely influence planned improvements at the perimeter of the site. However, given the remedial earthwork and foundation recommendations provided herein, the potential for the planned building to be affected by significant seismic densification or liquefaction of offsite soils may be considered low.

Other Geologic/Secondary Seismic Hazards

The following list includes other geologic/seismic related hazards that have been considered during our evaluation of the site. The hazards listed are considered negligible and/or mitigated as a result of site location, soil characteristics, freeboard, and typical site development procedures:

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

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

FARMLAND CLASSIFICATION

GSI evaluated the onsite soils with respect to their farmland classification. As part of this evaluation, GSI reviewed information pertaining to the onsite soils on the United States Department of Agriculture - Natural Resources Conservation Service (USDA-NRCS) soil survey website (http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx). Our review indicates that the onsite soils consist of the Escondido very fine sandy loam, 5 to 9 percent slopes; the Huerhuero loam, 2 to 9 percent slopes; the Wyman loam, 5 to 9 percent slopes; and the Escondido very fine sandy loam, 15 to 30 percent slopes, eroded. The USDA-NRCS indicates that none of these soils are classified as prime farmland (see Appendix E).

FLOODING HAZARD

According to Flood Insurance Rate Maps (FIRMs) prepared by the Federal Emergency Management Agency (FEMA, 2012), the subject site is located within Zone "X" or outside the 0.2 percent annual chance floodplain. Based on communication with a property owner representative, the onsite northwest-trending natural drainage receives periodic overflow from the water tank located to the southeast of the site. Although minimal, there is some potential that the site could receive flood waters should catastrophic failure of the nearby water tank occur. Site hydrology and inundation potential should be further evaluated by the project design civil engineer.

LABORATORY TESTING

General

Laboratory tests were performed on representative samples of the onsite earth materials in order to evaluate their physical characteristics. The test procedures used and results obtained are presented below.

Classification

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

Moisture-Density Relations

The field moisture contents and field dry densities of relatively undisturbed soil samples were evaluated in the laboratory, in general accordance with ASTM D 2216 and ASTM D 2937. The results of these tests are shown on the Test Pit Logs in Appendix B.

Expansion Potential

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

SAMPLE LOCATION AND DEPTH (FT)	EXPANSION INDEX	EXPANSION POTENTIAL*
TP-3 @ 5-6	29	Low**
* Classification per ASTM D 4829 ** Site soils may range from very low to high expansion potential		

Atterberg Limits

Tests were performed on a representative fine-grained soil sample to evaluate its liquid limit, plastic limit, and plasticity index (P.I.) in general accordance with ASTM D 4318. Test results are presented in the following table and in Appendix F.

SAMPLE LOCATION AND DEPTH (FT)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX
TP-3 @ ½-3	27	18	9

Particle-Size Analysis

An evaluation was performed on a representative, soil sample in general accordance with ASTM D 422-63. The grain-size distribution curve is presented in Appendix F. The testing was utilized to evaluate the soil classification in accordance with the Unified Soil Classification System (USCS). The testing indicates that the tested sample is a sand with trace amounts of silt (SP).

Consolidation Testing

Consolidation testing was performed on relatively undisturbed soil samples in general accordance with ASTM D 2435. Testing was performed on a sample of the older alluvium (Qoal)collected from Test Pit TP-3 at a depth of approximately 3 feet begs (TP-3 @ 3') and TP-4 at a depth of approximately 7 feet begs (TP-4 @ 7'). During testing, both samples were inundated with water while being subjected to a 1,000 pound per square foot (psf) load. Sample TP-3 @ 3' demonstrated approximately 0.06 percent of strain (compression) prior to inundation and then exhibited approximately 1.6 percent of strain (swell) following inundation and prior to additional loading. This sample then displayed approximately 2.5 percent of strain (compression) when loaded to 8,000 psf. When the load was removed, the sample demonstrated approximately 2.6 percent of strain (rebound). Sample TP-4 @ 7' exhibited approximately 0.19 percent of strain (compression) prior to inundation and then displayed approximately 0.05 percent of strain (swell) following inundation and prior to additional loading. This sample then demonstrated approximately 1.2 percent of strain (compression) when loaded to 8,000 psf. When the load was removed, the sample exhibited approximately 0.7 percent of strain (rebound). The consolidation test results are presented in Appendix F.

Direct Shear

Shear testing was performed on a relatively undisturbed sample of the site earth materials in general accordance with ASTM D 3080. The shear testing results are provided in the following table.

SAMPLE LOCATION AND DEPTH (FT)	PRIMARY		RESIDUAL	
	COHESION (PSF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)	FRICTION ANGLE (DEGREES)
TP-5 @ 6	495	30	503	26

Saturated Resistivity, pH, and Soluble Sulfates, and Chlorides

GSI conducted sampling of onsite earth materials for general soil corrosivity and soluble sulfates, and chlorides testing. Test results are presented in Appendix F and the following table:

SAMPLE LOCATION AND DEPTH (FT)	pH	SATURATED RESISTIVITY (ohm-cm)	SOLUBLE SULFATES (% by weight)	SOLUBLE CHLORIDES (ppm)
TP-3 @ ½-3	7.11	2,100	0.0060	140

Corrosion Summary

Laboratory testing indicates that tested sample of the onsite soils are neutral with respect to soil acidity/alkalinity; are moderately corrosive to exposed, buried metals when saturated; present negligible ("not applicable" per ACI 318-11) sulfate exposure to concrete; and slightly elevated chloride levels. Reinforced concrete mix design for foundations, slab-on-grade floors, and pavements should minimally conform to "Exposure Class C1" in Table 4.3.1 of ACI 318-11, as concrete would likely be exposed to moisture. It should be noted that GSI does not consult in the field of corrosion engineering. Therefore, additional comments and recommendations may be obtained from a qualified corrosion engineer based on the level of corrosion protection required for the project, as determined by the project architect, civil engineering, and/or structural engineer.

EMBANKMENT FACTORS (SHRINKAGE/BULKING)

The volume change of excavated materials upon compaction as engineered fill is anticipated to vary with material type and location. The overall earthwork shrinkage and bulking may be approximated by using the following parameters:

Undocumented Artificial Fill . . . . 5% to 10% shrinkage
Quaternary Colluvium . . . . 5% to 10% shrinkage
Quaternary Older Alluvium . . . . 8% shrinkage to 2% bulking

Bedrock

25% rock/75% earth . . . . 4% to 12% shrinkage
50% rock/50% earth . . . . 1% to 9% shrinkage
75% rock/25% earth . . . . 7% to 17% bulking
100% rock (drill and shoot zones) . . . . 33% bulking

It should be noted that the above factors are estimates only, based on preliminary data. Undocumented fill and colluvium may achieve higher shrinkage if organics or clay content is higher than anticipated. Final earthwork balance factors could vary. In this regard, it is recommended that balance areas be reserved where grades could be adjusted up or down near the completion of grading in order to accommodate any yardage imbalance for the project.

PRELIMINARY CONCLUSIONS AND RECOMMENDATIONS

Based on our field exploration, laboratory testing, and geotechnical engineering analysis, it is our opinion that the site appears suitable for the proposed residential development from a geotechnical engineering and geologic viewpoint, provided that the recommendations presented in the following sections are properly incorporated into the design and construction phases of site development. The primary geotechnical concerns with respect to the currently proposed development are:

- Earth materials characteristics and depth to competent bearing material.
- On-going expansion/corrosion potentials of the onsite site soils.
- Rock hardness/excavation difficulty.
- Potential to generate oversize materials that may require special handling.
- Potential for perched groundwater to occur during and after development.
- Non-structural zone on un-mitigated perimeter conditions (improvements subject to distress) or on existing fill/backfill (if not removed and recompacted).
- Temporary and permanent slope stability.
- Regional seismic activity.

The recommendations presented herein consider these as well as other aspects of the site. The engineering analyses, performed, concerning site preparation and the recommendations presented herein have been completed using the information provided and obtained during our field work. In the event that any significant changes are made to proposed site development, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and the recommendations of this report are evaluated or modified in writing by this office. Foundation design parameters are considered preliminary until the foundation design, layout, and structural loads are provided to this office for review.

1. Geotechnical observation, and testing services should be provided during earthwork to aid the contractor in removing unsuitable soils and in his effort to compact the fill.

2. Geologic observations should be performed during any grading to verify and/or further evaluate geologic conditions. Although unlikely, if adverse geologic structures are encountered, supplemental recommendations and earthwork may be warranted.

3. All undocumented fill, colluvium, near-surface older alluvium, and weathered portions of the granitic and metasedimentary bedrock are considered potentially compressible in their existing state and therefore, should not be used for the support of planned settlement-sensitive improvements (i.e., residential structure, underground utilities, pavements, walls, etc.) and/or new planned fills. These earth materials should be removed and reused as properly engineered fill during grading.

4. In general, remedial grading excavations for the removal and re-compaction of potentially compressible, near-surface earth materials are anticipated to be on the order of ½ foot to 6 feet across a majority of the site. However, local deeper remedial grading excavations cannot be precluded and should be anticipated. Remedial grading excavations should be completed below a 1:1 (h:v) projection down from the bottom, outermost edge of proposed settlement-sensitive improvements and/or limits of new planned fills unless constrained by property lines or existing structures to remain during and following construction.

5. Laboratory testing indicates that the expansion index of representative sample of the onsite earth materials is 29. This correlates to a low expansion potential. Atterberg limits testing performed on a fine-grained soil sample indicates a P.I. of 9. Thus, the available data indicates that the majority of onsite soils are non-detrimentally expansive. Specific structural design is not warranted for non-detrimentally expansive soil conditions. However, slab subgrade pre-soaking/pre-wetting is recommended for lots where near-finish grade soils have an expansion index greater than 20. Based on our experience with similar, nearby sites, the local occurrence of medium to highly expansive soils at the site cannot be precluded. As such, GSI is providing recommendations for the design and construction of post-tension and mat foundation systems for the structural mitigation of detrimentally expansive soils should they be encountered. Earthwork mitigation recommendations for detrimentally expansive soils are also included in this report.

6. Soil pH, saturated resistivity, soluble sulfate, and chloride testing indicates that a representative sample of the onsite soils is neutral with respect to soil acidity/alkalinity; is moderately corrosive to exposed, buried metals when saturated; possesses negligible ("not applicable" per ACI 318-11) sulfate exposure to concrete; and slightly elevated chloride levels. Reinforced concrete mix design for

foundations, slab-on-grade floors, and pavements should minimally conform to "Exposure Class C1" in Table 4.3.1 of ACI 318-11, as concrete would likely be exposed to moisture. GSI does not consult in corrosion engineering. Therefore, additional comments and recommendations may be obtained from a qualified corrosion engineer based on the level of corrosion protection desired or required for the project, as determined by the project architect and/or structural engineer.

7. The data indicates that in the vicinity of the seismic refraction lines performed at the site, planned excavations for grading generally should be feasible with a Caterpillar D9 bulldozer or equivalent. However, localized areas requiring blasting and/or rock breaking equipment cannot be precluded and should be anticipated. Non-trenchable conditions using a Caterpillar 235 trackhoe are anticipated to be encountered at depths ranging between approximately ½-foot and 6 feet below the existing grade. Thus, in order to ease the effort of excavating for underground utilities, overexcavation of street areas during mass grading to 1 foot below the depth of the lowest utility invert should be considered. This is not a geotechnical requirement in streets; however, it may be mandated by the governing agency. Based on the above, the need for overexcavation, blasting and/or line shooting, or the use of rock breaking equipment would be anticipated for underground utility excavations within the site.

8. Bedrock excavations from the surface downward may generate oversize rock. The bulk of the materials derived from the weathered portion of the bedrock (up to and including the ±3,800 to 6,000 fps cut-off) are anticipated to disintegrate to approximately 12 to 24 inches and smaller constituents. Any oversize materials generated would require special handling for use in fills, and should not be placed within 10 feet of finish grade or used as backfill in utility trench. Governing agencies may require smaller constituents within streets or utility line backfill.

9. In general and based upon the available data to date, regional groundwater is not expected to be encountered during construction of the proposed site improvements nor is it anticipated to adversely affect site development. However, there is potential for perched water conditions to manifest along zones of contrasting permeabilities (i.e., sandy/clayey fill lifts, fill/bedrock contacts, older alluvium/bedrock contacts, discontinuities, etc.) during and after construction. The potential for perched water to occur should be disclosed to all interested/affected parties.

10. It should be noted, that the 2013 CBC (CBSC, 2013) indicates that remedial grading be performed across all areas to be graded, not just within the influence of the proposed residential structures. Relatively deep removals may also necessitate a special zone of consideration, on perimeter/confining areas. This zone would be approximately equal to the thickness of potentially compressible earth materials if remedial grading cannot be performed onsite and offsite. For this site, the width of this zone is anticipated to be ½-foot to 6 feet, based on the available data. Any settlement-sensitive improvement (walls, brow ditches, curbs, streets, flatwork, etc.),

constructed within this zone, may require deepened foundations, reinforcements, etc., or will retain some potential for settlement and associated distress. This will require proper disclosure to all interested/affected parties, should this condition exist at the conclusion of grading.

11. On a preliminary basis, unsupported temporary excavation walls ranging between 4 and 20 feet in gross overall height, completed into existing artificial fill, colluvium, older alluvium, and weathered and unweathered granitic and metasedimentary bedrock should be constructed in accordance with Cal/OSHA guidelines for Type B soils, provided groundwater or running sands are not present. All temporary excavation walls should be observed by a licensed engineering geologist or geotechnical engineer prior to worker entry. Temporary slope gradients may need to be altered to flatter gradients should potentially adverse condition be exposed.

12. Graded 2:1 (h:v) slopes constructed to the currently proposed heights are considered grossly and surficially stable assuming proper construction and site drainage, normal conditions of care, maintenance, and average rainfall.

13. Rockfills may be difficult to excavate, and may not be suitable for some improvements such as pools or spas. These potential conditions should be disclosed to all interested/affected parties.

14. According to the USDA-NRSC soil survey, the onsite soils are not classified as prime farmland.

15. Site soils are considered erosive. As such, the proper control of surface drainage is considered essential in minimizing the adverse effects of erosion on the proposed improvements and slopes.

16. The subject site is susceptible to moderate to strong ground shaking from an earthquake occurring on any of the regional active fault systems. Therefore, the seismicity-acceleration values, provided herein, should be considered during the design and construction of the proposed development.

17. General Earthwork and Grading Guidelines are provided at the end of this report as Appendix G. Specific recommendations are provided below.

EARTHWORK CONSTRUCTION RECOMMENDATIONS

General

Remedial earthwork will be necessary for the support of the planned settlement-sensitive improvements (i.e., residential structures, walls, underground utilities, pavements, etc.). Remedial grading should conform to the guidelines presented in the 2013 CBC, the requirements of the County/City, and the Grading Guidelines presented in Appendix G, except where specifically superceded in the text of this report. In case of conflict, the more onerous code or recommendations should govern. Prior to grading, a GSI representative should be present at the pre-construction meeting to provide additional grading guidelines, if needed, and review the earthwork schedule.

During earthwork construction, all site preparation and the general grading procedures of the contractor should be observed and the fill selectively tested by a representative(s) of GSI. If unusual or unexpected conditions are exposed in the field, they should be reviewed by this office and, if warranted, modified and/or additional recommendations will be offered. All applicable requirements of local and national construction and general industry safety orders, the Occupational Safety and Health Act (OSHA), and the Construction Safety Act should be met. It is the onsite general contractor's and individual subcontractors' responsibility to provide a safe working environment for our field staff who are onsite. GSI does not consult in the area of safety engineering.

GSI also recommends that the contractor(s) take precautionary measure to protect work, especially during the rainy season. Failure to do so may result in additional remedial earthwork.

Demolition/Grubbing

1. Foundations, vegetation, and any miscellaneous deleterious debris generated from the demolition of existing site improvements should be removed from the areas of proposed grading/earthwork.

2. Cavities or loose soils remaining after demolition and site clearance should be cleaned out and observed by the geotechnical consultant. The cavities should be replaced with fill materials that have been moisture conditioned to at least optimum moisture content and compacted to at least 90 percent of the laboratory standard.

Remedial Excavation (Removal of Potentially Compressible Surficial Materials)

Where planned fills or settlement-sensitive improvements are proposed, potentially compressible undocumented artificial fill, colluvium, weathered near-surface older alluvium, and weathered granitic and metasedimentary bedrock should be removed to expose unweathered older alluvium, granitic bedrock, and metasedimentary bedrock. The criteria for suitable older alluvium is the absence of pore space and a field dry density of at least 105 pounds per cubic foot, or a minimum relative compaction of 85 percent of the laboratory standard (ASTM D 1557). Field density testing should be provided on all older alluvium to evaluate suitability prior to fill placement. Removed earth materials may be reused as properly engineered fill provided that major concentrations of organic and/or deleterious materials have been removed prior to placement. In general, the remedial grading excavations to remove potentially compressible soils are anticipated to be on the order of ½-foot to 6 feet across a majority of the site. However, local deeper remedial grading excavations cannot be precluded and should be anticipated. The removal of potentially compressible soils should be performed below a 1:1 (h:v) projection down from the bottom, outermost edge of proposed settlement-sensitive improvements and/or limits of new planned fills. Once the unsuitable soils have been removed, the exposed older alluvium and unweathered bedrock should be lightly scarified approximately 4 to 6 inches, moisture conditioned as necessary to achieve the soil's optimum moisture content and then be re-compacted to at least 90 percent of the laboratory standard (ASTM D 1557) prior to fill placement. All remedial grading excavations should be observed by the geotechnical consultant prior to scarification.

Overexcavation

bHA (2014) indicates cut/fill transitions in most of the planned lots. For uniform foundation support, all cut portions of planned cut/fill transitions lots or parts of the lots where the planned plus remedial fill thickness does not allow for at least 2 feet of engineered fill beneath footings should be overexcavated and replaced with compacted engineered fill. This would require that all suitable older alluvium and bedrock materials, exposed within 2 feet of the lowest foundation element, following the removal of potentially compressible soils, be overexcavated and replaced with engineered fill compacted to at least 90 percent of the laboratory standard (ASTM D 1557). The bottom of the overexcavation should be sloped away toward street areas or the thickening fill wedge, lightly scarified at least 4 to 6 inches, moisture-conditioned as necessary to achieve the soil's optimum moisture content, and then be re-compacted to at least 90 percent of the laboratory standard (ASTM D 1557), prior to fill placement. Overexcavation should be laterally completed to at least 5 feet outside the outermost foundation element of the residential structure. Overexcavations should be observed by the geotechnical consultant prior to scarification. The maximum to minimum fill thickness across the lots should not exceed a ratio of 3:1 (maximum:minimum). Overexcavation of bedrock materials should be considered within underground utility alignments during grading to help facilitate trenching operations for underground utility placement and retaining wall foundation construction. Within underground utility alignments, bedrock materials may be overexcavated to at least 1 foot below the lowest utility invert elevation and be replaced with engineered fill. Along retaining wall alignments, bedrock materials should be overexcavated at least 2 feet below the bottom of the wall footing or 1 foot below the bottom of the shear key and be replaced with engineered fill. Overexcavation for retaining wall footings should be completed for a lateral distance of 2 feet outside the outboard edge of the footing in all directions, or a 1:1 (h:v) downward projection, whichever is greater.

Earthwork Mitigation of Expansive Soils

The following recommendations are intended to comply with the requirements of Section 1808.6.3 of the 2013 CBC if specific structural design for the mitigation of expansive soils (per Section 1808.6.2) will not be incorporated into the project. This mitigative work would include the overexcavation of expansive earth materials to a sufficient depth such that the weighted plasticity index of replacement fills within the influence of the foundation/slab-on-grade floor system will have a P.I. less than 15. The following table provides recommended preliminary overexcavation depths for the use of very low expansive (E.I. 0 to 20) replacement fills with certain assumed plasticity indices. Our evaluation assumes a worst-case scenario that the soils within the upper 15 feet of pad grade, prior to mitigation, have a P.I. of 40. Additional lot-specific exploration and testing should be performed prior to or during grading to further evaluate the P.I. of soils within 15 feet of pad grade and in order to refine the recommended overexcavation depths provided herein. The lateral extent of the overexcavation, outside the building footprint, should be equal to the depth of the overexcavation.

PLASTICITY INDEX (P.I.) OF REPLACEMENT FILLS	OVEREXCAVATION DEPTH BELOW PAD GRADE (FT)
0	7
5	8½
10	11
14	15

Temporary Slopes

On a preliminary basis, unsupported temporary excavation walls ranging between 4 and 20 feet in gross overall height, completed into existing artificial fill, colluvium, older alluvium, and weathered and unweathered granitic and metasedimentary bed rock, should be constructed in accordance with CAL-OSHA guidelines for Type B soils, provided perched water and/or running sands are not present. All temporary excavation walls should be observed by a licensed engineering geologist or geotechnical engineer prior to worker entry. Based on the exposed field conditions, inclining temporary slopes to flatter gradients or the use of shoring may be necessary if adverse conditions are observed. If temporary slopes conflict with property boundaries, shoring or alternating slot excavations may be necessary. The need for shoring or alternating slot excavations could be further evaluated during the grading plan review stage.

Engineered Fill Placement

Engineered fill should be well blended, placed in thin lifts, moisture conditioned, and mixed to achieve 1.1 times the soil's optimum moisture content, and then be mechanically compacted to at least 90 percent of the laboratory standard (ASTM D 1557). Engineered fill placement should be observed and selectively tested for moisture content and compaction by the geotechnical consultant. Fill materials should not contain rock constituents greater than 12 inches in dimension. Keying and benching should be provided on surfaces steeper than 5:1 (h:v) prior to fill placement.

Graded Slopes

It is our opinion that the graded slopes shown on bHA (2014) will be grossly and surficially stable following the completion of construction provided that site drainage is directed away from the tops of slopes and the slope faces are protected with deep-rooted vegetative cover capable of surviving the prevailing climate without only the amount of irrigation water necessary to sustain plant vigor. Our evaluation also assumes regular and periodic care, maintenance, and normal rainfall conditions.

Vegetative cover should be provided as soon as possible following slope construction. In the interim, GSI recommends the slope faces be covered with County/City approved erosion control devices. Graded slope stability should be further evaluated during the grading plan review stage.

Graded fill slopes should be properly keyed and benched, and be compacted to at least 90 percent relative compaction throughout, including the slope face. The outer 7 feet of fill materials used in fill slope construction should have a target cohesion (C) of at least 250 pounds per square foot (psf). All graded cut slopes should be observed by this office following construction. Although not anticipated, if adverse geologic conditions (daylighted, out-of-slope joints/fractures, highly weathered bedrock, thick unsuitable soils, etc.) are noted in the slope face, GSI would provide recommendations for mitigation. Mitigation measures may included but not necessarily be limited to inclining the slope to gradients flatter than any adverse geologic structure, stabilization fills, or the use of an erosion control mat.

Import Fill Materials

Any import fill materials used on this project should possess an E.I. of 20 or less, with a P.I. not exceeding 15. All import fill material should be tested by GSI prior to placement within the site. GSI would also request environmental documentation (e.g., Phase I Environmental Site Assessment) pertaining to export site to evaluate if the proposed import could present an environmental risk to the planned residential development. At least three (3) business days of lead time will be necessary for the required laboratory testing and document review.

PRELIMINARY FOUNDATION RECOMMENDATIONS

General

The foundation design and construction recommendations are based on current laboratory testing and engineering evaluations of onsite earth materials by GSI. The following preliminary foundation construction recommendations are presented as a minimum criteria from a geotechnical engineering perspective. Testing indicates that the expansion index (E.I.) of a representative sample of the onsite earth materials is 29. This corresponds to low expansion potential. Atterberg Limits testing performed on a representative sample of fine-grained earth materials indicates a plasticity index (P.I.) of 9. Thus, the majority onsite earth materials do not meet the criteria of detrimentally expansive as indicated in Section 1803.5.3. of the 2013 CBC. However, based on our experience with similar nearby sites, detrimentally expansive soils may occur locally within the site. Should expansive soils be encountered near finish grade, structural mitigation would be recommended so the residential foundations and slab-on-grade floors could tolerate the shrink/swell effects of expansive soils for conformance with the requirements of Section 1808.6.2 of the 2013 CBC. Alternatively, expansive soils within the influence of the proposed residential foundations and slab-on-grade floors may be removed and replaced with very low expansive soils (E.I. less than 21) with a plasticity index (P.I.) less than 15 as previously discussed. Mitigation of expansive soils should be evaluated on a cost versus benefit basis prior to implementation.

GSI is providing preliminary design and construction recommendations for conventional foundation recommendations for non-detrimentally expansive soil conditions as well as post-tensioned and mat foundation recommendations for detrimental low to high expansive soil conditions (i.e., soils with an E.I. = 21 to 130 and with a P.I. = 15 or greater).

This report presents minimum design criteria for the design of foundations, concrete slab-on-grade floors, and other elements possibly applicable to the project. These criteria should not be considered as substitutes for actual designs by the structural engineer. Recommendations by the project's design-structural engineer or architect, which may exceed the geotechnical consultant's recommendations, should take precedence over the following minimum requirements. The foundation systems recommended herein may be used to support the proposed residences provided they are entirely founded in engineered fill tested and approved by GSI. The proposed foundation systems should be designed and constructed in accordance with the guidelines contained in the 2013 CBC.

In the event that the information concerning the proposed development plan is not correct, or any changes in the design, location or loading conditions of the proposed structures are made, the conclusions and recommendations contained in this report shall not be considered valid unless the changes are reviewed and conclusions of this report are modified or approved in writing by this office.

The information and recommendations presented in this section are not meant to supercede design by the project structural engineer or civil engineer specializing in structural design. Upon request, GSI could provide additional input/consultation regarding soil parameters, as they relate to foundation design.

General Foundation Design

1. The foundation systems should be designed and constructed in accordance with guidelines presented in the 2013 CBC.

2. An allowable bearing value of 2,000 psf may be used in the design of continuous spread footings that maintain a minimum width of 12 inches and extend at least12 inches (below the lowest adjacent grade) into properly engineered fill. An allowable bearing value of 2,000 psf may also be used in the design of isolated spread footings that are at least 24 square inches and extend at least 24 inches (below the lowest adjacent grade) into properly engineered fill. The allowable bearing values may be increased by 20 percent for each additional 12 inches in footing depth to a maximum value of 2,500 psf. These values may be increased by one-third when considering short duration seismic or wind loads. Foundation embedment excludes any landscaped zone, concrete slabs-on-grade, and/or slab underlayment.

3. Passive earth pressure may be computed as an equivalent fluid having a density of 250 pcf, with a maximum earth pressure of 2,500 psf for footings founded into properly engineered, non-detrimentally expansive fill. Lateral passive pressures for shallow foundations within 2013 CBC setback zones or within the influence of detrimentally expansive soils should be reduced following a review by the geotechnical engineer.

4. For lateral sliding resistance, a 0.35 coefficient of friction may be utilized for a concrete to soil contact when multiplied by the dead load.

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

6. All footing setbacks from slopes should comply with Figure 1808.7.1 of the 2013 CBC. GSI recommends a minimum horizontal setback distance of 10 feet as measured from the bottom, outboard edge of the footing to the slope face. Foundations should also extend below a 1:1 (h:v) projection up from the bottom outside edge of remedial grading excavations.

7. Footings for structures adjacent to retaining walls should be deepened so as to extend below a 1:1 (h:v) projection up from the heel of the wall. Alternatively, walls may be designed to accommodate structural loads from buildings or appurtenances.

8. Provided that the earthwork and foundation recommendations in this report are adhered, foundations bearing on approved non-detrimentally expansive, engineered fil overlying dense bedrock should be minimally designed to accommodate a total static settlement of 2 inches and a differential static settlement of 1-inch over a 40-foot horizontal span (angular distortion = 1/480), and up to ½ inch of seismic differential settlement over a 40-foot horizontal span (seismic angular distortion = 1/960).

9. Code-compliant foundations may be conventional-type if soils within the influence of the foundation have an E.I. of 20 or less and a P.I. less than 15. Otherwise post-tension or mat foundation systems should be used.

PRELIMINARY CONVENTIONAL FOUNDATION AND SLAB-ON-GRADE CONSTRUCTION RECOMMENDATIONS

The following recommendations are for building foundation and slab-on-grade floor systems underlain and laterally juxtaposed by non-detrimentally expansive soils (i.e., E.I.<21 and P.I. < 15), where the weighted plasticity index within the upper 15 feet of foundation soils is less than 15. The structural engineer's recommendations may more onerous, based on actual floor loads.

1. Exterior and interior footings should be founded into approved engineered fill at a minimum depth of 12 or 18 inches below the lowest adjacent grade for a one- or two-story floor load, respectively. For one- and two-story floor loads, footing widths should be at least 12 and 15 inches, respectively. Isolated footings, should be at least 24 inches square, and founded at a minimum depth of 24 inches into approved engineered fill. All footings should be minimally reinforced with four No. 4 reinforcing bars, two placed near the top and two placed near the bottom of the footing.

2. All interior and exterior column footings, and perimeter wall footings, should be tied together via grade beams in at least one direction, or two directions if soils with an E.I. ≥ are present. The grade beam should be at least 12 inches square in cross section, and should be provided with a minimum of one No.4 reinforcing bar near the top, and one No.4 reinforcing bar near the bottom of the grade beam. The base of the reinforced grade beam should be at the same elevation as the adjoining footings. A stepped grade beam, constructed per the structural engineer's specifications, may be necessary where the base of footings occur at different elevations.

3. A grade beam, reinforced as previously recommended and at least 12 inches square, should be provided across large (garage) entrances. The base of the reinforced grade beam should be at the same elevation as the adjoining footings. A stepped grade beam, constructed per the structural engineer's specifications, may be necessary where the base of footings occur at different elevations.

4. A minimum concrete slab-on-grade floor thickness of 4.5 inches is recommended. A maximum water to cement ratio of 0.5 is recommended for concrete used in the construction of foundations and slab-on-grade floors.

5. Concrete slabs should be reinforced with a minimum of No. 3 reinforcement bars placed at 18 inches on center, in two horizontally perpendicular directions (i.e., long axis and short axis).

6. The actual thickness and steel reinforcement for concrete slab-on-grade floors should be determined by the project structural engineer, based on the anticipated loading conditions and building use. However, the slab thickness and steel reinforcement, recommended above, are considered minimum guidelines.

7. All slab reinforcement should be supported to ensure proper mid-slab height positioning during placement of the concrete. "Hooking" of reinforcement is not an acceptable method of positioning.

8. Slab subgrade pre-soaking is recommended for all non-detrimentally expansive soil conditions when the expansion index is greater than 20. The Client should pre-soak the slab subgrade materials to at least the soil's optimum moisture content to a minimum depth of 12 inches for a one-story floor load and a minimum depth of 18 inches for a two-story floor load. The slab subgrade moisture content should be evaluated by the geotechnical consultant within 72 hours of the placement of the underlayment sand and vapor retarder.

9. Soils generated from footing excavations to be used onsite should be compacted to a minimum relative compaction of 90 percent of the laboratory standard (ASTM D 1557), whether the soils are to be placed inside the foundation perimeter or in the yard/right-of-way areas. This material must not alter positive drainage patterns that direct drainage away from the structural areas and toward traffic areas or approved drainage facilities.

POST-TENSIONED FOUNDATION SYSTEMS

Post-tension foundations may be used to mitigate the damaging shrink/swell effects of expansive soils on the planned foundations and slab-on-grade floors. The post-tension foundation designer may elect to exceed the minimal recommendations, provided herein, to increase slab stiffness performance. Post-tension (PT) design may be either ribbed or mat-type. The latter is also referred to as uniform thickness foundation (UTF). The use of a UTF is an alternative to the traditional ribbed-type. The UTF offers a reduction in grade beams. That is to say a UTE typically uses a single perimeter grade beam and possible "shovel" footings, but has a thicker slab than the ribbed-type.

The information and recommendations presented in this section are not meant to supercede design by a registered structural engineer or civil engineer qualified to perform post-tensioned design. Post-tensioned foundations should be designed using sound engineering practice and be in accordance with local and 2013 CBC requirements. Upon request, GSI can provide additional data/consultation regarding soil parameters as related to post-tensioned foundation design.

From a soil expansion/shrinkage standpoint, a common contributing factor to distress of structures using post-tensioned slabs is a "dishing" or "arching" of the slabs. This is caused by the fluctuation of moisture content in the soils below the perimeter of the slab primarily due to onsite and offsite irrigation practices, climatic and seasonal changes, and the presence of expansive soils. When the soil environment surrounding the exterior of the slab has a higher moisture content than the area beneath the slab, moisture tends to migrate inward, underneath the slab edges to a distance beyond the slab edges referred to as the moisture variation distance. When this migration of water occurs, the volume of the soils beneath the slab edges expands and causes the slab edges to lift in response. This is referred to as an edge-lift condition. Conversely, when the outside soil environment is drier, the moisture transmission regime is reversed and the soils underneath the slab edges lose their moisture and shrink. This process leads to dropping of the slab at the edges, which leads to what is commonly referred to as the center lift condition. A well-designed, post-tensioned slab having sufficient stiffness and rigidity provides a resistance to excessive bending that results from non-uniform swelling and shrinking slab subgrade soils, particularly within the moisture variation distance, near the slab edges. Other mitigation techniques typically used in conjunction with post-tensioned slabs consist of a combination of specific soil pre-saturation and the construction of a perimeter "cut-off" wall grade beam. Soil pre-saturation consists of moisture conditioning the slab subgrade soils prior to the post-tension slab construction. This effectively reduces soil moisture migration from the area located outside the building toward the soils underlying the post-tension slab. Perimeter cut-off walls are thickened edges of the concrete slab that impedes both outward and inward soil moisture migration.

Slab Subgrade Pre-Soaking

Pre-moistening of the slab subgrade soil is recommended for detrimentally expansive soil conditions. The moisture content of the subgrade soils should be equal to or greater than optimum moisture to a depth equivalent to the perimeter grade beam or cut-off wall depth in the slab areas (typically 12, 18, and 24 inches) for low, medium, and high expansive soil conditions.

Pre-moistening and/or pre-soaking should be evaluated by the soils engineer 72 hours prior to underlayment sand and vapor retarder placement. In summary:

EXPANSION POTENTIAL	PAD SOIL MOISTURE	CONSTRUCTION METHOD	SOIL MOISTURE RETENTION
Low (E.I. = 21-50)	Upper 12 inches of pad soil moisture 2 percent over optimum (or 1.2 times)	Wetting and/or reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.
Medium (E.I. = 51-90)	Upper 18 inches of pad soil moisture 2 percent over optimum (or 1.2 times)	Berm and flood or wetting and reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.
High (E.I. = 91-130)	Upper 24 inches of pad soil moisture 3 percent over optimum (or 1.3 times)	Berm and flood or wetting and reprocessing	Periodically wet or cover with plastic after trenching. Evaluation 72 hours prior to placement of concrete.

Perimeter Cut-Off Walls

Perimeter cut-off walls should be at least 12, 18, and 24 inches deep for low, medium, and high expansive soil conditions, respectively. The cut-off walls may be integrated into the slab design or independent of the slab. The cut-off walls should be a minimum of 6 inches thick (wide). The bottom of the perimeter cut-off wall should be designed to resist tension, using cable or reinforcement per the structural engineer.

Post-Tensioned Foundation Design

The following recommendations for design of post-tensioned slabs have been prepared in general compliance with the requirements of the recent Post Tensioning Institute's (PTI's) publication titled "Design of Post-Tensioned Slabs on Ground, Third Edition" (PTI, 2004), together with it's subsequent addendums (PTI, 2008).

Soil Support Parameters

The recommendations for soil support parameters have been provided based on the typical soil index properties for soils that are very low to high in expansion potential. The soil index properties are typically the upper bound values based on our experience and practice in the southern California area. Additional testing is recommended either during or following grading, and prior to foundation construction to further evaluate the soil conditions within the upper 7 to 15 feet of pad grade. The following table presents suggested minimum coefficients to be used in the Post-Tensioning Institute design method.

Thornthwaite Moisture Index	-20 inches/year
Correction Factor for Irrigation	20 inches/year
Depth to Constant Soil Suction	7 feet or overexcavation depth to bedrock
Constant soil Suction (pf)	3.6
Moisture Velocity	0.7 inches/month
Effective Plasticity Index (P.I.)*	15-45
* - The effective plasticity index should be evaluated for the upper 7 to 15 feet of foundation soils either during or following grading.	

Based on the above, the recommended soil support parameters are tabulated below:

DESIGN PARAMETERS	VERY LOW TO LOW EXPANSION (E.I. = 0-50)	MEDIUM EXPANSION (E.I. = 51-90)	HIGH EXPANSION (E.I. = 91-130)
e_m center lift	9.0 feet	8.7 feet	8.5 feet
e_m edge lift	5.2 feet	4.5 feet	4.0 feet
y_m center lift	0.4 inches	0.5 inches	0.66 inches
y_m edge lift	0.7 inch	1.3 inch	1.7 inches
Bearing Value (1)	1,000 psf	1,000 psf	1,000 psf
Lateral Pressure	250 psf	175 psf	150 psf
Subgrade Modulus (k)	100 pci/inch	85 pci/inch	70 pci/inch
Minimum Perimeter Footing Embedment (2)	12 inches	18 inches	24 inches

(1) Internal bearing values within the perimeter of the post-tension slab may be increased to 1,500 psf for a minimum embedment of 12 inches, then by 20 percent for each additional foot of embedment to a maximum of 2,500 psf.
(2) As measured below the lowest adjacent compacted subgrade surface without landscape layer or sand underlayment.
Note: The use of open bottomed raised planters adjacent to foundations will require more onerous design parameters.

The parameters are considered minimums and may not be adequate to represent all expansive soils and site conditions such as adverse drainage and/or improper landscaping and maintenance. The above parameters are applicable provided the structure has positive drainage that is maintained away from the structure. In addition, no trees with significant root systems are to be planted within 15 feet of the perimeter of foundations. Therefore, it is important that information regarding drainage, site maintenance, trees, settlements, and effects of expansive soils be passed on to future all interested/affected parties. The values tabulated above may not be appropriate to account for possible differential settlement of the slab due to other factors, such as excessive settlements. If a stiffer slab is desired, alternative Post-Tensioning Institute ([PTI] third edition) parameters may be recommended. All exterior columns not supported by the post-tensioned foundation should be supported by 24 square inch isolated footings extending at least 24 inches into approved engineered fill. Exterior column footings should be tied to the post-tensioned foundation with 12 square inch, reinforced grade beams in at least two directions.

MAT FOUNDATIONS

In lieu of using a post-tensioned foundation to resist expansive soil effects, the Client may consider a mat foundation which uses steel bar reinforcement instead of post-tensioned cables. The structural engineer may supercede the following recommendations based on the planned building loads and use. WRI (Wire Reinforcement Institute) methodologies for design may be used. Mat foundations may be incorporate exterior and interior stiffening beams or a uniform thickness slab. Minimum mat embedment should be 12 inches below the lowest adjacent grade.

Mat Foundation Design

The design of mat foundations should incorporate the vertical modulus of subgrade reaction. This value is a unit value for a 1-foot square footing and should be reduced in accordance with the following equation when used with the design of larger foundations. This is assumes that the bearing soils will consist of engineered fills with an average relative compaction of 90 percent of the laboratory (ASTM D 1557), overlying dense bedrock.

$$K_R = K_S\left[\frac{B+1}{2B}\right]^2$$

where: K_S = unit subgrade modulus
K_R = reduced subgrade modulus
B = foundation width (in feet)

The modulus of subgrade reaction (K_S) and effective plasticity index (PI) to be used in mat foundation design for various expansive soil conditions are presented in the following table. The effective plasticity index for the upper 7 to 15 feet of the foundation soils should be performed during or following grading. Lateral pressures for mat foundation design should conform to those previously provided in the "Post-Tensioned Foundation Systems" section of this report.

LOW EXPANSION (E.I. = 21-50)	MEDIUM EXPANSION (E.I. = 51-90)	HIGH EXPANSION (E.I. = 91-130)
K_S = 100 pci/inch, PI < 15	K_S = 85 pci/inch, PI = 25	K_S = 70 pci/inch, PI = 35

All exterior columns not supported by the mat foundation should be supported by 24 square inch isolated footings extending at least 24 inches into approved engineered fill. Exterior column footings should be tied to the mat foundation with 12 square inch, reinforced grade beams in at least 2 directions.

SOIL MOISTURE TRANSMISSION CONSIDERATIONS

GSI has evaluated the potential for vapor or water transmission through new concrete floor slabs, in light of typical floor coverings and improvements. Please note that slab moisture emission rates range from about 2 to 27 lbs/24 hours/1,000 square feet from a typical slab (Kanare, 2005), while floor covering manufacturers generally recommend about 3 lbs/24 hours as an upper limit. The recommendations in this section are not intended to preclude the transmission of water or vapor through the foundation or slabs. Foundation systems and slabs shall not allow water or water vapor to enter into the structure so as to cause damage to another building component or to limit the installation of the type of flooring materials typically used for the particular application (State of California, 2015). These recommendations may be exceeded or supplemented by a water "proofing" specialist, project architect, or structural consultant. Thus, the client will need to evaluate the following in light of a cost vs. benefit analysis (owner expectations and repairs/replacement), along with disclosure to all interested/affected parties. It should also be noted that vapor transmission will occur in new slab-on-grade floors as a result of chemical reactions taking place within the curing concrete. Vapor transmission through concrete floor slabs as a result of concrete curing has the potential to adversely affect sensitive floor coverings depending on the thickness of the concrete floor slab and the duration of time between the placement of concrete, and the floor covering. It is possible that a slab moisture sealant may be needed prior to the placement of sensitive floor coverings if a thick slab-on-grade floor is used and the time frame between concrete and floor covering placement is relatively short.

Considering the E.I. test results presented herein, and known soil conditions in the region, the anticipated typical water vapor transmission rates, floor coverings, and improvements (to be chosen by the Client and/or project architect) that can tolerate vapor transmission rates without significant distress, the following alternatives are provided:

- Concrete slabs, including the garage slab, should be a minimum of 5 inches thick. The project structural engineer may require a thicker slab-on-grade to mitigate expansive soil conditions.

- Concrete slab underlayment should consist of a 15-mil vapor retarder, or equivalent, with all laps sealed per the 2013 CBC and the manufacturer's recommendation. The vapor retarder should comply with the ASTM E 1745 - Class A criteria (i.e., Stego Wrap or approved equivalent), and be installed in accordance with ACI 302.1R-04 and ASTM E 1643.

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

- Concrete slabs, including the garage slab, should be underlain by 2 inches of clean, washed sand (SE $\geq$ 30) above a 15-mil vapor retarder (ASTM E-1745 - Class A, per Engineering Bulletin 119 [Kanare, 2005]) installed per the recommendations of the manufacturer, including all penetrations (i.e., pipe, ducting, rebar, etc.). The manufacturer shall provide instructions for lap sealing, including minimum width of lap, method of sealing, and either supply or specify suitable products for lap sealing (ASTM E 1745), and per code.

  ACI 302.1R-04 (2004) states "If a cushion or sand layer is desired between the vapor retarder and the slab, care must be taken to protect the sand layer from taking on additional water from a source such as rain, curing, cutting, or cleaning. Wet cushion or sand layer has been directly linked in the past to significant lengthening of time required for a slab to reach an acceptable level of dryness for floor covering applications." Therefore, additional observation and/or testing will be necessary for the cushion or sand layer for moisture content, and relatively uniform thicknesses, prior to the placement of concrete.

- For low to high expansive soil conditions (E.I. > 20), the vapor retarder shall be underlain by a capillary break consisting of at least 4 inches of clean crushed gravel with a maximum dimension of ¾ inch (less than 5 percent passing the No. 200 sieve).

- The maximum water to cement ratio of concrete used in foundation and slab-on-grade construction should not exceed 0.50. Additional concrete mix design recommendations should be provided by the structural consultant and/or waterproofing specialist. Concrete finishing and workablity should be addressed by the structural consultant and a waterproofing specialist.

- Where slab water/cement ratios are as indicated herein, and/or admixtures used, the structural consultant should also make changes to the concrete in the grade beams and footings in kind, so that the concrete used in the foundation and slabs are designed and/or treated for more uniform moisture protection.

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

- Additional recommendations regarding water or vapor transmission should be provided by the architect/structural engineer/slab or foundation designer and should be consistent with the specified floor coverings indicated by the architect.

Regardless of the mitigation, some limited moisture/moisture vapor transmission through the slab cannot be entirely precluded and should be anticipated. Construction crews may require special training for installation of certain product(s), as well as concrete finishing techniques. The use of specialized product(s) should be approved by the slab designer and water-proofing consultant. A technical representative of the flooring contractor should review the slab and moisture retarder plans and provide comment prior to the construction of the foundation or improvement. The vapor retarder contractor should have representatives onsite during the initial installation.

CONCRETE MASONRY UNIT (CMU) WALL DESIGN PARAMETERS (IF WARRANTED)

General

The following recommendations for the design and construction of conventional CMU retaining walls have been provided should they be included into the proposed development plan or into future homeowner landscape plans. Recommendations for specialty walls (i.e., crib, earthstone, geogrid, etc.) are also included herein in a subsequent section.

Conventional Retaining Walls

The design parameters provided below assume that either very low expansive soils (typically Class 2 permeable filter material or Class 3 aggregate base) or native onsite materials with an E.I. up to 20 and a P.I. less than 15 are used to backfill any retaining wall. Based on the available data, most of the onsite soils will not meet this criteria. Thus, the potential use of native backfill materials would require significant compliance testing. The type of backfill (i.e., select or native), should be specified by the wall designer, and clearly shown on the plans. Although not anticipated, any building walls, below grade (i.e., basement walls), should be water-proofed. Waterproofing should also be provided for site retaining walls in order to reduce the potential for efflorescence staining.

Preliminary Retaining Wall Foundation Design

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

> **Minimum Footing Embedment** - 24 inches below the lowest adjacent grade (excluding landscape layer, 6 inches).
>
> **Minimum Footing Width -** 24 inches
>
> **Allowable Bearing Pressure** - An allowable bearing pressure of 2,500 psf may be used in the preliminary design of retaining wall foundations provided that the footing maintains a minimum width of 24 inches and extends at least 24 inches into <u>approved engineered fill</u> overlying dense formational materials. This pressure may be increased by one-third for short-term wind and/or seismic loads.
>
> **Passive Earth Pressure** - Lateral pressures in CMU retaining wall foundation design should conform to those previously provided in the "Post-Tensioned Foundation Systems" section of this report, depending on the expansion potential of the foundation soils, with a maximum earth pressure of 2,500 psf.
>
> **Lateral Sliding Resistance** - A 0.35 coefficient of friction may be utilized for a concrete to soil contact when multiplied by the dead load. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one-third.
>
> **Backfill Soil Density** - Soil densities ranging between 105 pcf and 115 pcf may be used in the design of retaining wall foundations. This assumes an average engineered fill compaction of at least 90 percent of the laboratory standard.

Any retaining wall footings near the perimeter of the site will likely need to be deepened into unweathered bedrock. All retaining wall footing setbacks from slopes should comply with the recommendations provided in the "General Foundation Design" section of this report.

Restrained Walls

Any retaining walls that will be restrained prior to placing and compacting backfill material or that have re-entrant or male corners, should be designed for an at-rest equivalent fluid pressure (EFP) of 55 pcf and 65 pcf for select and very low expansive native backfill, respectively. The design should include any applicable surcharge loading. For areas of male or re-entrant corners, the restrained wall design should extend a minimum distance of twice the height of the wall (2H) laterally from the corner.

Cantilevered Walls

The recommendations presented below are for cantilevered retaining walls up to 10 feet high. Design parameters for walls less than 3 feet in height may be superceded by San Diego Regional Standard design. Active earth pressure may be used for retaining wall design, provided the top of the wall is not restrained from minor deflections. An equivalent fluid pressure approach may be used to compute the horizontal pressure against the wall. Appropriate fluid unit weights are given below for specific slope gradients of the retained material. These do not include other superimposed loading conditions due to traffic, structures, seismic events or adverse geologic conditions. When wall configurations are finalized, the appropriate loading conditions for superimposed loads can be provided upon request.

For preliminary planning purposes, the structural consultant should incorporate the surcharge of traffic on the back of retaining walls where vehicular traffic will occur within a horizontal distance equal to "H" from the back of any retaining wall (where "H" equals the height of the retaining wall). The traffic surcharge for light passenger cars, trucks, and vans may be taken as 100 psf/ft in the upper 5 feet of the backfill. For heavy emergency vehicle or multi-axle (HS20) truck traffic, the traffic surcharge should be 300 psf/ft in the upper 5 feet of the backfill. This does not include the surcharge of parked vehicles which should be evaluated at a higher surcharge to account for the effects of seismic loading. Equivalent fluid pressures for the design cantilevered retaining walls are provided in the following table:

SURFACE SLOPE OF RETAINED MATERIAL (HORIZONTAL:VERTICAL)	EQUIVALENT FLUID WEIGHT P.C.F. (SELECT BACKFILL)[2]	EQUIVALENT FLUID WEIGHT P.C.F. (NATIVE BACKFILL)[3]
Level[1]	38	50
2 to 1	55	65

[1] Level backfill behind a retaining wall is defined as compacted earth materials, properly drained, without a slope for a distance of 2H behind the wall, where H is the height of the wall.
[2] SE $\geq$ 30, P.I. < 15, E.I. < 21, and $\leq$ 10% passing No. 200 sieve.
[3] E.I. = 0 to 20, SE $\geq$ 25, P.I. < 15, and $\leq$ 15% passing No. 200 sieve.

Seismic Surcharge

For engineered retaining walls 6 feet or greater in overall height, retaining walls that are incorporated into a building, and/or retaining walls that may pose ingress or egress constraints to the residential structure, GSI recommends that the walls be evaluated for a seismic surcharge (in general accordance with 2013 CBC requirements). The site walls in this category should maintain an overturning Factor-of-Safety (FOS) of approximately 1.25 when the seismic surcharge (increment), is applied. For restrained walls, the seismic surcharge should be applied as a uniform surcharge load from the bottom of the footing (excluding shear keys) to the top of the backfill at the heel of the wall footing. This seismic surcharge pressure (seismic increment) may be taken as 15H where "H" for retained walls is the dimension previously noted as the height of the backfill to the bottom of the footing. The resultant force should be applied at a distance 0.6 H up from the bottom of the footing. For the evaluation of the seismic surcharge, the bearing pressure may exceed the static value by one-third, considering the transient nature of this surcharge. For cantilevered walls the pressure should be an inverted triangular distribution using 15H. Please note that the evaluation of the seismic surcharge is for local wall stability only.

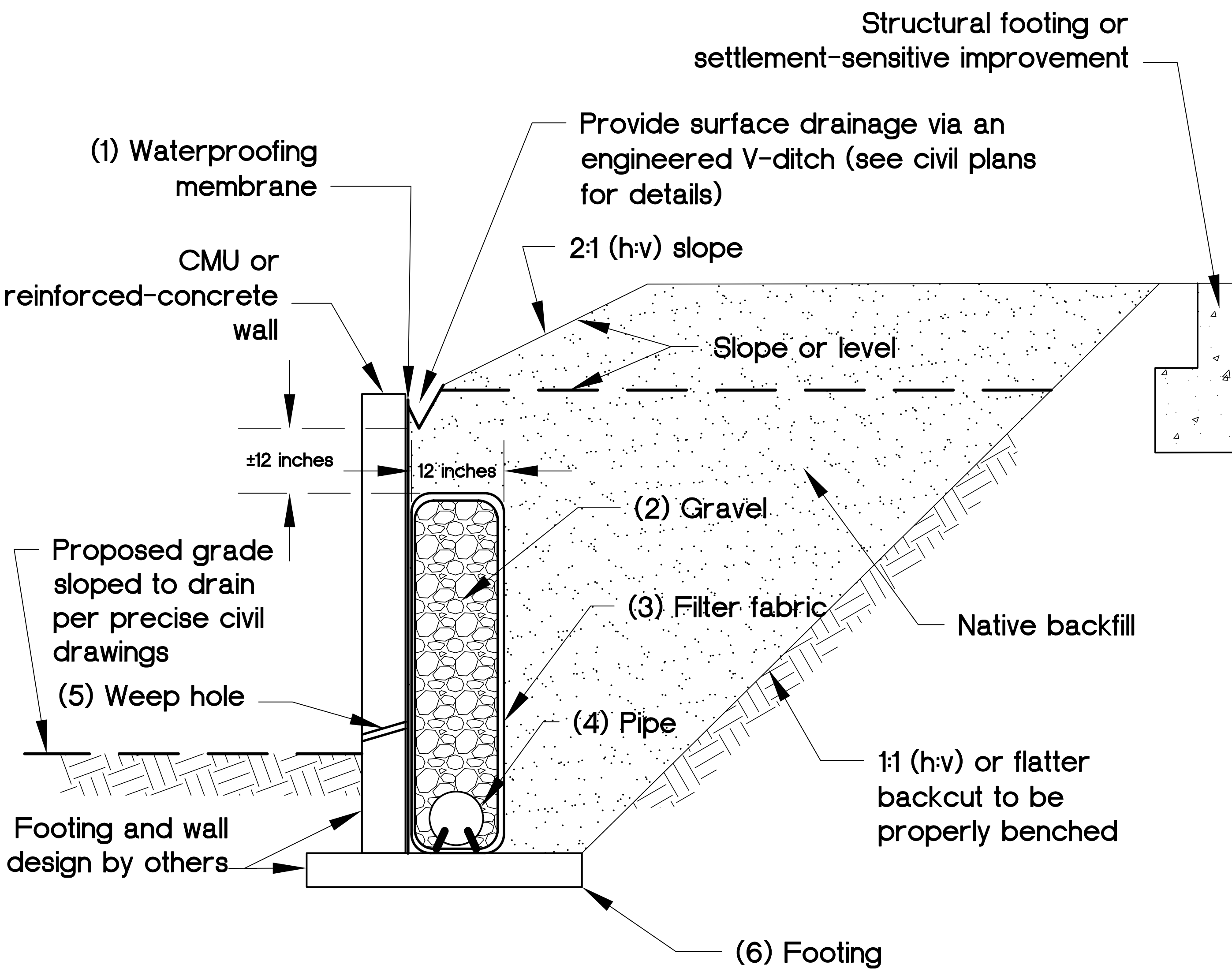
The 15H is derived from a Mononobe-Okabe solution for both restrained cantilever walls. This accounts for the increased lateral pressure due to shakedown or movement of the sand fill soil in the zone of influence from the wall or roughly a 45° - ϕ/2 plane away from the back of the wall. The 15H seismic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

$P_h = ⅜ \bullet a_h \bullet \gamma_t H$

Where:	P_h	=	Seismic increment
	a_h	=	Probabilistic horizontal site acceleration with a percentage of “g”
	γ_t	=	Total unit weight (115 to 125 pcf for site soils @ 90% relative compaction).
	H	=	Height of the wall from the bottom of the footing or point of pile fixity.

Retaining Wall Backfill and Drainage

Positive drainage must be provided behind all retaining walls in the form of gravel wrapped in geofabric and outlets. A backdrain system is considered necessary for retaining walls that are 2 feet or greater in height. Details 1, 2, and 3, present the backdrainage options discussed below. Backdrains should consist of a 4-inch diameter perforated PVC or ABS pipe encased in either Class 2 permeable filter material or ¾-inch to 1½-inch gravel wrapped in approved filter fabric (Mirafi 140 N or equivalent). The backdrain should flow via gravity (minimum 1 percent grade) toward an approved drainage facility. For select backfill, the filter material should extend a minimum of 1 horizontal foot behind the base of the walls and upward at least 1 foot. For native backfill that has up to E.I. = 20, continuous Class 2 permeable drain materials should be used behind the wall. This material should be continuous (i.e., full height) behind the wall, and it should be constructed in accordance with the enclosed Detail 1 (Typical Retaining Wall Backfill and Drainage Detail). For limited access and confined areas, (panel) drainage behind the wall may be constructed in accordance with Detail 2 (Retaining Wall Backfill and Subdrain Detail Geotextile Drain). Materials with an expansion index (E.I.) potential of greater than 20 should not be used as backfill for retaining walls. For more onerous expansive situations, backfill and drainage behind the retaining wall should conform with Detail 3



(1) Waterproofing membrane.

(2) Gravel: Clean, crushed, $^{3}/_{4}$ to $1^{1}/_{2}$ inch.

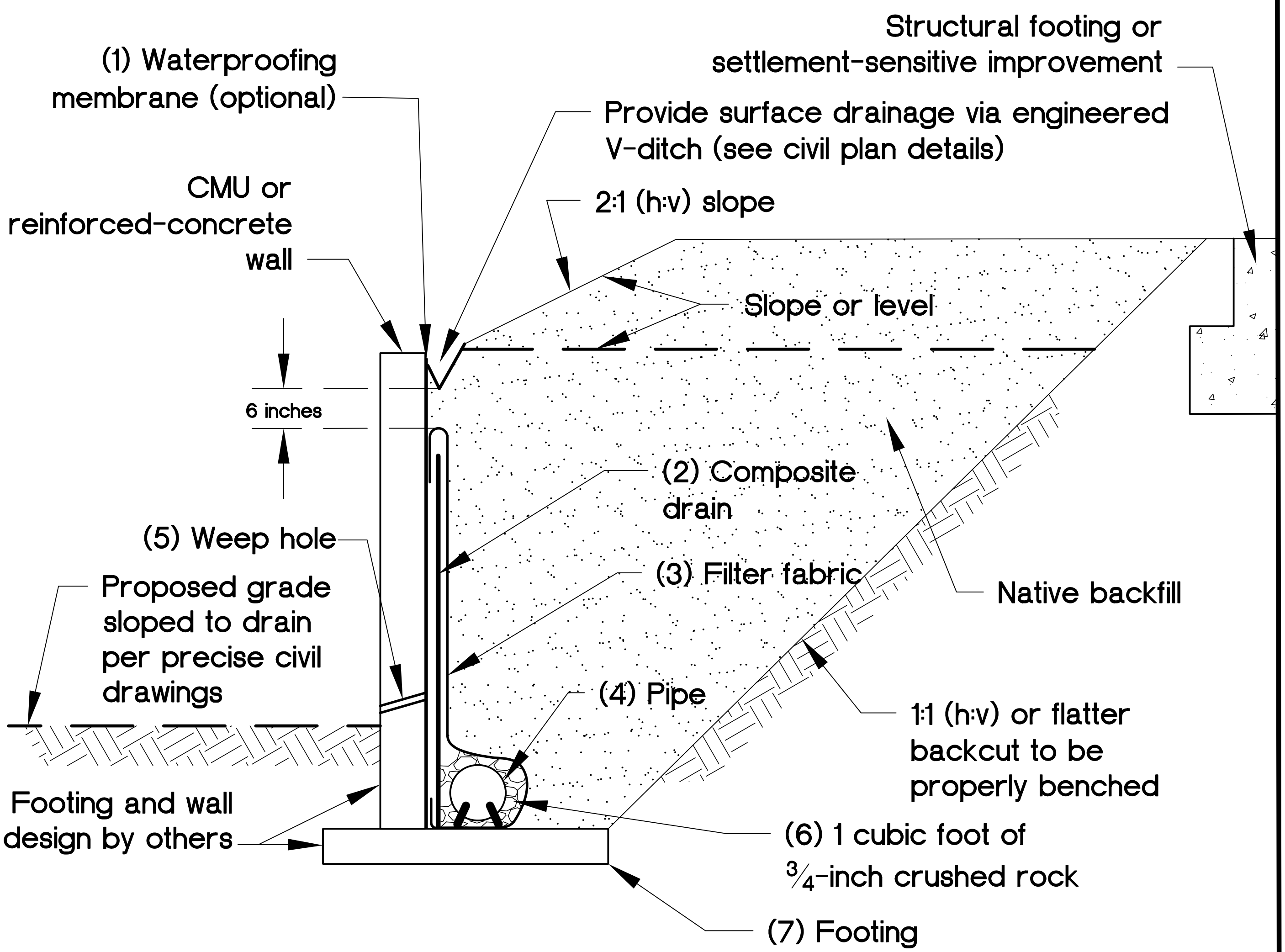
(3) Filter fabric: Mirafi 140N or approved equivalent.

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

(5) Weep hole: Minimum 2-inch diameter placed at 20-foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Footing: If bench is created behind the footing greater than the footing width, use level fill or cut natural earth materials. An additional "heel" drain will likely be required by geotechnical consultant.

GeoSoils, Inc.	RETAINING WALL DETAIL — ALTERNATIVE A	Detail 1



(1) Waterproofing membrane (optional): Liquid boot or approved mastic equivalent.

(2) Drain: Miradrain 6000 or J-drain 200 or equivalent for non-waterproofed walls; Miradrain 6200 or J-drain 200 or equivalent for waterproofed walls (all perforations down).

(3) Filter fabric: Mirafi 140N or approved equivalent; place fabric flap behind core.

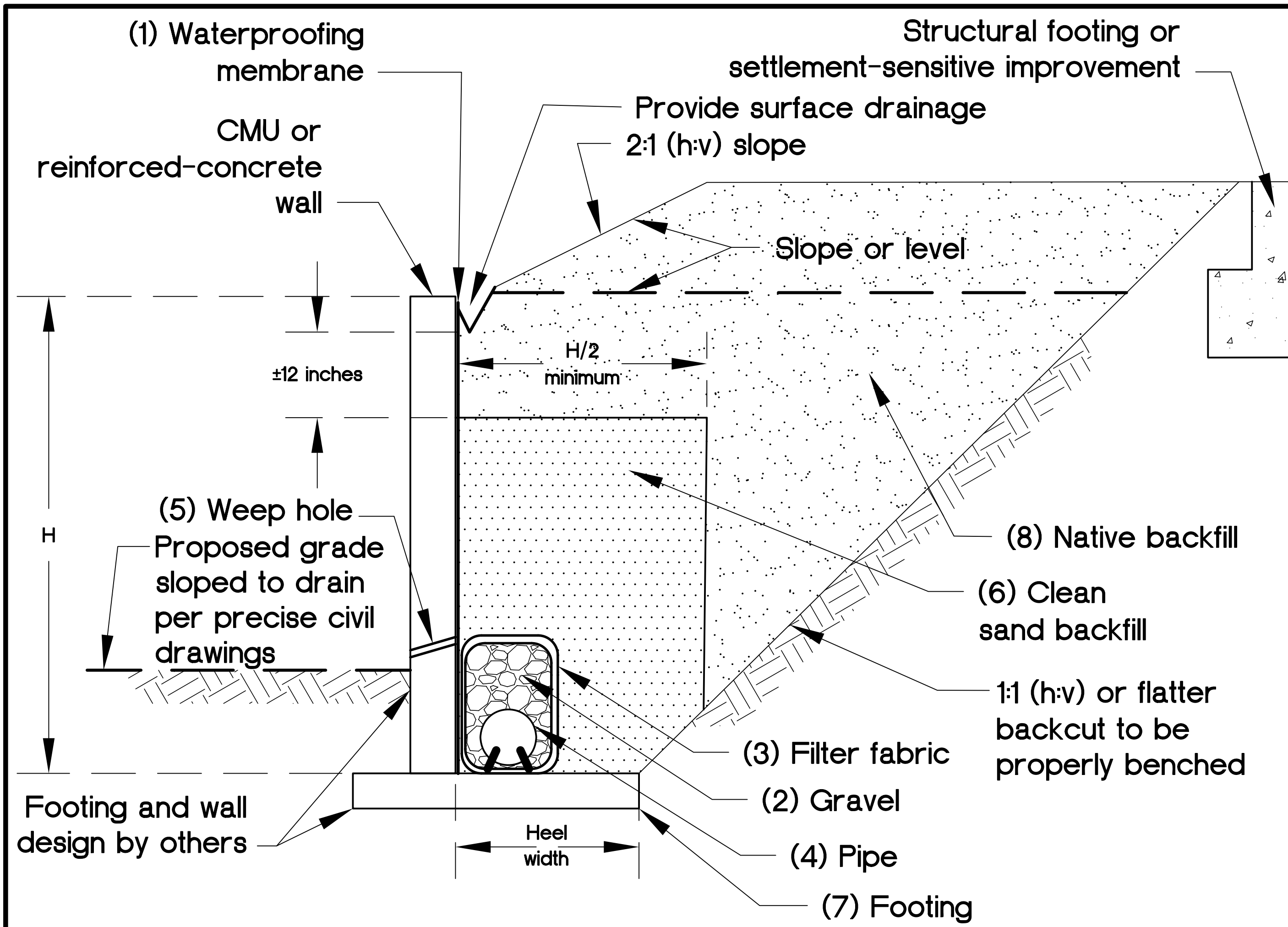
(4) Pipe: 4-inch-diameter perforated PVC, Schedule 40, or approved alternative with minimum of 1 percent gradient to proper outlet point (perforations down).

(5) Weep hole: Minimum 2-inch diameter placed at 20-foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Gravel: Clean, crushed, ¾ to 1½ inch.

(7) Footing: If bench is created behind the footing greater than the footing width, use level fill or cut natural earth materials. An additional "heel" drain will likely be required by geotechnical consultant.

GeoSoils, Inc.	RETAINING WALL DETAIL — ALTERNATIVE B	Detail 2



(1) Waterproofing membrane: Liquid boot or approved masticequivalent.

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

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

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

(5) Weep hole: Minimum 2-inch diameter placed at 20-foot centers along the wall and placed 3 inches above finished surface. Design civil engineer to provide drainage at toe of wall. No weep holes for below-grade walls.

(6) Clean sand backfill: Must have sand equivalent value (S.E.) of 35 or greater; can be densified by water jetting upon approval by geotechnical engineer.

(7) Footing: If bench is created behind the footing greater than the footing width, use level fill or cut natural earth materials. An additional "heel" drain will likely be required by geotechnical consultant.

(8) Native backfill: If E.I. $<$21 and S.E. $\geq$35 then all sand requirements also may not be required and will be reviewed by the geotechnical consultant.

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

(Retaining Wall And Subdrain Detail Clean Sand Backfill). Retaining wall backfill should be moisture conditioned to 1.1 to 1.2 times the soil's optimum moisture content, placed in relatively thin lifts, and compacted to at least 90 percent of the laboratory standard (ASTM D 1557).

Outlets should consist of a 4-inch diameter solid PVC or ABS pipe spaced no greater than ±100 feet apart, with a minimum of two outlets, one on each end. The use of weep holes, only, in walls higher than 2 feet, is not recommended. The surface of the backfill should be sealed by pavement or the top 18 inches compacted with native soil (E.I. ≤ 50). Proper surface drainage should also be provided. For additional mitigation, consideration should be given to applying a water-proof membrane to the back of all retaining structures. The use of a waterstop should be considered for all concrete and masonry joints. Site walls are anticipated to be founded on footings designed in accordance with the recommendations in this report. Should wall footings transition from cut to fill, the civil designer may specify either:

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

b) Increase of the amount of reinforcing steel and wall detailing (i.e., expansion joints or crack control joints) such that a angular distortion of 1/360 for a distance of 2H on either side of the transition may be accommodated. Expansion joints should be placed no greater than 20 feet on-center, in accordance with the structural engineer's/wall designer's recommendations, regardless of whether or not transition conditions exist. Expansion joints should be sealed with a flexible, non-shrink grout.

c) Embed the footings entirely into native formational material (i.e., deepened footings).

If transitions from cut to fill transect the wall footing alignment at an angle of less than 45 degrees (plan view), then the designer should follow recommendation "a" (above) and until such transition is between 45 and 90 degrees to the wall alignment.

TOP-OF-SLOPE WALLS/FENCES/IMPROVEMENTS AND EXPANSIVE SOILS

Expansive Soils and Slope Creep

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

Top of Slope Walls/Fences

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

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

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

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

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

Allowable Axial Capacity:

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

Tip capacity: 3,000 psf for a caisson bearing in engineered fill. 4,500 psf for a caisson bearing in unweathered bedrock. This assumes the absence of water in the drilled shafts and the bottom of the drilled shaft will be free of all loose soil or debris.

PRELIMINARY ASPHALTIC CONCRETE PAVEMENT DESIGN RECOMMENDATIONS

General

The County of San Diego may retain the authority to approve the final structural design sections after subgrade elevations and actual resistance values (R-values) have been obtained at the conclusion of earthwork. For estimation and bidding purposes, the asphaltic concrete pavement section for the planned Streets “A” and “B” and the improvement of Hollyberry Drive(where it fronts the subject site), provided herein, should be considered for preliminary design. Final pavement sections will likely vary throughout the site owing to variable subgrade resistance values (R-values) identified at the conclusion of grading and underground utility trench backfill. Therefore, final pavement sections will need to be re-evaluated following site earthwork and be based on actual R-value testing of soils exposed near subgrade.

The preliminary pavement sections presented in the following table are based on the general Traffic Index (T.I.) values for private cul-de-sac streets (i.e., Streets “A” and “B”) and residential collectors (I.e., Hollyberry Drive), provided by the County of San Diego Department of Public Works (undated and 2012), an assumed subgrade R-value of 20, and the guidelines presented in the latest revision to the California Department of Transportation "Highway Design Manual" fifth edition. GSI does not practice in the field of traffic engineering. Therefore, it is the responsibility of the project civil consultant and/or project traffic engineer to classify the planned roadways. Based on an assumed R-value of 20 and the aforementioned T.I. values, the following preliminary asphaltic concrete pavement designs are presented.

VEHICULAR TRAFFIC CLASSIFICATION	TRAFFIC INDEX (T.I.)(1)	STANDARD PAVEMENT DESIGNS		
		R-VALUE	AC INCHES	CLASS 2 AGGREGATE BASE(2) INCHES
Private Cul-De-Sac Street (Streets "A" and "B")	4.5	20	3.0	6.0
Private Cul-De-Sac Street (Streets "A" and "B") Alternative Pavement Section	4.5	20	4.0	4.0
Residential Collector (Hollyberry Drive)	5.0	20	3.0	7.5
Residential Collector (Hollyberry Drive) Alternative Pavement Section	5.0	20	4.0	5.0

[1] - County of San Diego Department of Public Works (undated and 2012)
[2] - Assumed R-value for Class 2 aggregate base R=78 - Cal-Trans standard Class 2 Aggregate Base.

The preliminary pavement section provided above is intended as a minimum guideline. If thinner or highly variable pavement sections are constructed, increased maintenance and repair could be expected. If the ADT (average daily traffic) or ADTT (average daily truck traffic) increases beyond that intended, as reflected by the T.I. used for design, increased maintenance and repair could be required for the pavement section. Consideration should be given to the increased potential for distress from overuse of paved street areas by heavy equipment and/or construction related heavy traffic (e.g., concrete trucks, loaded supply trucks, etc.), particularly when the final section is not in place (i.e., topcoat). Best management construction practices should be followed at all times, especially during inclement weather.

PAVEMENT GRADING RECOMMENDATIONS

General

All section changes should be properly transitioned. If adverse conditions are encountered during the preparation of subgrade materials, special construction methods may need to be employed. A GSI representative should be present for the preparation of subgrade, aggregate base, and asphaltic concrete.

Subgrade

Within drive lanes and parking areas, all surficial deposits of loose soil material should be removed and re-compacted as recommended. After the loose soils are removed, the bottom is to be scarified to a depth of at least 6 inches, moisture conditioned as necessary and compacted to 95 percent of the maximum laboratory density, as determined by ASTM D 1557.

Deleterious material, excessively wet or dry pockets, concentrated zones of oversized rock fragments, and any other unsuitable materials encountered during grading should be removed. The compacted fill material should then be brought to the elevation of the proposed subgrade for the pavement. The subgrade should be proof-rolled in order to promote a uniform firm and unyielding surface. All grading and fill placement should be observed by the project geotechnical consultant.

Aggregate Base

Compaction tests are required for the recommended aggregate base section. Minimum relative compaction required will be 95 percent of the laboratory maximum density as determined by ASTM D 1557. Base aggregate should be in accordance to the "Greenbook" crushed aggregate base rock (minimum R-value=78).

Paving

Prime coat may be omitted if all of the following conditions are met:

1. The asphalt pavement layer is placed within two weeks of completion of the aggregate base course.

2. Traffic is not routed over completed base before paving

3. Construction is completed during the dry season of May through October.

4. The aggregate base is kept free of debris prior to placement of asphaltic concrete.

If construction is performed during the wet season of November through April, prime coat may be omitted if no rain occurs between completion of the aggregate base course and paving and the time between completion of aggregate base and paving is reduced to three days, provided the aggregate base is free of loose soil or debris. Where prime coat has been omitted and rain occurs, traffic is routed over the aggregate base course, or paving is delayed, measures shall be taken to restore the aggregate base course, and subgrade to conditions that will meet specifications as directed by the geotechnical consultant.

Drainage

Positive drainage should be provided for all surface water to drain towards the area swale, curb and gutter, or to an approved drainage channel. Positive site drainage should be maintained at all times. Water should not be allowed to pond or seep into the ground, such as from behind unprotected curbs, both during and after grading. If planters or landscaping are adjacent to paved areas, measures should be taken to minimize the potential for water to enter the pavement section, such as thickened edges, subdrains, enclosed or lined planters, etc. Also, best management construction practices should be strictly adhered to at all times to minimize the potential for distress during construction and roadway improvements.

PCC Cross Gutters

PCC cross gutters should be designed in accordance with San Diego Regional Standard Drawing (SDRSD) G-12.

Additional Considerations

To mitigate perched groundwater, consideration should be given to installation of subgrade separators (cut-offs) between pavement subgrade and landscape areas, although this is not a requirement from a geotechnical standpoint. Cut-offs, if used, should be 6 inches wide and at least 12 inches below the pavement subgrade contact or 12 inches below the crushed aggregate base rock, if utilized.

EXPANSIVE SOILS, DRIVEWAY, FLATWORK, AND OTHER IMPROVEMENTS

Some of the soil materials on site are likely to be expansive. The effects of expansive soils are cumulative, and typically occur over the lifetime of any improvements. On relatively level areas, when the soils are allowed to dry, the dessication and swelling process tends to cause heaving and distress to flatwork and other improvements. The resulting potential for distress to improvements may be reduced, but not totally eliminated. To that end, it is recommended that the developer should notify all interested/affected parties of this long-term potential for distress. To reduce the likelihood of distress, the following recommendations are presented for all exterior flatwork:

1. The subgrade area for concrete slabs should be compacted to achieve a minimum 90 percent relative compaction, and then be pre-soaked to 2 to 3 percentage points above (or 125 percent of) the soils' optimum moisture content, to a depth of 18 inches below subgrade elevation. The moisture content of the subgrade should be proof tested within 72 hours prior to pouring concrete.

2. Concrete slabs should be cast over a relatively non-yielding surface, consisting of a 4-inch layer of crushed rock, gravel, or clean sand, that should be compacted and level prior to pouring concrete. The layer should wet-down completely prior to pouring concrete, to minimize loss of concrete moisture to the surrounding earth materials.

3. Exterior slabs should be a minimum of 4 inches thick. Driveway slabs and approaches should additionally have a thickened edge (12 inches) adjacent to all landscape areas, to help impede infiltration of landscape water under the slab.

4. The use of transverse and longitudinal control joints are recommended to help control slab cracking due to concrete shrinkage or expansion. Two ways to mitigate such cracking are: a) add a sufficient amount of reinforcing steel, increasing tensile strength of the slab; and, b) provide an adequate amount of control and/or expansion joints to accommodate anticipated concrete shrinkage and expansion.

   In order to reduce the potential for unsightly cracks, slabs should be reinforced at mid-height with a minimum of No. 3 bars placed at 18 inches on center, in each direction. The exterior slabs should be scored or saw cut, ½ to ⅜ inches deep, often enough so that no section is greater than 10 feet by 10 feet. For sidewalks or narrow slabs, control joints should be provided at intervals of every 6 feet. The slabs should be separated from the foundations and sidewalks with expansion joint filler material.

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

6. Driveways, sidewalks, and patio slabs adjacent to the house should be separated from the residence with thick expansion joint filler material. In areas directly adjacent to a continuous source of moisture (i.e., irrigation, planters, etc.), all joints should be additionally sealed with flexible mastic.

7. Planters and walls should not be tied to the house.

8. Overhang structures should be supported on the slabs, or structurally designed with continuous footings tied in at least two directions.

9. Any masonry landscape walls that are to be constructed throughout the property should be grouted and articulated in segments no more than 20 feet long. These segments should be keyed or doweled together.

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

11. Positive site drainage should be maintained at all times. Finish grade on the lots should provide a minimum of 1 to 2 percent fall to the street, as indicated herein. It should be kept in mind that drainage reversals could occur, including post-construction settlement, if relatively flat yard drainage gradients are not periodically maintained by the homeowner or homeowners association.

12. Due to expansive soils, air conditioning (A/C) units should be supported by slabs that are incorporated into the building foundation or constructed on a rigid slab with

flexible couplings for plumbing and electrical lines. A/C waste water lines should be drained to a suitable non-erosive outlet.

13. Shrinkage cracks could become excessive if proper finishing and curing practices are not followed. Finishing and curing practices should be performed per the Portland Cement Association Guidelines. Mix design should incorporate rate of curing for climate and time of year, sulfate content of soils, corrosion potential of soils, and fertilizers used on site.

ONSITE INFILTRATION-RUNOFF RETENTION SYSTEMS

General

Onsite infiltration-runoff retention systems (OIRRS) are anticipated to be used for Best Management Practices (BMP's) or Low Impact Development (LID) principles for the project. To that end, some guidelines should/must be followed in the planning, design, and construction of such systems. Such facilities, if improperly designed or implemented without consideration of the geotechnical aspects of site conditions, can contribute to flooding, saturation of bearing materials beneath site improvements, slope instability, and possible concentration and contribution of pollutants into the groundwater or storm drain and/or utility trench systems.

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

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

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

- Based on our review of the United States Department of Agriculture-Natural Resource Conservation Services's soil survey website (http://websoilsurvey.sc.egov.usda.gov/App/WebSoilSurvey.aspx), the onsite soils consist of the Escondido very fine sandy loam, 5 to 9 percent slopes; the Huerhuero loam, 2 to 9 percent slopes; the Wyman loam, 5 to 9 percent slopes; and the Escondido very fine sandy loam, 15 to 30 percent slopes, eroded. The capacity of the most limiting layer to transmit water (Ksat) for the Escondido very fine sandy loam, 5 to 9 percent and 15 to 30 percent slopes is moderately high to high (0.57 to 1.98 inches per hour [in/hr]. The Ksat for the Huerhuero loam, 2 to 9 percent slopes is characterized as very low to moderately low (0.00 to 0.06 inches per hour [in/hr]). The Ksat for the Wyman loam, 5 to 9 percent slopes is characterized as moderately high (0.20 to 0.57 in/hr). With the exception of the Huerhuero loam, 2 to 9 percent slopes, these mapped soil units fall into Hydrologic Soil Group (HSG) "C." The Huerhuero loam, 2 to 9 percent slopes falls into HSG "D." County of San Diego (2007) indicates that infiltration in HSG "C" and "D" is severely limited.

- It is not good engineering practice to allow water to saturate soils, especially near slopes or improvements; however, the controlling agency/authority is now requiring this for OIRRS purposes on many projects.

- If infiltration is planned, infiltration system design should be based on actual infiltration testing results/data, preferably utilizing double-ring infiltrometer testing (ASTM D 3385) to determine the infiltration rate of the earth materials being contemplated for infiltration.

- Wherever possible, infiltration systems should not be installed within ±50 feet of the tops of slopes steeper than 15 percent or within H/3 from the tops of slopes (where H equals the height of slope).

- Wherever possible, infiltrations systems should not be placed within a distance of H/2 from the toes of slopes (where H equals the height of slope).

- Impermeable liners and subdrains should be used along the bottom of bioretention swales/basins located within the influence of slopes.

- Impermeable liners and subdrains should be used along the bottom of bioretention swales/basins located within the influence of slopes. Impermeable liners used in conjunction with bioretention basins should consist of a 30-mil polyvinyl chloride (PVC) membrane that is covered by a minimum of 12-inches of clean soil, free from rocks and debris, with a maximum 4:1 (h:v) slope inclination, or flatter, and meets the following minimum specifications:

Specific Gravity (ASTM D792): 1.2 (g/cc, min.); Tensile (ASTM D882): 73 (lb/in-width, min); Elongation at Break (ASTM D882): 380 (%, min); Modulus (ASTM D882): 30 (lb/in-width, min.); and Tear Strength (ASTM D1004): 8 (lb/in, min); Seam Shear Strength (ASTM D882) 58.4 (lb/in, min); Seam Peel Strength (ASTM D882) 15 (lb/in, min).

- Subdrains should consist of at least 4-inch diameter Schedule 40 or SDR 35 drain pipe with perforations oriented down. The drain pipe should be sleeved with a filter sock.

- The landscape architect should be notified of the location of the proposed OIRRS. If landscaping is proposed within the OIRRS, consideration should be given to the type of vegetation chosen and their potential effect upon subsurface improvements (i.e., some trees/shrubs will have an effect on subsurface improvements with their extensive root systems). Over-watering landscape areas above, or adjacent to, the proposed OIRRS could adversely affect performance of the system.

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

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

- If subsurface infiltration galleries/chambers are proposed, the appropriate size, depth interval, and ultimate placement of the detention/infiltration system should be evaluated by the design engineer, and be of sufficient width/depth to achieve optimum performance, based on the infiltration rates provided. In addition, proper debris filter systems will need to be utilized for the infiltration galleries/chambers. Debris filter systems will need to be self cleaning and periodically and regularly maintained on a regular basis. Provisions for the regular and periodic maintenance of any debris filter system is recommended and this condition should be disclosed to all interested/affected parties.

- Infiltrations systems should not be installed within ±8 feet of building foundations utility trenches, and walls, or a 1:1 (h:v) slope (down and away) from the bottom elements of these improvements. Alternatively, deepened foundations and/or pile/pier supported improvements may be used.

- Infiltrations systems should not be installed adjacent to pavement and/or hardscape improvements. Alternatively, deepened/thickened edges and curbs and/or impermeable liners may be utilized in areas adjoining the OIRRS.

- As with any OIRRS, localized ponding and groundwater seepage should be anticipated. The potential for seepage and/or perched groundwater to occur after site development should be disclosed to all interested/affected parties.

- Installation of infiltrations systems should avoid expansive soils (E.I. $\geq$51) or soils with a relatively high plasticity index (P.I. > 20).

- Infiltration systems should not be installed where the vertical separation of the groundwater level is less than $\pm$10 feet from the base of the system.

- Where permeable pavements are planned as part of the system, the site Traffic Index (T.I.) should be less than 25,000 Average Daily Traffic (ADT), as recommended in Allen, et al. (2011).

- Infiltration systems should be designed using a suitable factor of safety (FOS) to account for uncertainties in the known infiltration rates (as generally required by the controlling authorities), and reduction in performance over time.

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

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

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

- Newly established vegetation/landscaping (including phreatophytes) may have root systems that will influence the performance of the OIRRS or nearby LID systems.

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

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

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

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

DEVELOPMENT CRITERIA

Slope Deformation

Compacted fill slopes designed using customary factors of safety for gross or surficial stability and constructed in general accordance with the design specifications should be expected to undergo some differential vertical heave or settlement in combination with differential lateral movement in the out-of-slope direction, after grading. This post-construction movement occurs in two forms: slope creep, and lateral fill extension (LFE). Slope creep is caused by alternate wetting and drying of the fill soils which results in slow downslope movement. This type of movement is expected to occur throughout the life of the slope, and is anticipated to potentially affect improvements or structures (e.g., separations and/or cracking), placed near the top-of-slope, up to a maximum distance of approximately 15 feet from the top-of-slope, depending on the slope height. This movement generally results in rotation and differential settlement of improvements located within the creep zone. LFE occurs due to deep wetting from irrigation and rainfall on slopes comprised of expansive materials. Although some movement should be expected, long-term movement from this source may be minimized, but not eliminated, by placing the fill throughout the slope region, wet of the fill's optimum moisture content.

It is generally not practical to attempt to eliminate the effects of either slope creep or LFE. Suitable mitigative measures to reduce the potential of lateral deformation typically include: setback of improvements from the slope faces (per the 2013 CBC), positive structural separations (i.e., joints) between improvements, and stiffening and deepening of foundations. Expansion joints in walls should be placed no greater than 20 feet on-center, and in accordance with the structural engineer's recommendations. All of these measures are recommended for design of structures and improvements. The ramifications of the above conditions, and recommendations for mitigation, should be provided to all interested/affected parties.

Slope Maintenance and Planting

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

Drainage

Adequate surface drainage is a very important factor in reducing the likelihood of adverse performance of foundations, hardscape, and slopes. Surface drainage should be sufficient to mitigate ponding of water anywhere on the property, and especially near structures and tops of slopes. Surface drainage should be carefully taken into consideration during fine grading, landscaping, and building construction. Therefore, care should be taken that future landscaping or construction activities do not create adverse drainage conditions. Positive site drainage within the property should be provided and maintained at all times. Drainage should not flow uncontrolled down any descending slope. Water should be directed away from foundations and tops of slopes, and not allowed to pond and/or seep into the ground. In general, site drainage should conform to Section 1804.3 of the 2013 CBC. Consideration should be given to avoiding construction of planters adjacent to structures (buildings, pools, spas, etc.). Building pad drainage should be directed toward the street or other approved area(s). Although not a geotechnical requirement, roof gutters, down spouts, or other appropriate means may be utilized to control roof drainage. Down spouts, or drainage devices should outlet a minimum of 5 feet from structures or into a subsurface drainage system. Areas of seepage may develop due to irrigation or heavy rainfall, and should be anticipated. Minimizing irrigation will lessen this potential. If areas of seepage develop, recommendations for minimizing this effect could be provided upon request.

Erosion Control

Cut and fill slopes will be subject to surficial erosion during and after grading. Onsite earth materials have a moderate to high erosion potential. Consideration should be given to providing hay bales and silt fences for the temporary control of surface water, from a geotechnical viewpoint.

Landscape Maintenance

Only the amount of irrigation necessary to sustain plant life should be provided. Over-watering the landscape areas will adversely affect proposed site improvements. We would recommend that any proposed open-bottom planters adjacent to proposed structures be eliminated for a minimum distance of 10 feet. As an alternative, closed-bottom type planters could be utilized. An outlet placed in the bottom of the planter, could be installed to direct drainage away from structures or any exterior concrete flatwork. If planters are constructed adjacent to structures, the sides and bottom of the planter should be provided with a moisture barrier to prevent penetration of irrigation water into the subgrade. Provisions should be made to drain the excess irrigation water from the planters without saturating the subgrade below or adjacent to the planters. Graded slope areas should be planted with drought resistant vegetation. Consideration should be given to the type of vegetation chosen and their potential effect upon surface improvements (i.e., some trees will have an effect on concrete flatwork with their extensive root systems). From a geotechnical standpoint leaching is not recommended for establishing landscaping. If the surface soils are processed for the purpose of adding amendments, they should be recompacted to 90 percent minimum relative compaction.

Gutters and Downspouts

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

Subsurface and Surface Water

Subsurface and surface water are not anticipated to affect site development, provided that the recommendations contained in this report are incorporated into final design and construction and that prudent surface and subsurface drainage practices are incorporated into the construction plans. Perched groundwater conditions along zones of contrasting permeabilities may not be precluded from occurring in the future due to site irrigation, poor drainage conditions, or damaged utilities, and should be anticipated. Should perched groundwater conditions develop, this office could assess the affected area(s) and provide the appropriate recommendations to mitigate the observed groundwater conditions. Groundwater conditions may change with the introduction of irrigation, rainfall, or other factors.

Site Improvements

If in the future, any additional improvements (e.g., pools, spas, etc.) are planned for the site, recommendations concerning the geological or geotechnical aspects of design and construction of said improvements could be provided upon request. Pools and/or spas should not be constructed without specific design and construction recommendations from GSI, and this construction recommendation should be provided to all interested/affected parties. This office should be notified in advance of any fill placement, grading of the site, or trench backfilling after rough grading has been completed. This includes any grading, utility trench and retaining wall backfills, flatwork, etc.

Tile Flooring

Tile flooring can crack, reflecting cracks in the concrete slab below the tile, although small cracks in a conventional slab may not be significant. Therefore, the designer should consider additional steel reinforcement for concrete slabs-on-grade where tile will be placed. The tile installer should consider installation methods that reduce possible cracking of the tile such as slipsheets. Slipsheets or a vinyl crack isolation membrane (approved by the Tile Council of America/Ceramic Tile Institute) are recommended between tile and concrete slabs on grade.

Additional Grading

This office should be notified in advance of any fill placement, supplemental regrading of the site, or trench backfilling after rough grading has been completed. This includes completion of grading in the street, driveway approaches, driveways, parking areas, and utility trench and retaining wall backfills.

Footing Trench Excavation

All footing excavations should be observed by a representative of this firm subsequent to trenching and prior to concrete form and reinforcement placement. The purpose of the observations is to evaluate that the excavations have been made into the recommended bearing material and to the minimum widths and depths recommended for construction. If loose or compressible materials are exposed within the footing excavation, a deeper footing or removal and recompaction of the subgrade materials would be recommended at that time. Footing trench spoil and any excess soils generated from utility trench excavations should be compacted to a minimum relative compaction of 90 percent, if not removed from the site.

Trenching/Temporary Construction Backcuts

Considering the nature of the onsite earth materials, it should be anticipated that caving or sloughing could be a factor in subsurface excavations and trenching. Shoring or excavating the trench walls/backcuts at the angle of repose (typically 25 to 45 degrees [except as specifically superceded within the text of this report]), should be anticipated. All excavations should be observed by an engineering geologist or soil engineer from GSI, prior to workers entering the excavation or trench, and minimally conform to CAL-OSHA, state, and local safety codes. Should adverse conditions exist, appropriate recommendations would be offered at that time. The above recommendations should be provided to any contractors and/or subcontractors, or homeowners, etc., that may perform such work.

Utility Trench Backfill

1. All interior utility trench backfill should be brought to at least 2 percent above optimum moisture content and then compacted to obtain a minimum relative compaction of 90 percent of the laboratory standard. As an alternative for shallow (12-inch to 18-inch) under-slab trenches, sand having a sand equivalent value of 30 or greater may be utilized and jetted or flooded into place. Observation, probing and testing should be provided to evaluate the desired results.

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

3. All trench excavations should conform to CAL-OSHA, state, and local safety codes.

4. Utilities crossing grade beams, perimeter beams, or footings should either pass below the footing or grade beam utilizing a hardened collar or foam spacer, or pass through the footing or grade beam in accordance with the recommendations of the structural engineer.

SUMMARY OF RECOMMENDATIONS REGARDING GEOTECHNICAL OBSERVATION AND TESTING

We recommend that observation and/or testing be performed by GSI at each of the following construction stages:

- During grading/recertification.

- During excavation.

- During placement of subdrains or other subdrainage devices, prior to placing fill and/or backfill.

- After excavation of building footings, retaining wall footings, and free standing walls footings, prior to the placement of reinforcing steel or concrete.

- Prior to pouring any slabs or flatwork, after presoaking/presaturation of building pads and other flatwork subgrade, before the placement of concrete, reinforcing steel, capillary break (i.e., sand, pea-gravel, etc.), or vapor retarders (i.e., visqueen, etc.).

- During retaining wall subdrain installation, prior to backfill placement.

- During placement of backfill for area drain, interior plumbing, utility line trenches, and retaining wall backfill.

- During slope construction/repair.

- When any unusual soil conditions are encountered during any construction operations, subsequent to the issuance of this report.

- When any developer or homeowner improvements, such as flatwork, spas, pools, walls, etc., are constructed, prior to construction.

- A report of geotechnical observation and testing should be provided at the conclusion of each of the above stages, in order to provide concise and clear documentation of site work, and/or to comply with code requirements.

OTHER DESIGN PROFESSIONALS/CONSULTANTS

The design civil engineer, structural engineer, post-tension designer, architect, landscape architect, wall designer, etc., should review the recommendations provided herein, incorporate those recommendations into all their respective plans, and by explicit reference, make this report part of their project plans. This report presents minimum design criteria for the design of slabs, foundations and other elements possibly applicable to the project. These criteria should not be considered as substitutes for actual designs by the structural engineer/designer. Please note that the recommendations contained herein are not intended to preclude the transmission of water or vapor through the slab or foundation. The structural engineer/foundation and/or slab designer should provide recommendations to not allow water or vapor to enter into the structure so as to cause damage to another building component, or so as to limit the installation of the type of flooring materials typically used for the particular application.

The structural engineer/designer should analyze actual soil-structure interaction and consider, as needed, bearing, expansive soil influence, and strength, stiffness and deflections in the various slab, foundation, and other elements in order to develop appropriate, design-specific details. As conditions dictate, it is possible that other influences will also have to be considered. The structural engineer/designer should consider all applicable codes and authoritative sources where needed. If analyses by the structural engineer/designer result in less critical details than are provided herein as minimums, the minimums presented herein should be adopted. It is considered likely that some, more restrictive details will be required.

If the structural engineer/designer has any questions or requires further assistance, they should not hesitate to call or otherwise transmit their requests to GSI. In order to mitigate potential distress, the foundation and/or improvement's designer should confirm to GSI and the governing agency, in writing, that the proposed foundations and/or improvements can tolerate the amount of differential settlement and/or expansion characteristics and other design criteria specified herein.

PLAN REVIEW

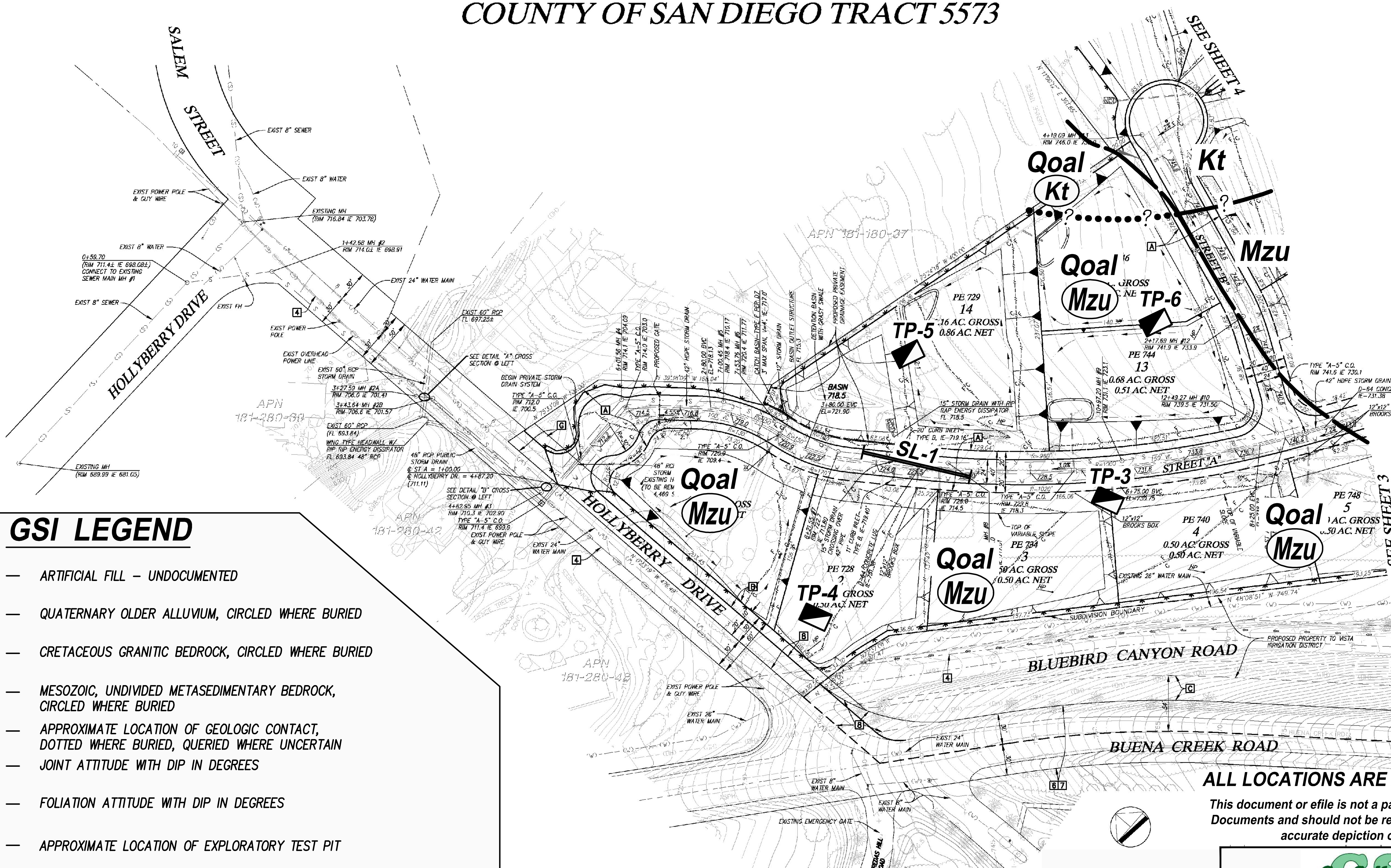
Final project plans (grading, precise grading, foundation, retaining wall, landscaping, etc.), should be reviewed by this office prior to construction, so that construction is in accordance with the conclusions and recommendations of this report. Based on our review, supplemental recommendations and/or further geotechnical studies may be warranted.

LIMITATIONS

The materials encountered on the project site and utilized for our analysis are believed representative of the area; however, soil and bedrock materials vary in character between excavations and natural outcrops or conditions exposed during mass grading. Site conditions may vary due to seasonal changes or other factors.

Inasmuch as our study is based upon our review and engineering analyses and laboratory data, the conclusions and recommendations are professional opinions. These opinions have been derived in accordance with current standards of practice, and no warranty, either express or implied, is given. Standards of practice are subject to change with time. GSI assumes no responsibility or liability for work or testing performed by others, or their inaction; or work performed when GSI is not requested to be onsite, to evaluate if our recommendations have been properly implemented. Use of this report constitutes an agreement and consent by the user to all the limitations outlined above, notwithstanding any other agreements that may be in place. In addition, this report may be subject to review by the controlling authorities. Thus, this report brings to completion our scope of services for this portion of the project. All samples will be disposed of after 30 days, unless specifically requested by the client, in writing.

TENTATIVE SUBDIVISION MAP
COUNTY OF SAN DIEGO TRACT 5573



GSI LEGEND

afu	—	ARTIFICIAL FILL – UNDOCUMENTED
Qoal	—	QUATERNARY OLDER ALLUVIUM, CIRCLED WHERE BURIED
Kt	—	CRETACEOUS GRANITIC BEDROCK, CIRCLED WHERE BURIED
Mzu	—	MESOZOIC, UNDIVIDED METASEDIMENTARY BEDROCK, CIRCLED WHERE BURIED
?	—	APPROXIMATE LOCATION OF GEOLOGIC CONTACT, DOTTED WHERE BURIED, QUERIED WHERE UNCERTAIN
80°	—	JOINT ATTITUDE WITH DIP IN DEGREES
50°	—	FOLIATION ATTITUDE WITH DIP IN DEGREES
TP-10	—	APPROXIMATE LOCATION OF EXPLORATORY TEST PIT
SL-3	—	APPROXIMATE LOCATION OF SEISMIC REFRACTION LINE

BASE MAP PROVIDED BY:

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GRAPHIC SCALE

100 0 50 100 200

1" = 100'

ALL LOCATIONS ARE APPROXIMATE

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

GeoSoils, Inc.

GEOTECHNICAL MAP

Plate 1

W.O. 6814-A-SC | DATE: 02/15 | SCALE: 1" = 100'

TENTATIVE SUBDIVISION MAP
COUNTY OF SAN DIEGO TRACT 5573

EASEMENT NOTES:

[A] PROPOSED PRIVATE ROAD, PUBLIC UTILITY, EMERGENCY ACCESS, & BUENA SANITATION DISTRICT PUBLIC SEWER & ACCESS EASEMENT.

[B] PROPOSED PUBLIC RIGHT-OF-WAY DEDICATION TO THE COUNTY OF SAN DIEGO.

[C] PROPOSED IRREVOCABLE OFFER OF DEDICATION TO THE COUNTY OF SAN DIEGO

[1] EXISTING EASEMENT TO PACIFIC TELEPHONE & TELEGRAPH CO., PER BOOK 511, PAGE 234, RECORDED: 3-4-1911, OF DEEDS (NON-PLOTTABLE)

[2] EXISTING EASEMENT TO VISTA IRRIGATION DISTRICT, PER BOOK 1221, PAGE 189, RECORDED: 6-17-1926, OF DEEDS (NON-PLOTTABLE)

[3] EXISTING EASEMENT TO VISTA IRRIGATION DISTRICT, PER BOOK 1244, PAGE 335, RECORDED: 9-20-1926, OF DEEDS (NON-PLOTTABLE)

[4] EXISTING EASEMENT TO SAN DIEGO GAS & ELECTRIC CO., PER BOOK 693, PAGE 482, RECORDED: 10-22-1937, OF OFFICIAL RECORDS.

[5] EXISTING 40' RIGHT-OF-WAY (BLUEBIRD CANYON ROAD 05134) TO THE COUNTY OF SAN DIEGO, PER BOOK 1266, PAGE 372, RECORDED: 10-20-1926, OF DEEDS.

[6] EXISTING 60' EASEMENT FOR COUNTY HIGHWAY (RS681) TO THE COUNTY OF SAN DIEGO, PER BOOK 955, PAGE 249, RECORDED: 10-31-1939, OF OFFICIAL RECORDS.

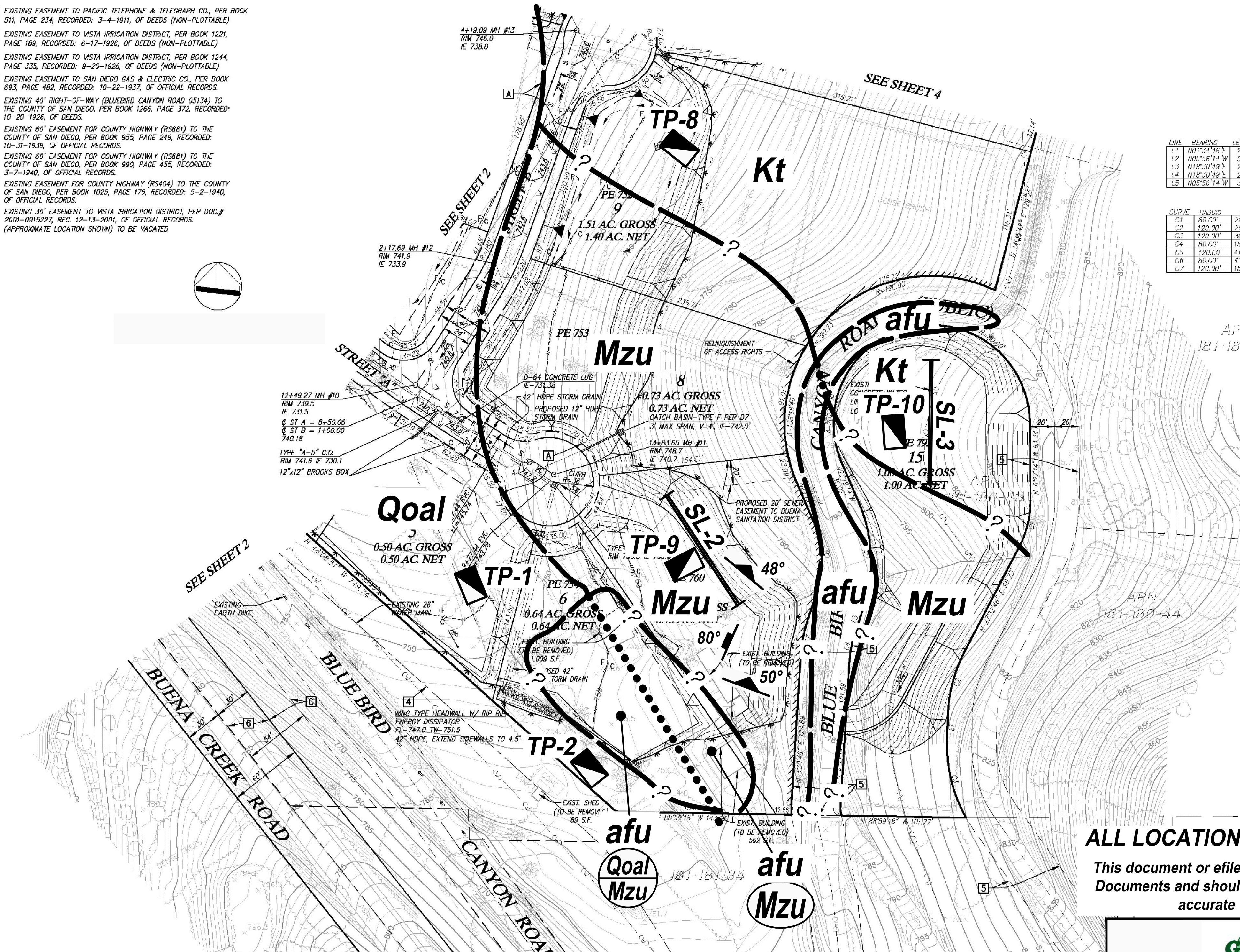
[7] EXISTING 60' EASEMENT FOR COUNTY HIGHWAY (RS681) TO THE COUNTY OF SAN DIEGO, PER BOOK 990, PAGE 455, RECORDED: 3-7-1940, OF OFFICIAL RECORDS.

[8] EXISTING EASEMENT FOR COUNTY HIGHWAY (RS404) TO THE COUNTY OF SAN DIEGO, PER BOOK 1025, PAGE 176, RECORDED: 5-2-1940, OF OFFICIAL RECORDS.

[9] EXISTING 30' EASEMENT TO VISTA IRRIGATION DISTRICT, PER DOC.# 2001-0915227, REC. 12-13-2001, OF OFFICIAL RECORDS. (APPROXIMATE LOCATION SHOWN) TO BE VACATED

NOTES:

1) SEWER STATIONS ARE INDEPENDENT OF STREET STATIONING
2) SEE SHEET 3 FOR EASEMENT INFORMATION



LINE	BEARING	LENGTH
L1	N01°34'46"E	20.59'
L2	N05°56'14"W	5.28'
L3	N18°30'49"E	22.97'
L4	N18°30'49"E	22.97'
L5	N05°56'14"W	3.72'

CURVE	RADUIS	DELTA	LENGTH
C1	80.00'	28°00'00"	39.10'
C2	120.00'	25°58'00"	54.38'
C3	120.00'	30°57'29"	64.84'
C4	80.00'	15°00'00"	20.94'
C5	120.00'	41°50'00"	87.62'
C6	60.00'	41°50'00"	58.41'
C7	120.00'	15°00'00"	31.42'

ALL LOCATIONS ARE APPROXIMATE

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SEE PLATE 1 FOR LEGEND

GRAPHIC SCALE
100 0 50 100 200
1" = 100'

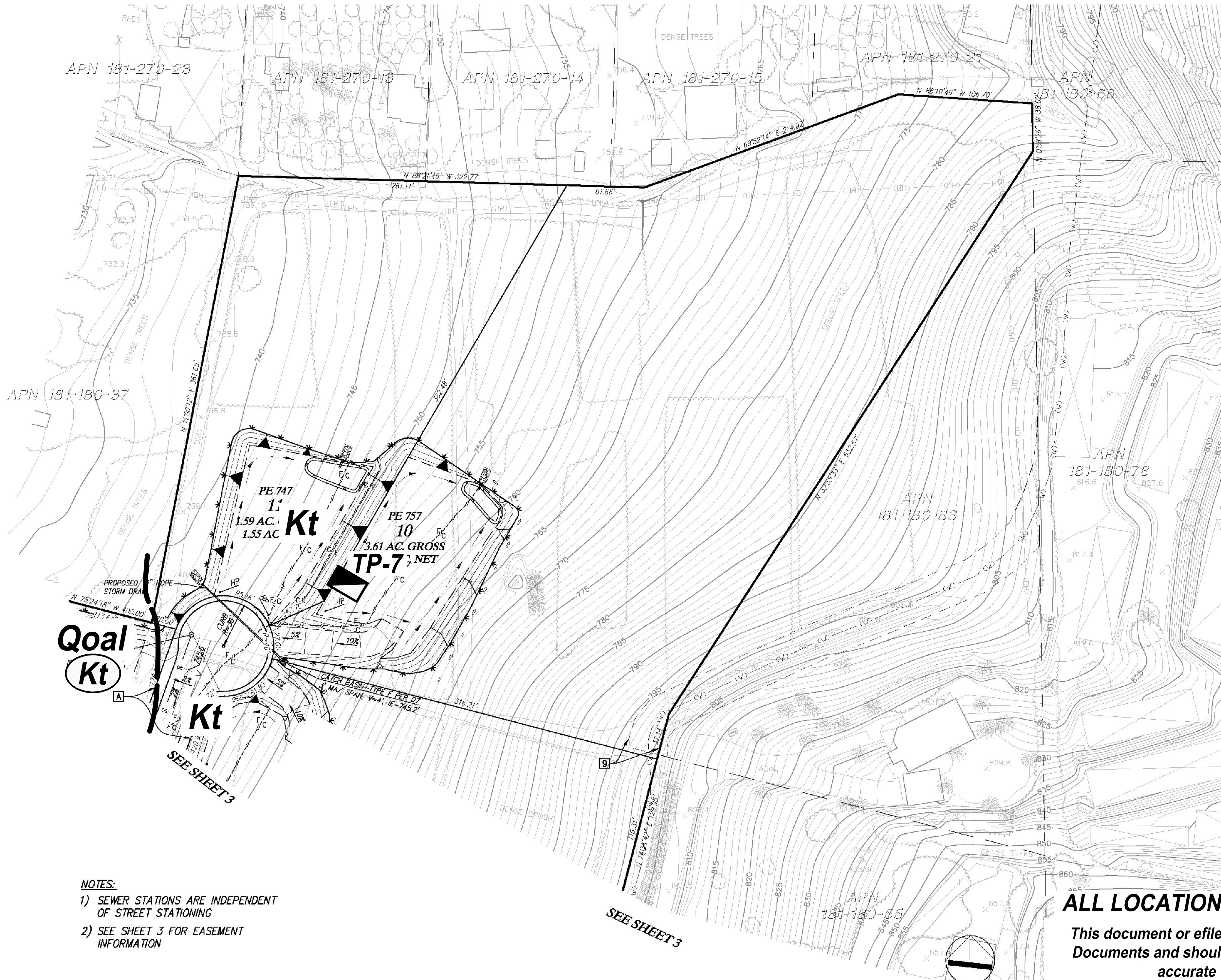
GeoSoils, Inc.

GEOTECHNICAL MAP

Plate 2

W.O. 6814-A-SC | DATE: 03/15 | SCALE: 1" = 100'

TENTATIVE SUBDIVISION MAP
COUNTY OF SAN DIEGO TRACT 5573



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SEE PLATE 1 FOR LEGEND

GRAPHIC SCALE
100 0 50 100 200
1" = 100'

GeoSoils, Inc.

GEOTECHNICAL MAP

Plate 3

W.O. 6814-A-SC	DATE: 02/15	SCALE: 1" = 100'

APPENDIX A

REFERENCES

APPENDIX A

REFERENCES

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APPENDIX B

TEST EXCAVATION LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM				
Major Divisions			**Group Symbols**	**Typical Names**
Coarse-Grained Soils More than 50% retained on No. 200 sieve	Gravels 50% or more of coarse fraction retained on No. 4 sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravel with	GM	Silty gravels gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands more than 50% of coarse fraction passes No. 4 sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils 50% or more passes No. 200 sieve	Silts and Clays Liquid limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, mucic, and other highly organic soils

CONSISTENCY OR RELATIVE DENSITY

CRITERIA

Standard Penetration Test

Penetration Resistance N (blows/ft)	Relative Density
0 - 4	Very loose
4 - 10	Loose
10 - 30	Medium
30 - 50	Dense
> 50	Very dense

Standard Penetration Test

Penetration Resistance N (blows/ft)	Consistency	Unconfined Compressive Strength (tons/ft^2)
<2	Very Soft	<0.25
2 - 4	Soft	0.25 - .050
4 - 8	Medium	0.50 - 1.00
8 - 15	Stiff	1.00 - 2.00
15 - 30	Very Stiff	2.00 - 4.00
>30	Hard	>4.00

3" 3/4" #4 #10 #40 #200 U.S. Standard Sieve

Unified Soil Classification	Cobbles	Gravel		Sand			Silt or Clay
		coarse	fine	coarse	medium	fine	

MOISTURE CONDITIONS

Dry	Absence of moisture: dusty, dry to the touch
Slightly Moist	Below optimum moisture content for compaction
Moist	Near optimum moisture content
Very Moist	Above optimum moisture content
Wet	Visible free water; below water table

MATERIAL QUANTITY

trace	0 - 5 %
few	5 - 10 %
little	10 - 25 %
some	25 - 45 %

OTHER SYMBOLS

C	Core Sample
S	SPT Sample
B	Bulk Sample
–	Groundwater
Qp	Pocket Penetrometer

BASIC LOG FORMAT:
Group name, Group symbol, (grain size), color, moisture, consistency or relative density. Additional comments: odor, presence of roots, mica, gypsum, coarse grained particles, etc.

EXAMPLE:
Sand (SP), fine to medium grained, brown, moist, loose, trace silt, little fine gravel, few cobbles up to 4" in size, some hair roots and rootlets.

GeoSoils, Inc.

W.O. 6814-A-SC
Tomlinson
Tract 5573, SD Co.
Logged By: RGC
November 26, 2014

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-1	747	0-1	SC				**UNDOCUMENTED ARTIFICIAL FILL:** CLAYEY SAND with GRAVEL, gray brown, dry, loose.
		1-3	SC				**QUATERNARY OLDER ALLUVIUM:** CLAYEY SAND, dark brown to gray brown, dry, loose; very porous.
		3-8	SC				CLAYEY SAND, dark brown, damp to moist, medium dense; no visible porosity, difficult excavation.
							Total Depth =8' No Groundwater Encountered Backfilled 11-26-2014
TP-2	755	0-1	SM/SC				**UNDOCUMENTED ARTIFICIAL FILL:** SILTY to CLAYEY SAND, brown, dry, loose; many roots, some gravel, old pipe @ -1'.
		1-3½	SC				**QUATERNARY COLLUVIUM:** CLAYEY SAND, dark brown, dry, loose; porous.
		3-4	SM				**QUATERNARY OLDER ALLUVIUM:** SILTY SAND with CLAY, dark yellowish brown to dark brown, dry to damp, medium dense; no visible porosity.
		4-6	SC				CLAYEY SAND and GRAVEL, dark brown, dry, medium dense; porous, sub angular, gravel size fragments of metasedimentary rock, cemented, difficult excavation.
		6-7	SM				SILTY SAND with CLAY, dark brown, damp, medium dense; cemented, no visible porous, difficult excavation.
							Total Depth =7' No Groundwater Encountered Backfilled 11-26-2014

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Tomlinson
Tract 5573, SD Co.
Logged By: RGC
November 26, 2014

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-3	735	0-½	SC				**FILL:** CLAYEY SAND with GRAVEL, brown to dark brown, dry, loose; many rootlet porous.
		½-3	SC	Bulk @ ½-3			**QUATERNARY COLLUVIUM:** CLAYEY SAND, dark yellowish brown, dry, loose; very porous, tight, random fractures.
		3-5	SC	Ring @ 3	10.7	112.1	**OLDER COLLUVIUM:** CLAYEY SAND, dark yellowish brown to dark brown, damp to moist, medium dense; no visible porosity.
		5-6	SM	Bulk @ 5-6			SILTY SAND with CLAY, mottled brown, dark yellowish brown and gray, moist, medium dense; few carbonate filaments.
		6-10	SM				SILTY SAND, brown, moist, medium dense to dense; fine grained.
		10-14	SM	Bulk @12 Bulk @14			SILTY SAND with some GRAVEL, olive brown, moist, dense; gravels are sub angular closets of metasedimentary rock.
							Total Depth =14' No Groundwater Encountered Backfilled 11-26-2014
TP-4	727	0-2	SC				**QUATERNARY COLLUVIUM:** CLAYEY SAND, brown, dry, loose; porous, dessicated.
		2-3½	SC				**QUATERNARY OLDER ALLUVIUM:** CLAYEY SAND, dark brown to dark yellowish brown, dry, medium dense; porous.
		3½-7	SM	Sm. Chunk @ 7	7.9	112.4	SILTY SAND with CLAY, dark yellowish brown, damp, medium dense; cemented no visible porosity, difficult excavation.
							Total Depth =7' No Groundwater Encountered Backfilled 11-26-2014

GeoSoils, Inc.

W.O. 6814-A-SC
Tomlinson
Tract 5573, SD Co.
Logged By: RGC
November 26, 2014

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-5	727	0-2½	SC/CL				**QUATERNARY COLLUVIUM:** CLAYEY SAND to SANDY CLAY, dark brown, dry, loose; porous, desiccated.
		2½-4	SC				**QUATERNARY OLDER ALLUVIUM:** CLAYEY SAND, dark brown, damp.
		4-6½	SM	Ring @ 6	14.3	114.4	CLAYEY SAND, dark brown yellowish, damp to moist, medium dense.
							Total Depth =6½' No Groundwater Encountered Backfilled 11-26-2014
TP-6	741	1-2	SC/CL				**QUATERNARY COLLUVIUM:** CLAYEY SAND to SANDY CLAY, dark brown, dry, loose; porous, desiccated.
		2-4	SC				**QUATERNARY OLDER ALLUVIUM:** CLAYEY SAND, dark brown, damp.
		4-9½	SM				SILTY SAND with CLAY, dark brown yellowish, damp to moist, medium dense.
							Total Depth =9½' No Groundwater Encountered Backfilled 11-26-2014

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W.O. 6814-A-SC
Tomlinson
Tract 5573, SD Co.
Logged By: RGC
November 26, 2014

LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-7	753	0-2	SC				**QUATERNARY COLLUVIUM:** CLAYEY SAND, dark brown, dry, loose; porous, dessicated, few roots in upper 12", bioturbated.
		2-4	CL	Bulk @ 2-4			SANDY CLAY, dark brown, dry, stiff; porous, randomly fractured, caliche mottling.
		4-5	SC				**HIGHLY WEATHERED BEDROCK:** HIGHLY WEATHERED BEDROCK breaking to CLAYEY SAND upon excavation, olive brown gray, moist, dense; caliche mottling.
		5-6	SM/SP	Sm. Bag @ 6			**GRANITIC BEDROCK:** GRANITE ROCK breaking to SILTY SAND and SAND upon excavation. gray brown, dry, dense.
							Total Depth = 6' No Groundwater Encountered Backfilled 11-26-2014
TP-8	757	0-2	SC				**QUATERNARY COLLUVIUM:** CLAYEY SAND, dark brown, dry loose; porous, dessicated, bioturbated.
		2-4	CL				SANDY CLAY, dark brown, damp, stiff; porous, randomly fractured.
		4-4½	SC				**HIGHLY WEATHERED BEDROCK:** CLAYEY SAND, brown to olive brown, damp, medium dense.
		4½-7	SM				**CRETACEOUS GRANITIC BEDROCK:** GRANITIC ROCK breaking to SILTY SAND upon excavation, gray brown, moist, medium dense; massive.
							Total Depth = 7' No Groundwater Encountered Backfilled 11-26-2014

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Tomlinson
Tract 5573, SD Co.
Logged By: RGC
November 26, 2014

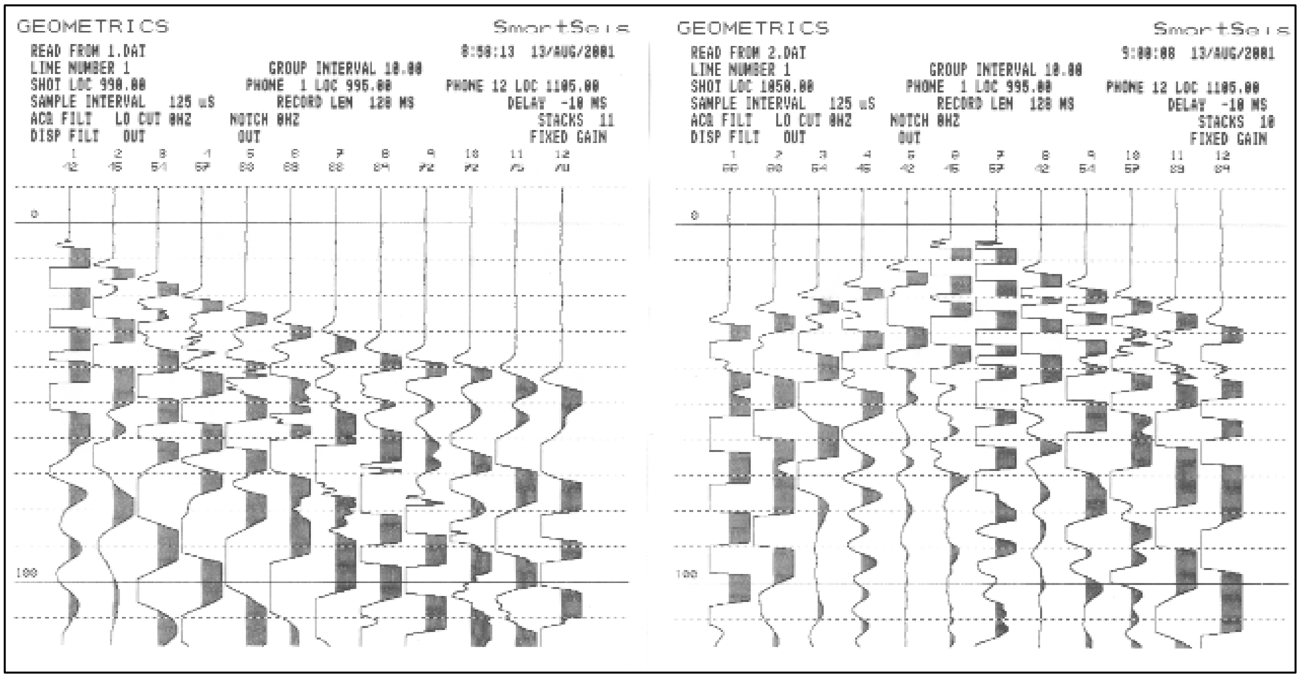
LOG OF EXPLORATORY TEST PITS

TEST PIT NO.	ELEV. (ft.)	DEPTH (ft.)	GROUP SYMBOL	SAMPLE DEPTH (ft.)	MOISTURE (%)	FIELD DRY DENSITY (pcf)	DESCRIPTION
TP-9	768	0-2½	SC				**QUATERNARY COLLUVIUM:** CLAYEY SAND, strong brown, dry, loose; few roots, bioturbated, desiccated.
		2½-5	SM				**MESOZOIC METAMORPHIC BEDROCK:** METASEDIMENTARY ROCK breaking to dark gray, SILTY SAND and platy rock fragments, planar schistosity, orienting N50˚W, 48˚NE, difficult excavation.
							Total Depth = 5' No Groundwater Encountered Backfilled 11-26-2014
TP-10	802	0-1½	SM				**QUATERNARY COLLUVIUM:** SILTY SAND, brown, dry, loose; few roots.
		1½-4	SM	Bulk @ 3			**CRETACEOUS GRANITIC BEDROCK:** GRANITIC ROCK breaking to SILTY SAND upon excavation, brown, dry, dense.
							Total Depth = 4' (Practical Refusal) No Groundwater Encountered Backfilled 11-26-2014

APPENDIX C

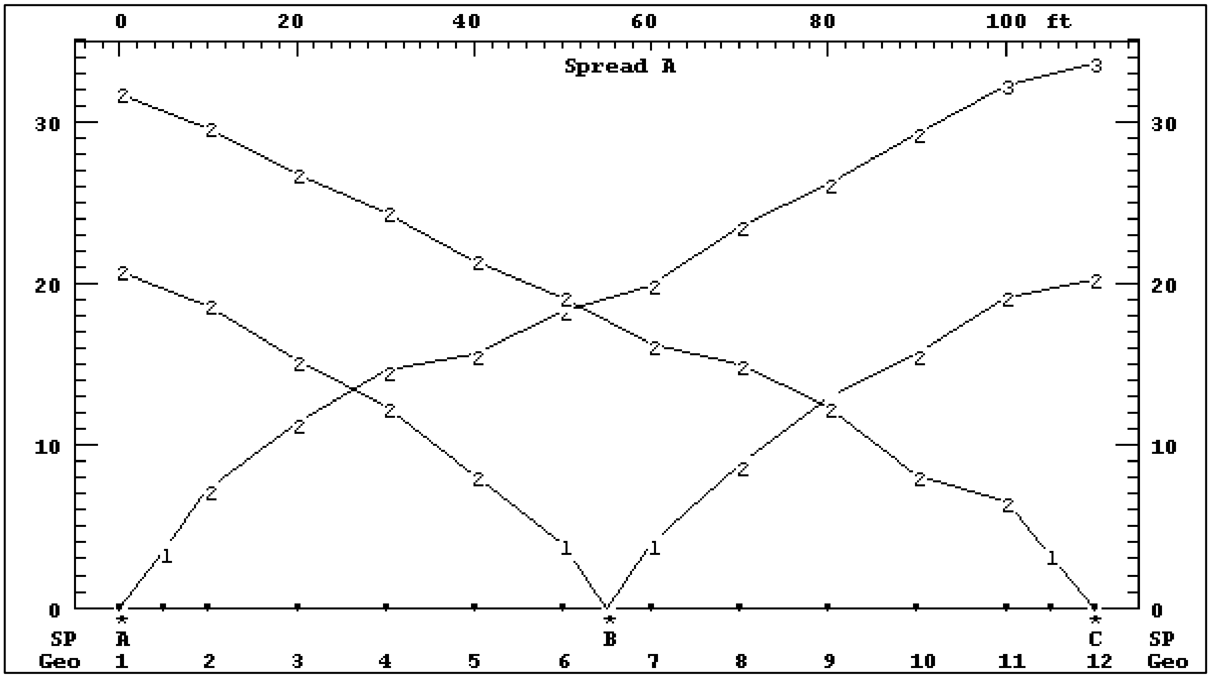
SEISMIC REFRACTION

Example
Raw Seismic Data

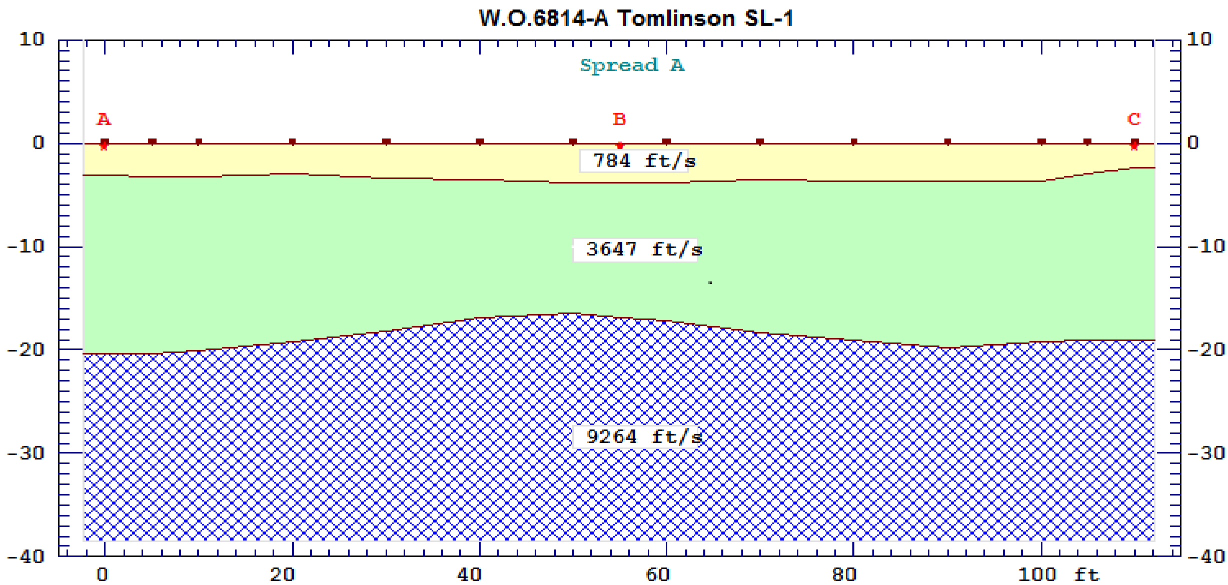


Example

Seismic Line

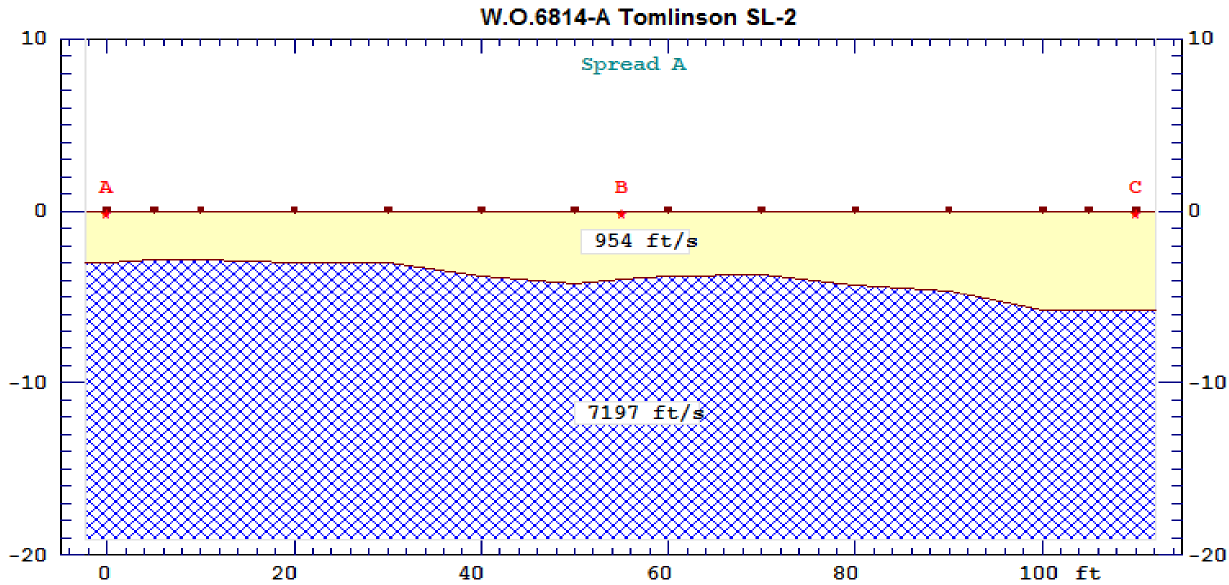


Seismic Line
SL-1



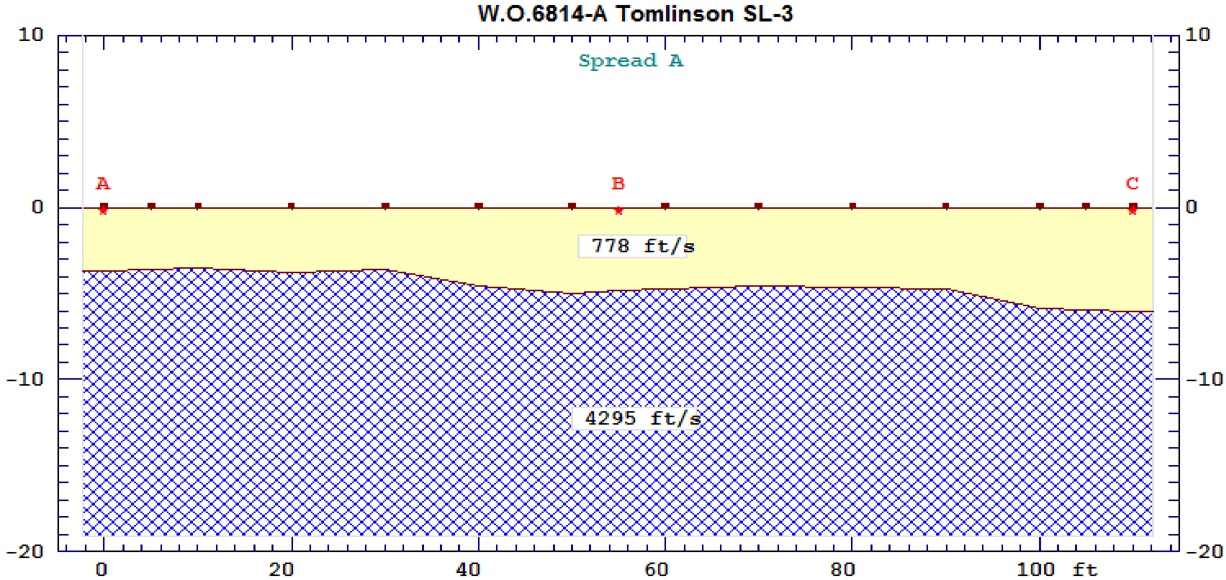
South North

Seismic Line
SL-2



North South

Seismic Line
SL-3



North

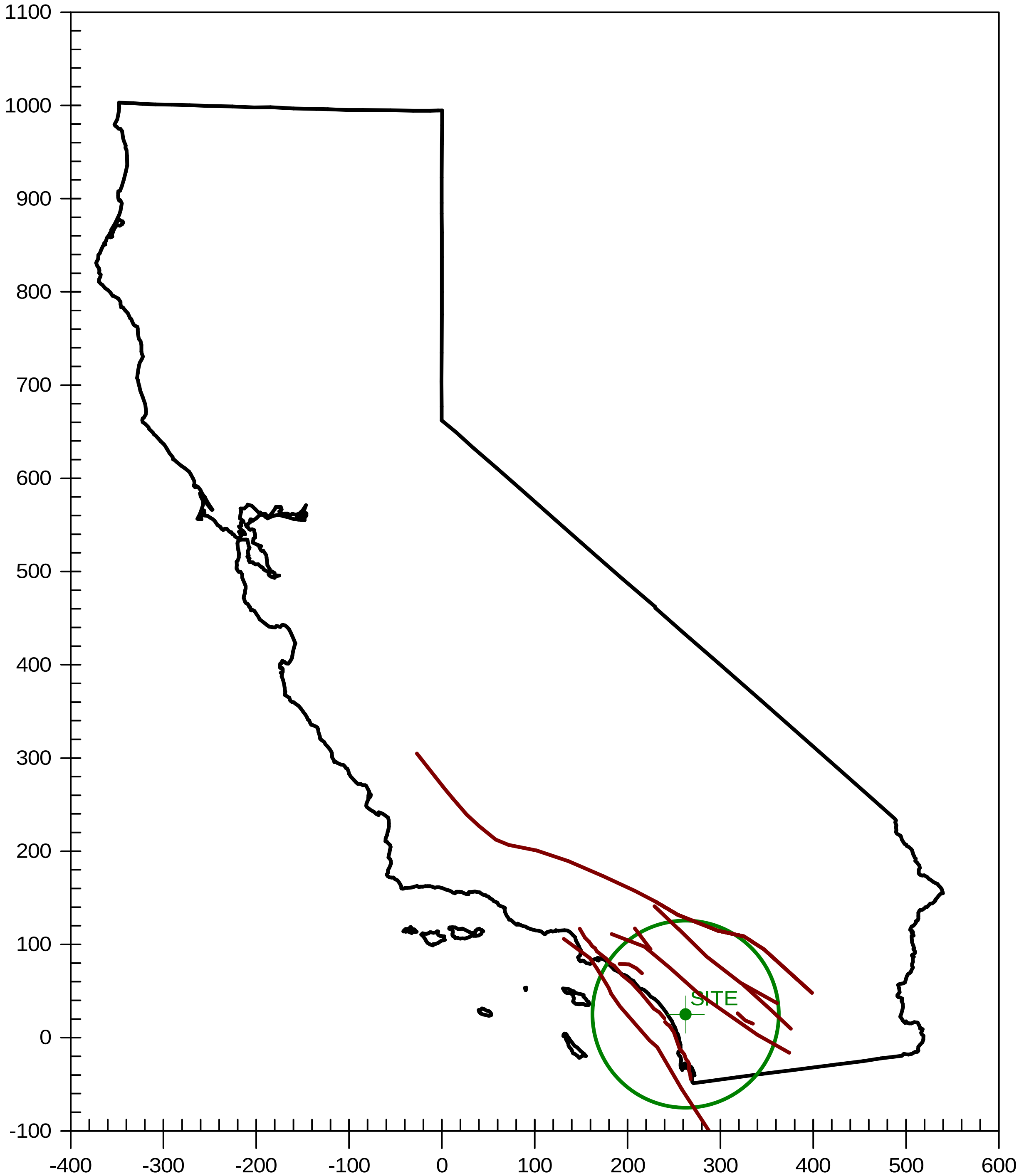
South

APPENDIX D

SEISMICITY DATA

CALIFORNIA FAULT MAP

Tomlinson



W.O. 6814-A-SC
PLATE D-1

TEST.OUT

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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 6814

DATE: 01-23-2015

JOB NAME: Tomlinson

CALCULATION NAME: Test Run Analysis

FAULT-DATA-FILE NAME: C:\EQ\EQFAULT\CGSFLTE.DAT

SITE COORDINATES:
SITE LATITUDE: 33.1878
SITE LONGITUDE: 117.1812

SEARCH RADIUS: 62.5 mi

ATTENUATION RELATION: 13) Bozorgnia Campbell Niazi (1999) Hor.-Hard Rock-Cor.
UNCERTAINTY (M=Median, S=Sigma): S Number of Sigmas: 1.0
DISTANCE MEASURE: cdist
SCOND: 1
Basement Depth: 5.00 km Campbell SSR: 0 Campbell SHR: 1
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\EQ\EQFAULT\CGSFLTE.DAT

MINIMUM DEPTH VALUE (km): 3.0

TEST.OUT

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

Page 1

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
ROSE CANYON	13.5(21.7)	7.2	0.285	IX
NEWPORT-INGLEWOOD (Offshore)	14.6(23.5)	7.1	0.246	IX
ELSINORE (TEMECULA)	16.4(26.4)	6.8	0.179	VIII
ELSINORE (JULIAN)	16.4(26.4)	7.1	0.219	IX
CORONADO BANK	29.1(46.8)	7.6	0.173	VIII
ELSINORE (GLEN IVY)	32.9(53.0)	6.8	0.086	VII
EARTHQUAKE VALLEY	34.7(55.9)	6.5	0.067	VI
SAN JACINTO-ANZA	39.2(63.1)	7.2	0.095	VII
SAN JOAQUIN HILLS	40.2(64.7)	6.6	0.087	VII
SAN JACINTO-SAN JACINTO VALLEY	41.1(66.2)	6.9	0.073	VII
SAN JACINTO-COYOTE CREEK	43.2(69.5)	6.6	0.057	VI
PALOS VERDES	44.4(71.4)	7.3	0.090	VII
CHINO-CENTRAL AVE. (Elsinore)	49.5(79.6)	6.7	0.074	VII
ELSINORE (COYOTE MOUNTAIN)	49.8(80.2)	6.8	0.056	VI
NEWPORT-INGLEWOOD (L.A.Basin)	51.8(83.4)	7.1	0.066	VI
WHITTIER	53.4(85.9)	6.8	0.052	VI
SAN JACINTO - BORREGO	57.1(91.9)	6.6	0.042	VI
SAN JACINTO-SAN BERNARDINO	57.4(92.4)	6.7	0.045	VI
SAN ANDREAS - Whole M-1a	60.1(96.7)	8.0	0.110	VII
SAN ANDREAS - San Bernardino M-1	60.1(96.7)	7.5	0.075	VII
SAN ANDREAS - SB-Coach. M-1b-2	60.1(96.7)	7.7	0.087	VII
SAN ANDREAS - SB-Coach. M-2b	60.1(96.7)	7.7	0.087	VII

-END OF SEARCH- 22 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

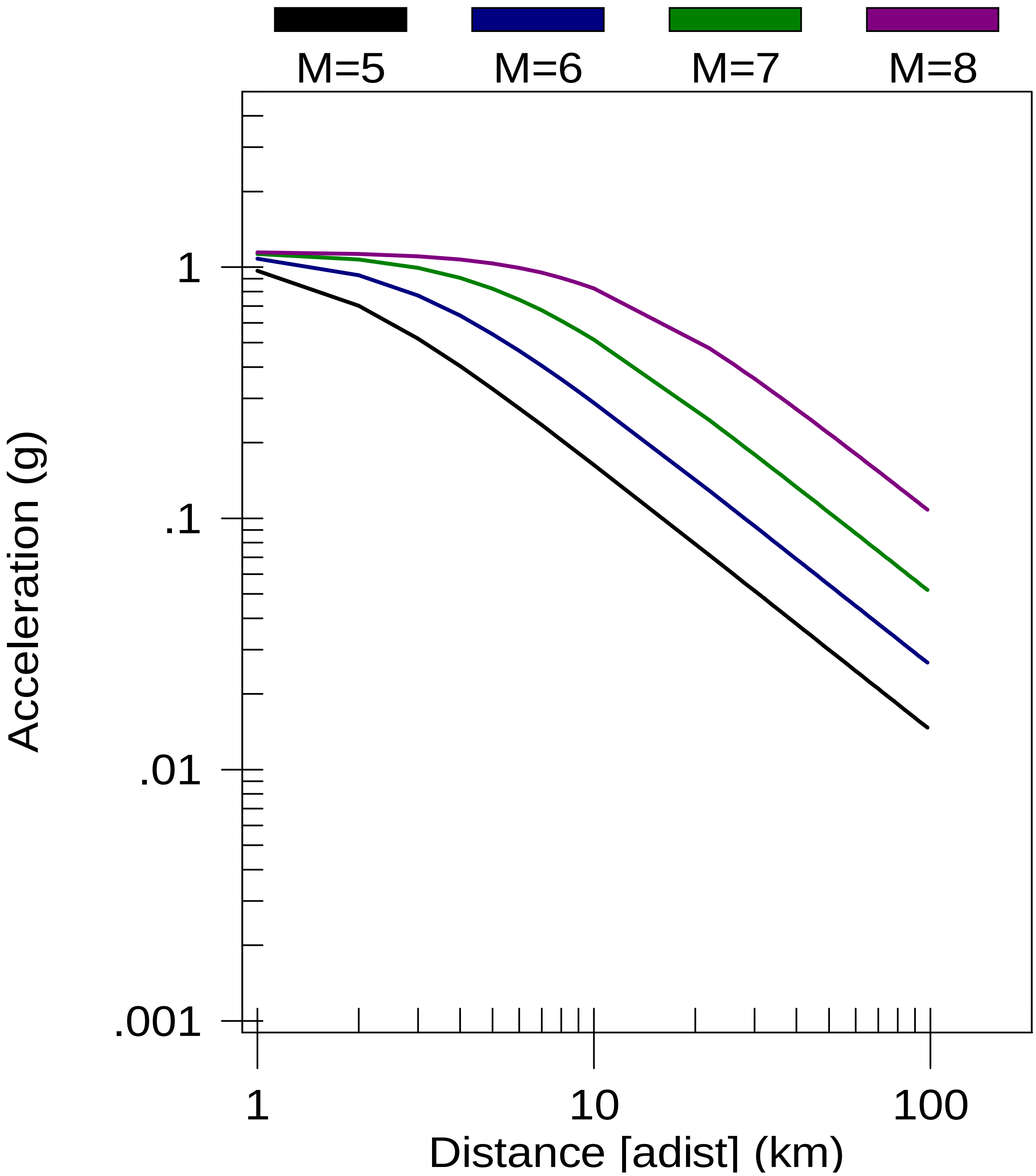
THE ROSE CANYON FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 13.5 MILES (21.7 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.2852 g

W.O. 6814-A-SC
PLATE D-3

STRIKE-SLIP FAULTS

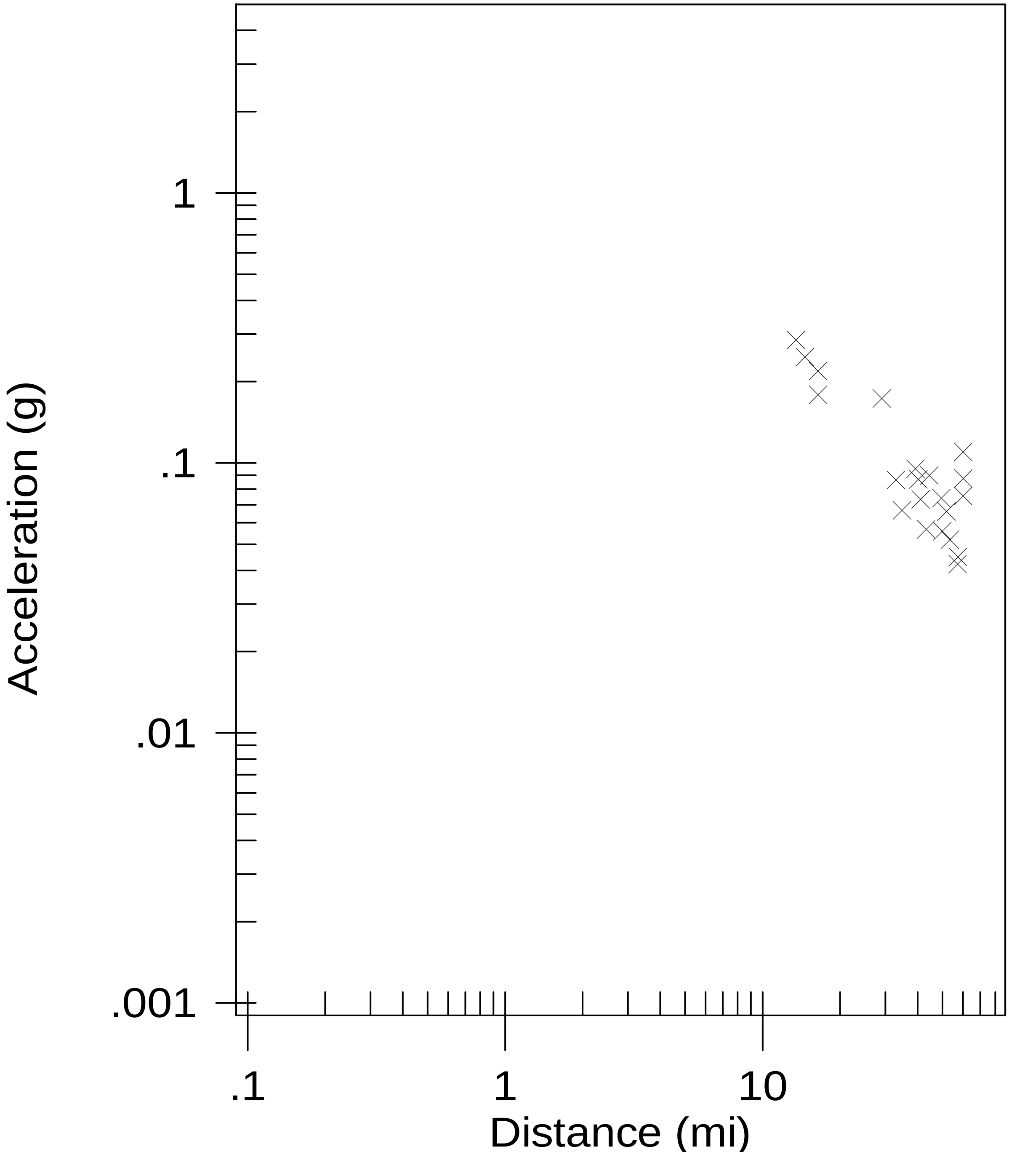
13) Bozorgnia Campbell Niazi (1999) Hor.-Hard Rock-Cor.



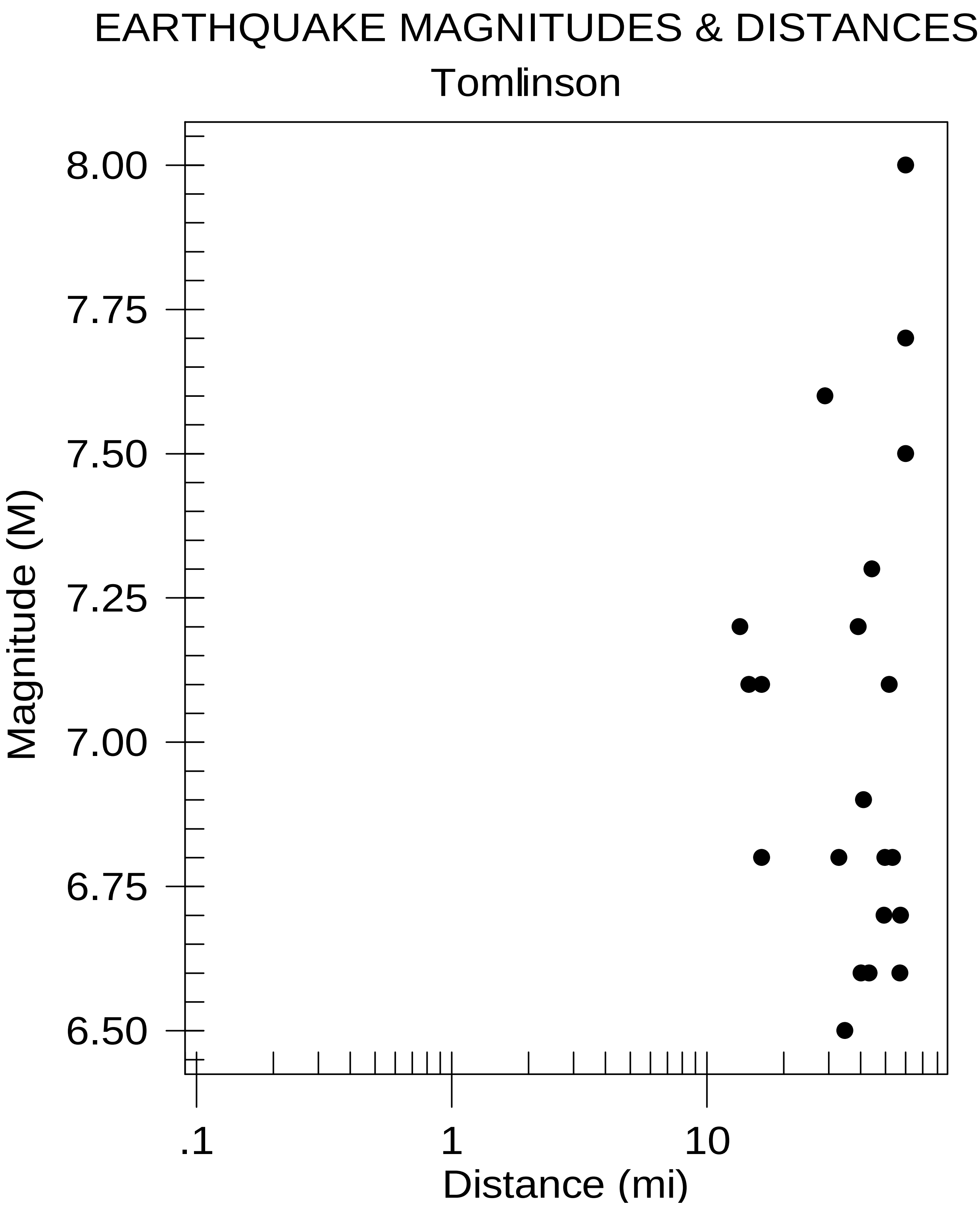
W.O. 6814-A-SC
PLATE D-4

MAXIMUM EARTHQUAKES

Tomlinson



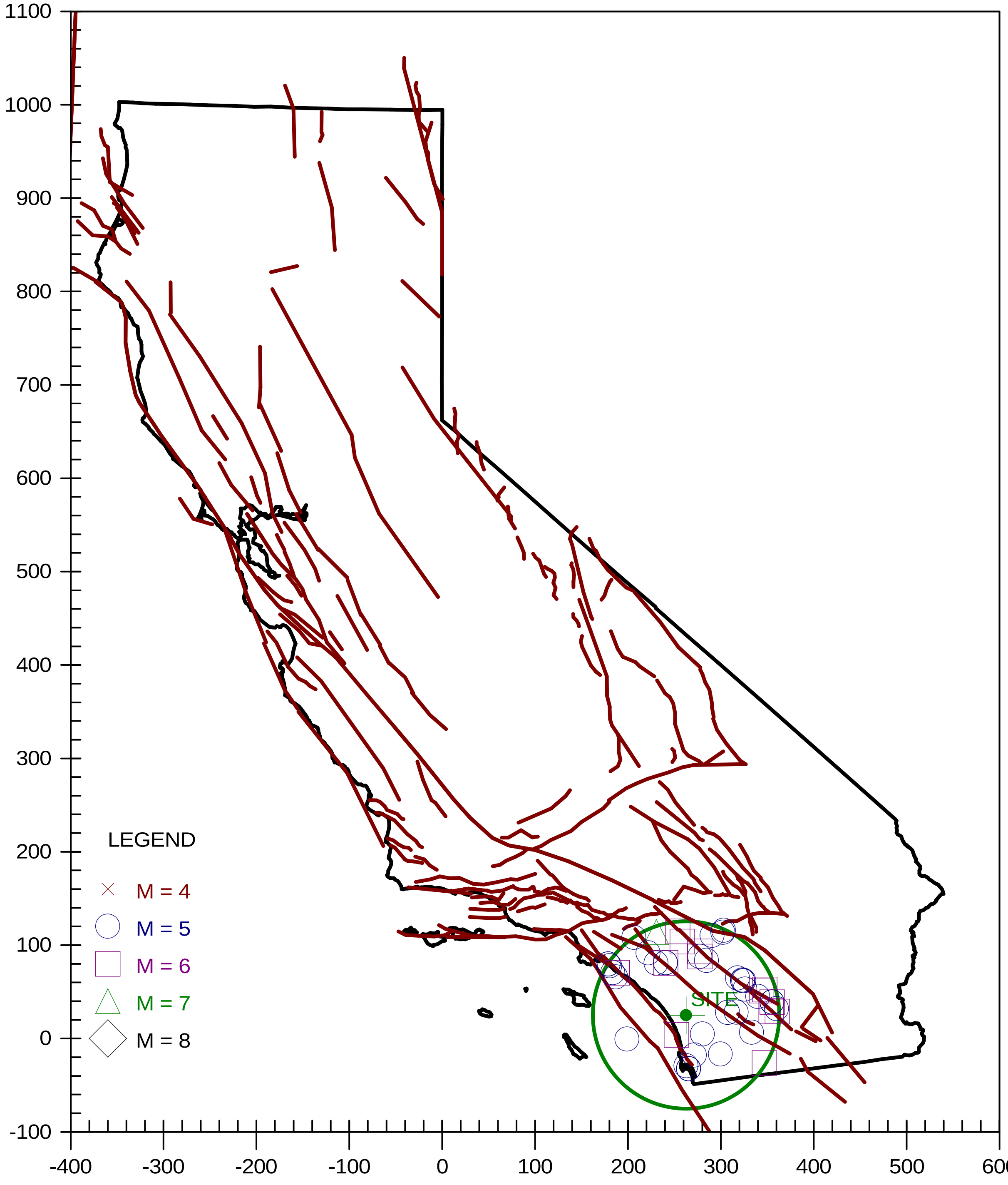
W.O. 6814-A-SC
PLATE D-5



W.O. 6814-A-SC
PLATE D-6

EARTHQUAKE EPICENTER MAP

Tomlinson



W.O. 6814-A-SC
PLATE D-7

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ESTIMATION OF
PEAK ACCELERATION FROM
CALIFORNIA EARTHQUAKE CATALOGS

JOB NUMBER: 6814

DATE: 01-23-2015

JOB NAME: Tomlinson

EARTHQUAKE-CATALOG-FILE NAME: ALLQUAKE.DAT

MAGNITUDE RANGE:
   MINIMUM MAGNITUDE: 5.00
   MAXIMUM MAGNITUDE: 9.00

SITE COORDINATES:
   SITE LATITUDE: 33.1878
   SITE LONGITUDE: 117.1812

SEARCH DATES:
   START DATE: 1800
   END DATE: 2015

SEARCH RADIUS:
   62.5 mi
   100.6 km

ATTENUATION RELATION: 13) Bozorgnia Campbell Niazi (1999) Hor.-Hard Rock-Cor.
   UNCERTAINTY (M=Median, S=Sigma): S   Number of Sigmas: 1.0
   ASSUMED SOURCE TYPE: SS [SS=Strike-slip, DS=Reverse-slip, BT=Blind-thrust]
   SCOND: 1 Depth Source: A
   Basement Depth: 5.00 km   Campbell SSR: 0   Campbell SHR: 1
   COMPUTE PEAK HORIZONTAL ACCELERATION

MINIMUM DEPTH VALUE (km): 3.0

EARTHQUAKE SEARCH RESULTS

Page 1

FILE CODE	LAT. NORTH	LONG. WEST	DATE	TIME (UTC) H M Sec	DEPTH (km)	QUAKE MAG.	SITE ACC. g	SITE MM INT.	APPROX. DISTANCE mi [km]
DMG	33.0000	117.3000	11/22/1800	2130 0.0	0.0	6.50	0.163	VIII	14.7(23.6)
MGI	33.0000	117.0000	09/21/1856	730 0.0	0.0	5.00	0.056	VI	16.7(26.8)
MGI	32.8000	117.1000	05/25/1803	0 0 0.0	0.0	5.00	0.034	V	27.2(43.7)
DMG	33.2000	116.7000	01/01/1920	235 0.0	0.0	5.00	0.033	V	27.8(44.8)
MGI	33.2000	116.6000	10/12/1920	1748 0.0	0.0	5.30	0.033	V	33.6(54.1)
DMG	32.7000	117.2000	05/27/1862	20 0 0.0	0.0	5.90	0.047	VI	33.7(54.2)
DMG	32.8000	116.8000	10/23/1894	23 3 0.0	0.0	5.70	0.040	V	34.7(55.8)
T-A	32.6700	117.1700	05/24/1865	0 0 0.0	0.0	5.00	0.026	V	35.8(57.5)
T-A	32.6700	117.1700	12/00/1856	0 0 0.0	0.0	5.00	0.026	V	35.8(57.5)
T-A	32.6700	117.1700	10/21/1862	0 0 0.0	0.0	5.00	0.026	V	35.8(57.5)
DMG	33.7000	117.4000	04/11/1910	757 0.0	0.0	5.00	0.024	V	37.5(60.4)
DMG	33.7000	117.4000	05/15/1910	1547 0.0	0.0	6.00	0.044	VI	37.5(60.4)
DMG	33.7000	117.4000	05/13/1910	620 0.0	0.0	5.00	0.024	V	37.5(60.4)
DMG	33.7100	116.9250	09/23/1963	144152.6	16.5	5.00	0.023	IV	39.0(62.7)
DMG	33.6990	117.5110	05/31/1938	83455.4	10.0	5.50	0.030	V	40.1(64.5)
DMG	33.7500	117.0000	06/06/1918	2232 0.0	0.0	5.00	0.023	IV	40.2(64.7)
DMG	33.7500	117.0000	04/21/1918	223225.0	0.0	6.80	0.070	VI	40.2(64.7)
GSP	33.5290	116.5720	06/12/2005	154146.5	14.0	5.20	0.024	V	42.3(68.1)
PAS	32.9710	117.8700	07/13/1986	1347 8.2	6.0	5.30	0.025	V	42.6(68.5)
GSG	33.4200	116.4890	07/07/2010	235333.5	14.0	5.50	0.028	V	43.0(69.3)
DMG	33.8000	117.0000	12/25/1899	1225 0.0	0.0	6.40	0.049	VI	43.5(70.1)
PAS	33.5010	116.5130	02/25/1980	104738.5	13.6	5.50	0.027	V	44.2(71.1)
GSP	33.5080	116.5140	10/31/2001	075616.6	15.0	5.10	0.022	IV	44.4(71.4)
DMG	33.5000	116.5000	09/30/1916	211 0.0	0.0	5.00	0.020	IV	44.8(72.1)
DMG	33.0000	116.4330	06/04/1940	1035 8.3	0.0	5.10	0.021	IV	45.2(72.7)
MGI	33.8000	117.6000	04/22/1918	2115 0.0	0.0	5.00	0.019	IV	48.7(78.3)
DMG	33.9000	117.2000	12/19/1880	0 0 0.0	0.0	6.00	0.033	V	49.2(79.1)
DMG	33.3430	116.3460	04/28/1969	232042.9	20.0	5.80	0.029	V	49.4(79.5)
DMG	33.4000	116.3000	02/09/1890	12 6 0.0	0.0	6.30	0.037	V	52.9(85.2)
DMG	33.5750	117.9830	03/11/1933	518 4.0	0.0	5.20	0.019	IV	53.4(85.9)
DMG	33.6170	117.9670	03/11/1933	154 7.8	0.0	6.30	0.037	V	54.1(87.1)
DMG	33.4080	116.2610	03/25/1937	1649 1.8	10.0	6.00	0.030	V	55.2(88.9)
DMG	33.9500	116.8500	09/28/1946	719 9.0	0.0	5.00	0.016	IV	56.0(90.1)
DMG	34.0000	117.2500	07/23/1923	73026.0	0.0	6.25	0.034	V	56.2(90.5)
DMG	33.6170	118.0170	03/14/1933	19 150.0	0.0	5.10	0.017	IV	56.6(91.0)
DMG	33.2000	116.2000	05/28/1892	1115 0.0	0.0	6.30	0.035	V	56.7(91.2)
DMG	33.2830	116.1830	03/23/1954	41450.0	0.0	5.10	0.016	IV	58.0(93.4)
DMG	33.2830	116.1830	03/19/1954	102117.0	0.0	5.50	0.021	IV	58.0(93.4)
DMG	33.2830	116.1830	03/19/1954	95429.0	0.0	6.20	0.032	V	58.0(93.4)
DMG	33.2830	116.1830	03/19/1954	95556.0	0.0	5.00	0.015	IV	58.0(93.4)
MGI	34.0000	117.5000	12/16/1858	10 0 0.0	0.0	7.00	0.054	VI	59.0(94.9)
DMG	33.9760	116.7210	06/12/1944	104534.7	10.0	5.10	0.016	IV	60.5(97.4)
DMG	33.2170	116.1330	08/15/1945	175624.0	0.0	5.70	0.022	IV	60.6(97.5)

DMG	33.6830	118.0500	03/11/1933	658 3.0	0.0	5.50	0.020	IV	60.6(97.5)
DMG	33.1900	116.1290	04/09/1968	22859.1	11.1	6.40	0.035	V	60.8(97.8)
DMG	32.7000	116.3000	02/24/1892	720 0.0	0.0	6.70	0.042	VI	61.2(98.4)
DMG	33.9940	116.7120	06/12/1944	111636.0	10.0	5.30	0.017	IV	61.9(99.5)
DMG	33.7000	118.0670	03/11/1933	85457.0	0.0	5.10	0.015	IV	62.1(99.9)
DMG	33.7000	118.0670	03/11/1933	51022.0	0.0	5.10	0.015	IV	62.1(99.9)
GSG	33.9530	117.7610	07/29/2008	184215.7	14.0	5.30	0.017	IV	62.5(100.5)

****

-END OF SEARCH- 50 EARTHQUAKES FOUND WITHIN THE SPECIFIED SEARCH AREA.

TIME PERIOD OF SEARCH: 1800 TO 2015

LENGTH OF SEARCH TIME: 216 years

THE EARTHQUAKE CLOSEST TO THE SITE IS ABOUT 14.7 MILES (23.6 km) AWAY.

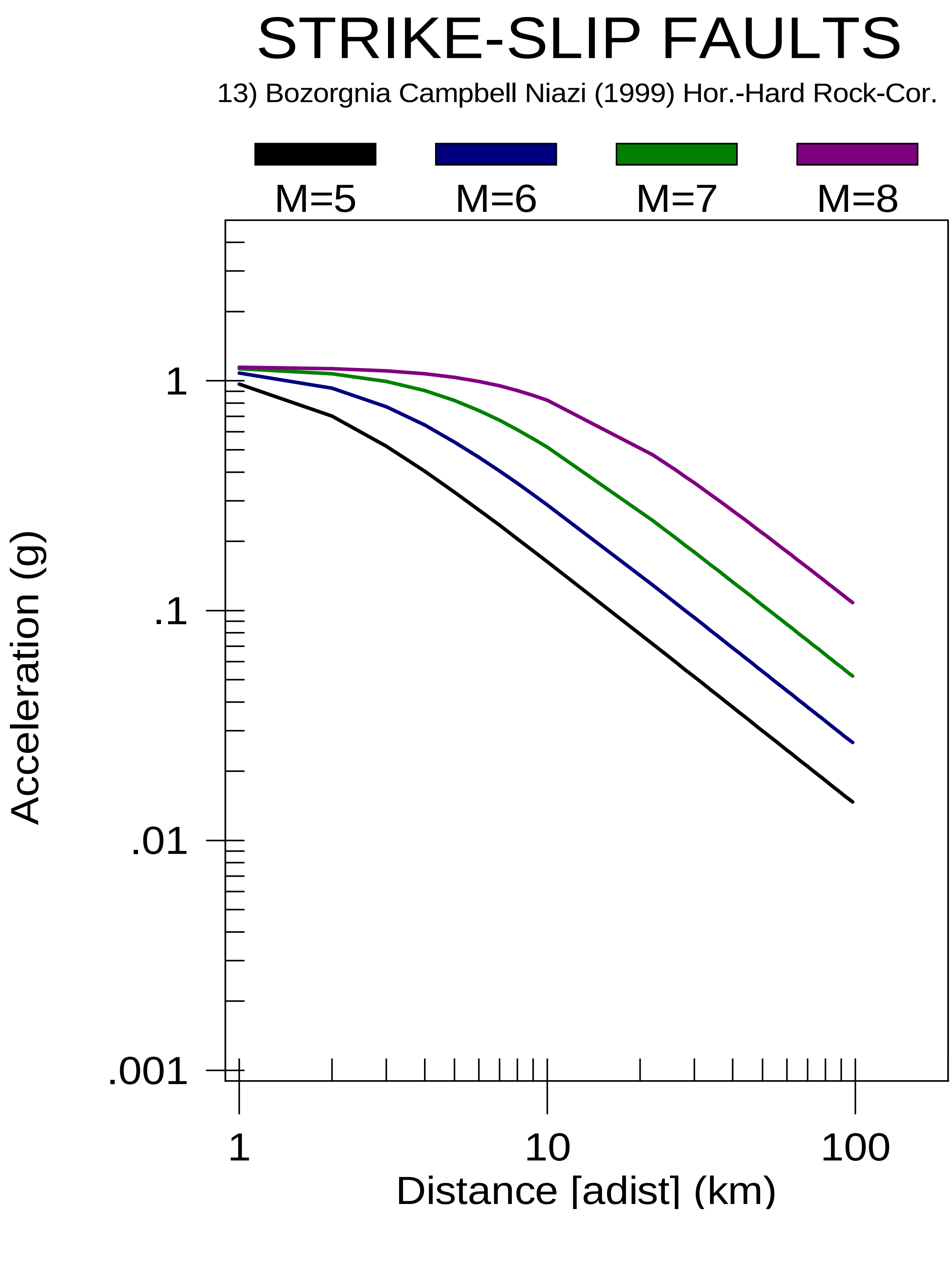
LARGEST EARTHQUAKE MAGNITUDE FOUND IN THE SEARCH RADIUS: 7.0

LARGEST EARTHQUAKE SITE ACCELERATION FROM THIS SEARCH: 0.163 g

COEFFICIENTS FOR GUTENBERG & RICHTER RECURRENCE RELATION:
a-value= 0.629
b-value= 0.289
beta-value= 0.664

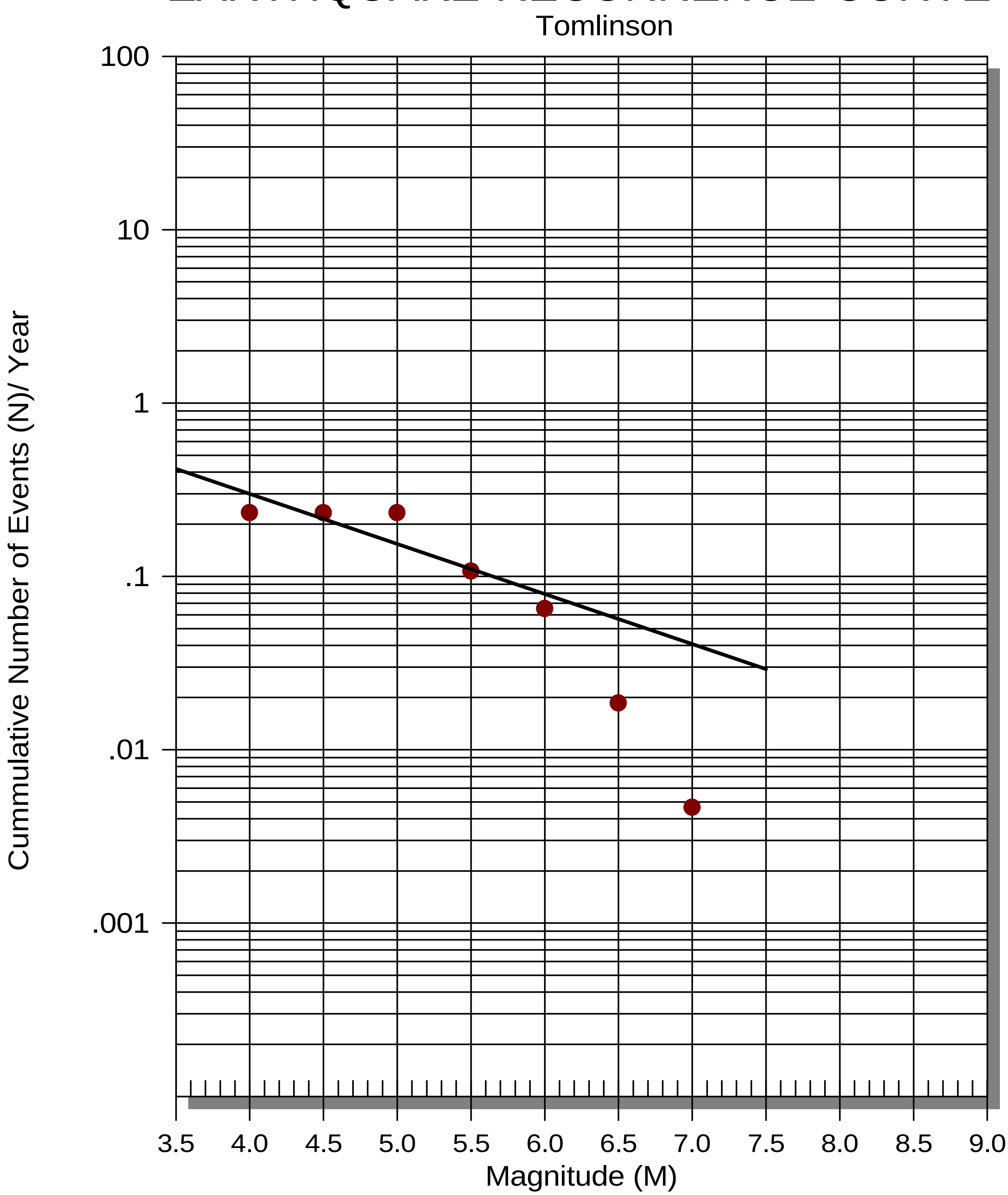
TABLE OF MAGNITUDES AND EXCEEDANCES:

Earthquake Magnitude	Number of Times Exceeded	Cumulative No. / Year
4.0	50	0.23148
4.5	50	0.23148
5.0	50	0.23148
5.5	23	0.10648
6.0	14	0.06481
6.5	4	0.01852
7.0	1	0.00463

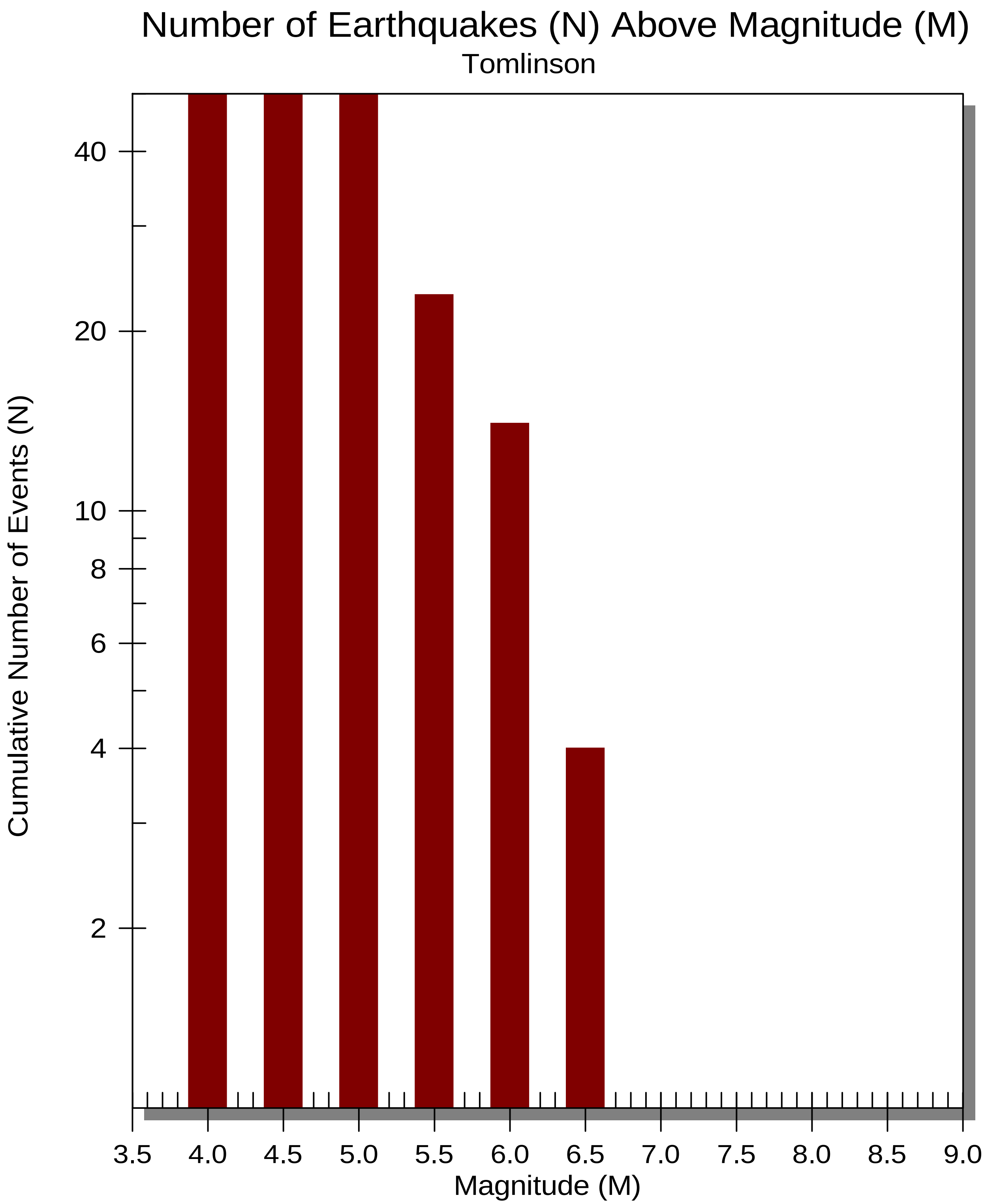


W.O. 6814-A-SC
PLATE D-11

EARTHQUAKE RECURRENCE CURVE



W.O. 6814-A-SC
PLATE D-12



W.O. 6814-A-SC
PLATE D-13

APPENDIX E

SOIL SURVEY REPORT

United States Department of Agriculture

NRCS

Natural Resources Conservation Service

A product of the National Cooperative Soil Survey, a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local participants

Custom Soil Resource Report for

San Diego County Area, California

December 31, 2014

Preface

Soil surveys contain information that affects land use planning in survey areas. They highlight soil limitations that affect various land uses and provide information about the properties of the soils in the survey areas. Soil surveys are designed for many different users, including farmers, ranchers, foresters, agronomists, urban planners, community officials, engineers, developers, builders, and home buyers. Also, conservationists, teachers, students, and specialists in recreation, waste disposal, and pollution control can use the surveys to help them understand, protect, or enhance the environment.

Various land use regulations of Federal, State, and local governments may impose special restrictions on land use or land treatment. Soil surveys identify soil properties that are used in making various land use or land treatment decisions. The information is intended to help the land users identify and reduce the effects of soil limitations on various land uses. The landowner or user is responsible for identifying and complying with existing laws and regulations.

Although soil survey information can be used for general farm, local, and wider area planning, onsite investigation is needed to supplement this information in some cases. Examples include soil quality assessments (http://www.nrcs.usda.gov/wps/portal/nrcs/main/soils/health/) and certain conservation and engineering applications. For more detailed information, contact your local USDA Service Center (http://offices.sc.egov.usda.gov/locator/app?agency=nrcs) or your NRCS State Soil Scientist (http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/contactus/?cid=nrcs142p2_053951).

Great differences in soil properties can occur within short distances. Some soils are seasonally wet or subject to flooding. Some are too unstable to be used as a foundation for buildings or roads. Clayey or wet soils are poorly suited to use as septic tank absorption fields. A high water table makes a soil poorly suited to basements or underground installations.

The National Cooperative Soil Survey is a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local agencies. The Natural Resources Conservation Service (NRCS) has leadership for the Federal part of the National Cooperative Soil Survey.

Information about soils is updated periodically. Updated information is available through the NRCS Web Soil Survey, the site for official soil survey information.

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Contents

How Soil Surveys Are Made

Soil surveys are made to provide information about the soils and miscellaneous areas in a specific area. They include a description of the soils and miscellaneous areas and their location on the landscape and tables that show soil properties and limitations affecting various uses. Soil scientists observed the steepness, length, and shape of the slopes; the general pattern of drainage; the kinds of crops and native plants; and the kinds of bedrock. They observed and described many soil profiles. A soil profile is the sequence of natural layers, or horizons, in a soil. The profile extends from the surface down into the unconsolidated material in which the soil formed or from the surface down to bedrock. The unconsolidated material is devoid of roots and other living organisms and has not been changed by other biological activity.

Currently, soils are mapped according to the boundaries of major land resource areas (MLRAs). MLRAs are geographically associated land resource units that share common characteristics related to physiography, geology, climate, water resources, soils, biological resources, and land uses (USDA, 2006). Soil survey areas typically consist of parts of one or more MLRA.

The soils and miscellaneous areas in a survey area occur in an orderly pattern that is related to the geology, landforms, relief, climate, and natural vegetation of the area. Each kind of soil and miscellaneous area is associated with a particular kind of landform or with a segment of the landform. By observing the soils and miscellaneous areas in the survey area and relating their position to specific segments of the landform, a soil scientist develops a concept, or model, of how they were formed. Thus, during mapping, this model enables the soil scientist to predict with a considerable degree of accuracy the kind of soil or miscellaneous area at a specific location on the landscape.

Commonly, individual soils on the landscape merge into one another as their characteristics gradually change. To construct an accurate soil map, however, soil scientists must determine the boundaries between the soils. They can observe only a limited number of soil profiles. Nevertheless, these observations, supplemented by an understanding of the soil-vegetation-landscape relationship, are sufficient to verify predictions of the kinds of soil in an area and to determine the boundaries.

Soil scientists recorded the characteristics of the soil profiles that they studied. They noted soil color, texture, size and shape of soil aggregates, kind and amount of rock fragments, distribution of plant roots, reaction, and other features that enable them to identify soils. After describing the soils in the survey area and determining their properties, the soil scientists assigned the soils to taxonomic classes (units). Taxonomic classes are concepts. Each taxonomic class has a set of soil characteristics with precisely defined limits. The classes are used as a basis for comparison to classify soils systematically. Soil taxonomy, the system of taxonomic classification used in the United States, is based mainly on the kind and character of soil properties and the arrangement of horizons within the profile. After the soil scientists classified and named the soils in the survey area, they compared the individual soils with similar soils in the same taxonomic class in other areas so that they could confirm data and assemble additional data based on experience and research.

The objective of soil mapping is not to delineate pure map unit components; the objective is to separate the landscape into landforms or landform segments that have similar use and management requirements. Each map unit is defined by a unique combination of soil components and/or miscellaneous areas in predictable proportions. Some components may be highly contrasting to the other components of the map unit. The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The delineation of such landforms and landform segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, onsite investigation is needed to define and locate the soils and miscellaneous areas.

Soil scientists make many field observations in the process of producing a soil map. The frequency of observation is dependent upon several factors, including scale of mapping, intensity of mapping, design of map units, complexity of the landscape, and experience of the soil scientist. Observations are made to test and refine the soil-landscape model and predictions and to verify the classification of the soils at specific locations. Once the soil-landscape model is refined, a significantly smaller number of measurements of individual soil properties are made and recorded. These measurements may include field measurements, such as those for color, depth to bedrock, and texture, and laboratory measurements, such as those for content of sand, silt, clay, salt, and other components. Properties of each soil typically vary from one point to another across the landscape.

Observations for map unit components are aggregated to develop ranges of characteristics for the components. The aggregated values are presented. Direct measurements do not exist for every property presented for every map unit component. Values for some properties are estimated from combinations of other properties.

While a soil survey is in progress, samples of some of the soils in the area generally are collected for laboratory analyses and for engineering tests. Soil scientists interpret the data from these analyses and tests as well as the field-observed characteristics and the soil properties to determine the expected behavior of the soils under different uses. Interpretations for all of the soils are field tested through observation of the soils in different uses and under different levels of management. Some interpretations are modified to fit local conditions, and some new interpretations are developed to meet local needs. Data are assembled from other sources, such as research information, production records, and field experience of specialists. For example, data on crop yields under defined levels of management are assembled from farm records and from field or plot experiments on the same kinds of soil.

Predictions about soil behavior are based not only on soil properties but also on such variables as climate and biological activity. Soil conditions are predictable over long periods of time, but they are not predictable from year to year. For example, soil scientists can predict with a fairly high degree of accuracy that a given soil will have a high water table within certain depths in most years, but they cannot predict that a high water table will always be at a specific level in the soil on a specific date.

After soil scientists located and identified the significant natural bodies of soil in the survey area, they drew the boundaries of these bodies on aerial photographs and identified each as a specific map unit. Aerial photographs show trees, buildings, fields, roads, and rivers, all of which help in locating boundaries accurately.

Soil Map

The soil map section includes the soil map for the defined area of interest, a list of soil map units on the map and extent of each map unit, and cartographic symbols displayed on the map. Also presented are various metadata about data used to produce the map, and a description of each soil map unit.

Soil Map


WmC
HrC
EsE2
EsC
Hollyberry Dr
Buena Creek
Bluebird Canyon Rd
117° 11' 4" W
117° 10' 41" W
33° 11' 22" N
33° 11' 10" N
482820
482880
482940
483000
483060
483120
483180
483240
483300
483360
3672320
3672260
3672200
3672140
3672080
3672020
3671960
Map Scale: 1:2,650 if printed on A landscape (11" x 8.5") sheet.
Meters
0
35
70
140
210
Feet
0
100
200
400
600
N
Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 11N WGS84

MAP LEGEND

Area of Interest (AOI)

Area of Interest (AOI)

Soils

Soil Map Unit Polygons

Soil Map Unit Lines

Soil Map Unit Points

Special Point Features

Blowout

Borrow Pit

Clay Spot

Closed Depression

Gravel Pit

Gravelly Spot

Landfill

Lava Flow

Marsh or swamp

Mine or Quarry

Miscellaneous Water

Perennial Water

Rock Outcrop

Saline Spot

Sandy Spot

Severely Eroded Spot

Sinkhole

Slide or Slip

Sodic Spot

Spoil Area

Stony Spot

Very Stony Spot

Wet Spot

Other

Special Line Features

Water Features

Streams and Canals

Transportation

Rails

Interstate Highways

US Routes

Major Roads

Local Roads

Background

Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: San Diego County Area, California
Survey Area Data: Version 8, Sep 17, 2014

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: May 3, 2010—Jun 19, 2010

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

San Diego County Area, California (CA638)			
Map Unit Symbol	**Map Unit Name**	**Acres in AOI**	**Percent of AOI**
EsC	Escondido very fine sandy loam, 5 to 9 percent slopes	5.4	34.7%
EsE2	Escondido very fine sandy loam, 15 to 30 percent slopes , eroded	0.0	0.0%
HrC	Huerhuero loam, 2 to 9 percent slopes	10.0	64.4%
WmC	Wyman loam, 5 to 9 percent slopes	0.1	0.9%
Totals for Area of Interest		**15.5**	**100.0%**

Map Unit Descriptions

The map units delineated on the detailed soil maps in a soil survey represent the soils or miscellaneous areas in the survey area. The map unit descriptions, along with the maps, can be used to determine the composition and properties of a unit.

A map unit delineation on a soil map represents an area dominated by one or more major kinds of soil or miscellaneous areas. A map unit is identified and named according to the taxonomic classification of the dominant soils. Within a taxonomic class there are precisely defined limits for the properties of the soils. On the landscape, however, the soils are natural phenomena, and they have the characteristic variability of all natural phenomena. Thus, the range of some observed properties may extend beyond the limits defined for a taxonomic class. Areas of soils of a single taxonomic class rarely, if ever, can be mapped without including areas of other taxonomic classes. Consequently, every map unit is made up of the soils or miscellaneous areas for which it is named and some minor components that belong to taxonomic classes other than those of the major soils.

Most minor soils have properties similar to those of the dominant soil or soils in the map unit, and thus they do not affect use and management. These are called noncontrasting, or similar, components. They may or may not be mentioned in a particular map unit description. Other minor components, however, have properties and behavioral characteristics divergent enough to affect use or to require different management. These are called contrasting, or dissimilar, components. They generally are in small areas and could not be mapped separately because of the scale used. Some small areas of strongly contrasting soils or miscellaneous areas are identified by a special symbol on the maps. If included in the database for a given area, the contrasting minor components are identified in the map unit descriptions along with some characteristics of each. A few areas of minor components may not have been observed, and consequently they are not mentioned in the descriptions, especially where the pattern was so complex that it was impractical to make enough observations to identify all the soils and miscellaneous areas on the landscape.

The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The objective of mapping is not to delineate pure taxonomic classes but rather to separate the landscape into landforms or landform segments that have similar use and management requirements. The delineation of such segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, however, onsite investigation is needed to define and locate the soils and miscellaneous areas.

An identifying symbol precedes the map unit name in the map unit descriptions. Each description includes general facts about the unit and gives important soil properties and qualities.

Soils that have profiles that are almost alike make up a *soil series*. Except for differences in texture of the surface layer, all the soils of a series have major horizons that are similar in composition, thickness, and arrangement.

Soils of one series can differ in texture of the surface layer, slope, stoniness, salinity, degree of erosion, and other characteristics that affect their use. On the basis of such differences, a soil series is divided into *soil phases*. Most of the areas shown on the detailed soil maps are phases of soil series. The name of a soil phase commonly indicates a feature that affects use or management. For example, Alpha silt loam, 0 to 2 percent slopes, is a phase of the Alpha series.

Some map units are made up of two or more major soils or miscellaneous areas. These map units are complexes, associations, or undifferentiated groups.

A *complex* consists of two or more soils or miscellaneous areas in such an intricate pattern or in such small areas that they cannot be shown separately on the maps. The pattern and proportion of the soils or miscellaneous areas are somewhat similar in all areas. Alpha-Beta complex, 0 to 6 percent slopes, is an example.

An *association* is made up of two or more geographically associated soils or miscellaneous areas that are shown as one unit on the maps. Because of present or anticipated uses of the map units in the survey area, it was not considered practical or necessary to map the soils or miscellaneous areas separately. The pattern and relative proportion of the soils or miscellaneous areas are somewhat similar. Alpha-Beta association, 0 to 2 percent slopes, is an example.

An *undifferentiated group* is made up of two or more soils or miscellaneous areas that could be mapped individually but are mapped as one unit because similar interpretations can be made for use and management. The pattern and proportion of the soils or miscellaneous areas in a mapped area are not uniform. An area can be made up of only one of the major soils or miscellaneous areas, or it can be made up of all of them. Alpha and Beta soils, 0 to 2 percent slopes, is an example.

Some surveys include *miscellaneous areas*. Such areas have little or no soil material and support little or no vegetation. Rock outcrop is an example.

San Diego County Area, California

EsC—Escondido very fine sandy loam, 5 to 9 percent slopes

Map Unit Setting

National map unit symbol: hbbk
Elevation: 400 to 2,800 feet
Mean annual precipitation: 10 to 20 inches
Mean annual air temperature: 63 degrees F
Farmland classification: Farmland of statewide importance

Map Unit Composition

Escondido and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Escondido

Setting

Landform: Hillslopes
Landform position (two-dimensional): Backslope
Landform position (three-dimensional): Side slope
Down-slope shape: Convex
Across-slope shape: Convex
Parent material: Residuum weathered from metamorphic rock and sandstone

Typical profile

H1 - 0 to 8 inches: very fine sandy loam
H2 - 8 to 34 inches: silt loam, very fine sandy loam, fine sandy loam
H2 - 8 to 34 inches: unweathered bedrock
H2 - 8 to 34 inches:
H3 - 34 to 38 inches:

Properties and qualities

Slope: 5 to 9 percent
Depth to restrictive feature: 20 to 40 inches to lithic bedrock
Natural drainage class: Well drained
Runoff class: Medium
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.57 to 1.98 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water storage in profile: Very high (about 12.9 inches)

Interpretive groups

Land capability classification (irrigated): 3e
Land capability classification (nonirrigated): 3e
Hydrologic Soil Group: C
Ecological site: Loamy (1975) (R019XD029CA)

Minor Components

Friant

Percent of map unit: 10 percent

Fallbrook

Percent of map unit: 3 percent

Vista

Percent of map unit: 2 percent

EsE2—Escondido very fine sandy loam, 15 to 30 percent slopes , eroded

Map Unit Setting

National map unit symbol: hbbm
Elevation: 400 to 2,800 feet
Mean annual precipitation: 10 to 20 inches
Mean annual air temperature: 63 degrees F
Farmland classification: Not prime farmland

Map Unit Composition

Escondido and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Escondido

Setting

Landform: Hillslopes
Landform position (two-dimensional): Backslope
Landform position (three-dimensional): Side slope
Down-slope shape: Convex
Across-slope shape: Convex
Parent material: Residuum weathered from metamorphic rock and sandstone

Typical profile

H1 - 0 to 6 inches: very fine sandy loam
H2 - 6 to 29 inches: silt loam, very fine sandy loam, fine sandy loam
H2 - 6 to 29 inches: unweathered bedrock
H2 - 6 to 29 inches:
H3 - 29 to 33 inches:

Properties and qualities

Slope: 15 to 30 percent
Depth to restrictive feature: 20 to 40 inches to lithic bedrock
Natural drainage class: Well drained
Runoff class: High
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.57 to 1.98 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water storage in profile: High (about 11.3 inches)

Interpretive groups

Land capability classification (irrigated): 6e

Land capability classification (nonirrigated): 6e
Hydrologic Soil Group: C
Ecological site: Loamy (1975) (R019XD029CA)

Minor Components

Friant
Percent of map unit: 10 percent

Fallbrook
Percent of map unit: 3 percent

Vista
Percent of map unit: 2 percent

HrC—Huerhuero loam, 2 to 9 percent slopes

Map Unit Setting
National map unit symbol: hbcm
Elevation: 1,100 feet
Mean annual precipitation: 12 to 20 inches
Mean annual air temperature: 57 degrees F
Frost-free period: 260 days
Farmland classification: Farmland of statewide importance

Map Unit Composition
Huerhuero and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Huerhuero

Setting
Landform: Marine terraces
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Calcareous alluvium derived from sedimentary rock

Typical profile
H1 - 0 to 12 inches: loam
H2 - 12 to 55 inches: clay loam, clay
H2 - 12 to 55 inches: stratified sand to sandy loam
H3 - 55 to 72 inches:

Properties and qualities
Slope: 2 to 9 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Moderately well drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): Very low to moderately low (0.00 to 0.06 in/hr)
Depth to water table: More than 80 inches

Frequency of flooding: None
Frequency of ponding: None
Salinity, maximum in profile: Nonsaline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 25.0
Available water storage in profile: Moderate (about 6.6 inches)

Interpretive groups

Land capability classification (irrigated): 3e
Land capability classification (nonirrigated): 4e
Hydrologic Soil Group: D
Ecological site: Claypan (1975) (R019XD061CA)

Minor Components

Las flores

Percent of map unit: 5 percent

Stockpen

Percent of map unit: 5 percent

Olivenhain

Percent of map unit: 3 percent

Unnamed, ponded

Percent of map unit: 2 percent
Landform: Depressions

WmC—Wyman loam, 5 to 9 percent slopes

Map Unit Setting

National map unit symbol: hbhl
Elevation: 300 to 2,500 feet
Mean annual precipitation: 9 to 25 inches
Mean annual air temperature: 59 to 63 degrees F
Frost-free period: 200 to 300 days
Farmland classification: Farmland of statewide importance

Map Unit Composition

Wyman and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Wyman

Setting

Landform: Alluvial fans
Landform position (two-dimensional): Toeslope
Landform position (three-dimensional): Riser
Down-slope shape: Linear
Across-slope shape: Convex
Parent material: Basic alluvium derived from granite

Typical profile

H1 - 0 to 13 inches: loam
H2 - 13 to 40 inches: clay loam
H3 - 40 to 67 inches: loam
H4 - 67 to 72 inches: fine sandy loam

Properties and qualities

Slope: 5 to 9 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Well drained
Runoff class: High
Capacity of the most limiting layer to transmit water (Ksat): Moderately high (0.20 to 0.57 in/hr)
Depth to water table: More than 80 inches
Frequency of flooding: None
Frequency of ponding: None
Available water storage in profile: High (about 10.0 inches)

Interpretive groups

Land capability classification (irrigated): 2e
Land capability classification (nonirrigated): 4e
Hydrologic Soil Group: C
Ecological site: Loamy (1975) (R019XD029CA)

Minor Components

Placentia

Percent of map unit: 5 percent

Ramona

Percent of map unit: 5 percent

Visalia

Percent of map unit: 3 percent

Las posas

Percent of map unit: 2 percent

Soil Information for All Uses

Suitabilities and Limitations for Use

The Suitabilities and Limitations for Use section includes various soil interpretations displayed as thematic maps with a summary table for the soil map units in the selected area of interest. A single value or rating for each map unit is generated by aggregating the interpretive ratings of individual map unit components. This aggregation process is defined for each interpretation.

Land Classifications

Land Classifications are specified land use and management groupings that are assigned to soil areas because combinations of soil have similar behavior for specified practices. Most are based on soil properties and other factors that directly influence the specific use of the soil. Example classifications include ecological site classification, farmland classification, irrigated and nonirrigated land capability classification, and hydric rating.

Farmland Classification (Tomlinson)

Farmland classification identifies map units as prime farmland, farmland of statewide importance, farmland of local importance, or unique farmland. It identifies the location and extent of the soils that are best suited to food, feed, fiber, forage, and oilseed crops. NRCS policy and procedures on prime and unique farmlands are published in the "Federal Register," Vol. 43, No. 21, January 31, 1978.