Appendix E

Geotechnical Reports

October 11, 2017
(Updated February 4, 2020)
Kleinfelder Project No.: 20181587.001A

Costco Wholesale
9 Corporate Park, Suite 230
Irvine, California 92606

Attention: Ms. Jenifer Murillo
Director of Real Estate Development

Subject: Geotechnical Review
Proposed Costco Wholesale Warehouse
Northeast of Interstate 215 and Clinton Keith Road
Murrieta, California
CW# 17-0237

Dear Ms. Murillo:

This letter presents the results of Kleinfelder's geotechnical review for the proposed Costco Wholesale Warehouse (CW#17-0237) in Murrieta, California. We understand Costco plans to purchase an approximately 18.8-acre site located at the northeast corner of the Interstate 215 (I-215) and Clinton Keith Road interchange for a new approximately 153,362-square-foot warehouse and fuel facility. The purpose of our services was to provide readily-available geotechnical and geologic information for the project site and the development plans to apprise Costco of potential geotechnical-related risks that may affect the cost and long-term performance of the proposed Costco development. A review of the environmental-related risks is also being performed by Kleinfelder and the results are presented in a Phase I Environmental Site Assessment (ESA) report.

PROJECT INFORMATION

We understand that the project will consist of an approximately 153,362-square-foot Costco Wholesale warehouse and a fuel facility. The warehouse will be a single-story, steel-framed structure (approximately 30 feet in height) with concrete-masonry-unit (CMU) and metal walls. The fuel facility will contain three underground storage tanks (UST), a fuel additive UST, fueling stations, and a pre-manufactured metal canopy. The building surroundings will consist mainly of surface parking with some landscape areas. Parking and drive areas will be paved with either Portland cement concrete or asphalt concrete pavements.

The site has been subject to an ongoing excavation (mass grading) project since approximately 2009 that has resulted in the cutting down of hillsides on the property, with use of the generated rock and gravel material on nearby road and other local projects. In order to construct the Costco project, additional mass grading will be required. Based on recent surface elevations interpreted from survey data from August 14, 2019, surface elevations vary from a high of approximately

24411 Ridge Route Drive, Suite 225, Laguna Hills, CA 92653 p | 949.727.4466 f | 949.727.9242

1,555 feet above sea level near the central part of the Costco site to a low of approximately 1,515 feet along the eastern boundary of the site. The site also slopes towards Antelope Road on the west to an elevation ranging from approximately 1,540 to 1,530 feet along the road. Along the northern boundary of the site, an existing slope has been exposed during the ongoing grading operations, with current elevations ranging from approximately 1,540 to 1,610 feet.

According to the Concept Grading Plan, dated May 14, 2019, prepared by Fuscoe Engineering (2019c), the finished floor elevation (FFE) for the warehouse will be established at approximately Elevation 1,538 feet. The Costco site will be over-excavated an additional 10 feet below the finished pad grade and then backfill with fill soils derived from the on-site material to bring the site to finished grades. Additionally, a large cut slope with a maximum height of 70 feet is planned north of the proposed warehouse, with a slope inclination of 2:1 (H:V). Fill slopes on the order of 10 feet tall are planned along the east and south boundaries of the site with a 2:1 slope inclination. A concrete masonry unit (CMU) retaining wall with a maximum height of 6 feet is planned on the south side of the proposed driveway entrance located east of the proposed warehouse. A new road, Warm Springs Parkway, is planned along the eastern project site boundary extending northward from Clinton Keith Road, and will be constructed by the City of Murrieta.

EXISTING SITE CONDITIONS

The Costco site comprises approximately 18.8 acres within an undeveloped parcel located northeast of the Antelope and Clinton Keith Roads intersection in the northern part of the City of Murrieta, California. A residential neighborhood and I-215 are to the east and west of the site, respectively. The Vista Murrieta High School is to the south of the Costco site and the land bordering the north is undeveloped. A review of historical topographic maps and aerial photographs show that prior to about 2009 the site was undisturbed and appeared to be used for farming. The site's ground surface generally sloped gently to the south from an elevation of approximately 1,550 feet to 1,520 feet. However, this southerly slope was interrupted by two prominent hills which rise above the site about 100-130 feet, or to elevations of about 1,602 and 1,637 feet, south to north respectively. After 2009, it appears that the site has been used as a soil borrow-site with excavation generally limited at the two hill areas located in the northeast and southwest portion of the site. A review of recent survey data provided by Fuscoe Engineering (2019 a and b) dated August 14, 2019 indicates that most of the project site's ground surface has now been disturbed by the ongoing excavation activities. The hills have been heavily excavated with approximately 55 feet being removed from the northernmost hill down to Elevation 1,582 feet. In addition to excavation activities, there is rock crushing equipment present, as well as stockpiles of rock, gravel and soil. Erosional control fencing appears to be present around the perimeter of the site.

GEOTECHNICAL/GEOLOGICAL CONDITIONS

The Costco site is located within the Perris Block, a structural subdivision of the Peninsular Range Geomorphic Province, which is characterized by steep, elongated ranges and valleys that generally trend northwestward [California Geological Survey (CGS), 2002]. The Perris Block is underlain by pre-Cretaceous age (greater than 144 million years old) metasedimentary rocks and Cretaceous age (66 to 144 million years old) plutonic rocks of the Southern California batholith (Larsen, 1948; and Woodford et al., 1971). Within the southern part of the Perris Block, including the project site, plutonic rocks (i.e., granite, granodiorite and gabbro) intruded into the existing, country metamorphic rock (i.e., metasedimentary rocks). Subsequent to the plutonic intrusion was the emplacement of short-arced igneous ring-dikes into the granite and gabbro rock, producing an elliptical shaped deposit known as the Paloma Valley ring complex (Morton and Baird, 1976). As the rocks of the Paloma Valley ring complex weather, the gabbro is more resistant than the granite, granodiorite and igneous ring-dikes, therefore, they generally produce the remnant hills seen in the area, including the two hills onsite.

The project site is underlain by Cretaceous-age gabbro (Kennedy and Morton, 2003; and Geosoils Inc., 2005), or more specifically it is known as the San Marcos gabbro (Miller, 1937). The San Marcos gabbro is medium- to very coarse-grained, with a mineral composition of abundant hornblende, and minor amounts of quartz, biotite and plagioclase (Larsen, 1948; Kennedy and Morton, 2003). An onsite geophysical investigation by Geosoils, Inc. (2005) found that the gabbro to be weathered to varying depths of about 30 to 86 feet across the site from south to north, respectively.

The project site, like the rest of southern California, is located within a seismically active region. Based on the information provided in CGS Special Publication 42 (CGS, 2018), the site is not located within a State-designated Alquist-Priolo (AP) Earthquake Fault Zone, where site-specific studies addressing the potential for surface fault rupture would be required. The most significant known active fault zones that are capable of seismic ground shaking and can impact the site include, the Elsinore Fault Zone, to the southwest and the San Jacinto Fault Zone to the east. The Wildomar fault splay of the Elsinore Fault Zone is closest to the site at about 3.5 miles to the southwest (CGS, 2017), and is capable of generating an earthquake of M6.8. No faults are known to exist on the Costco site and no known faults are mapped trending towards the site (Kennedy and Morton, 2003; and CGS, 2017).

The site is expected to experience strong ground shaking within the life of the Costco building. Liquefaction is not a hazard at the site and the site is not in a CGS-designated liquefaction zone (CGS, 2017; Silva et al., 2017). Groundwater is not reported present onsite (Silva et al., 2017; and Geosoils, Inc., 2005) and the San Marcos gabbro is composed of interlocking minerals with little to no permeability.

FINDINGS

Based on the results of our geotechnical review, it is our professional opinion that the proposed project is geotechnically feasible. The primary geotechnical consideration is the presence of hard bedrock conditions when performing excavations at the site. The following should be considered for cost estimating purposes.

- Based on on-site geophysical investigation by Geosoils, Inc. (2005), the bedrock beneath the building pad, the fuel facility and most of the parking area will most likely be marginally rippable (with some non-rippable areas) and an aggressive means of excavation, such as heavy ripping using a single shank ripper or large ram-hoe and/or micro rock popping, will likely be required. These excavation methods are currently being used on site. The Geosoils investigation indicated the parking area to the east of the warehouse may be rippable down to an elevation of 1,502 feet.
- Based on our review of the Concept Grading Plan (Fuscoe, 2019c), there is a large cut slope planned on the northern boundary of the property and several smaller slopes and a retaining wall located on the east and south site boundaries. Excavations into hard bedrock will be required to construct slopes or retaining walls.
- It is anticipated that some utilities, such as storm drains and sewer lines, will extend deeper

than 10 feet below the finished grades. It is recommended that a conceptual utility plan be developed to identify areas where excavation in hard bedrock will occur.

- Oversized bedrock materials will need to be crushed prior to use as backfill. We understand that the contractor intends to process oversized material (e.g., rock crushing) in order to handle oversized material and avoid the need for import soils.

CLOSING

This work was performed in a manner consistent with that level of care and skill ordinarily exercised by other members of Kleinfelder's profession practicing in the same locality, under similar conditions, and at the date the services are provided. Our conclusions, opinions, and recommendations are based on a limited number of observations and data. It is possible that conditions could vary between or beyond the data evaluated. Kleinfelder makes no other representation, guarantee, or warranty, express or implied, regarding the services, communication (oral or written), report, opinion, or instrument of service provided.

We appreciate the opportunity to be of professional service to you on this project. If you have any questions or require additional information, please do not hesitate to contact the undersigned.

Respectfully submitted,

KLEINFLDER, INC.

Robert Lemmer, PG, CEG
Principal Engineering Geologist

Brian E. Crystal, PE, GE
Senior Project Manager

Attachment: References

REFERENCES

California Geological Survey (CGS), 2002, California Geomorphic Provinces, Note 36.

California Geological Survey (CGS), 2017, Earthquake Zones of Required Investigation, Murrieta Quadrangle, Preliminary Review Map, dated August 17, 2017, scale: 1"=24,000".

California Geological Survey (CGS), 2018, Earthquake Fault Zones – A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California: California Geological Survey Special Publication 42, 83p.

Fuscoe, 2019a, Earthwork Exhibit Costco Parcel, Topo Dated Aug. 14, 2019 VS. Proposed Rough Grade, dated August 14, 2019.

Fuscoe, 2019b, Earthwork Exhibit – CK 17, Topo Dated Aug. 14, 2019 VS. Proposed Rough Grade, dated August 14, 2019.

Fuscoe Engineering, 2019c, Concept Grading Plan, Sheets C1 and C2, Costco Wholesale, Murrieta, CA, dated May 14, 2019, scale: 1"=40'.

GeoSoils, Inc., 2005, Rock Hardness/Rippability and Slope Stability Study, Proposed Borrow Site, Parcel 1 Through 4 of Parcel Map 16499, City of Murrieta, California, W.O. 4919-A-SC, dated October 5, 2005, 16p.

Kennedy, M.P. and Morton, D.M., 2003, Preliminary geologic map of the Murrieta 7.5' quadrangle, Riverside County, California: U.S. Geological Survey, Open-File Report 03-189, scale: 1"=24,000".

Larsen, E.S., 1948, Batholith and associated rocks of Corona, Elsinore, and San Luis Rey quadrangles, southern California: Geological Society of America, Memoir, Volume 29, 182p.

Miller, F.S., 1937, Petrology of the San Marcos Gabbro, southern California: Geological Society of America, Bulletin, Volume 48, No. 10, pp. 1397-1425.

Morton, D.M. and Baird, A.K., 1976, Petrology of the Paloma Valley ring complex, southern California batholith: U.S. Geological Survey, Journal of Research, Volume 4, No. 1, pp. 83-89.

Silva, M.A., Pridmore, C.L., Hernandez, J.L., Irvine, P.J. and Mlinarevic, A.N., 2017, Evaluation Report for Liquefaction Hazard in the Murrieta 7.5-Minute Quadrangle, Riverside County, California *in* Preliminary Seismic Hazard Zone Report for the Murrieta 7.5-Minute Quadrangle, Riverside County, California: California Geological Survey Seismic Hazard Zone Report 115, pp. 1-20.

Woodford, A.O., Shelton, J.S., Doehring, D.O. and Morton, R.K., 1971, Pliocene-Pleistocene history of the Perris Block, southern California: Geological Society of America, Bulletin, Volume 82, No. 12, pp. 3421-3448.

GEOTECHNICAL
PROFESSIONALS INC

PRELIMINARY GEOTECHNICAL INVESTIGATION
PROPOSED VINEYARD/VAL VISTA CENTER
NEC CLINTON KEITH ROAD AND 215 FREEWAY
MURRIETA, CALIFORNIA

Prepared for:
CK 17 LP
c/o Retail Development Advisors
27890 Clinton Keith Road, Suite D-490
Murrieta, California 92562

Prepared by:
Geotechnical Professionals Inc.
5736 Corporate Avenue
Cypress, California 90630
(714) 220-2211

Project No. 2833.I

September 7, 2017

September 7, 2017

CK 17 LP
c/o Retail Development Advisors
27890 Clinton Keith Road, Suite D-490
Murrieta, California 92562

Attention: Mr. Allan Davis

Subject: Report of Preliminary Geotechnical Investigation
Proposed Vineyard/Val Vista Center
NEC Clinton Keith Road and 215 Freeway
Murrieta, California
GPI Project No. 2833.I

Dear Mr. Davis:

Transmitted herewith is our report of preliminary geotechnical investigation for the subject project. The report presents our evaluation of the foundation conditions at the site and recommendations for design and construction.

We are providing this report in an electronic format. Further copies of the report can be provided if required for City submittal upon request.

We appreciate the opportunity of offering our services on this project and look forward to seeing the project through its successful completion. Feel free to call us if you have any questions regarding our report or need further assistance.

Very truly yours,
Geotechnical Professionals Inc.

Paul R. Schade, G.E.
Principal

PRS:sph

2833-I-01L.doc

TABLE OF CONTENTS

LIST OF FIGURES

APPENDIX A

APPENDIX B

1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a preliminary geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for the proposed Vineyard/Val Vista Center development to be located near the northeast corner of Clinton Keith Road and the 215 Freeway in Murrieta, California. The geographical site location is shown on the Site Location Map, Figure 1.

1.2 PROJECT DESCRIPTION

The Vineyard project will be a mixed-use development extending from the northbound I-215 on-ramp to an existing residential development east of the newly planned Warm Springs Parkway alignment on the north side of Clinton Keith Road. The subject site is approximately 46 acres in size. Based on a conceptual site plan provided by CK-17, the proposed development will consist of a 4-story hotel, a Costco warehouse, numerous 1-story retail pads, and associated surface parking. The Costco parcel is being investigated by others and will not be included in our studies. The hotel building will be approximately 91,000 square feet in plan area while the 1-story retail buildings will vary from about 3,500 to 34,000 square feet in plan area. The proposed site configuration and limits of the development covered by this preliminary report are shown on the Site Plan, Figure 2.

Structural loads for the hotel building and retail pads were not available at the time of this preliminary report but are anticipated to be up to 300 kips and 50 kips, respectively. The grades at the site are currently being lowered or are planned to be lowered during the ongoing mining and exporting activities. Within an area covered by this preliminary investigation (between the northbound I-215 on-ramp and the new Warm Springs Parkway), the existing grades appear to be up to about 20 feet above the planned finished site grades. These cuts are planned to be performed prior to the completion of the project design. It is our understanding that select areas of the site have already been overexcavated to allow engineered fill to be placed to reach finished grade.

Based on the provided site plan, the proposed development will also likely include a storm water retention/infiltration system located along the southern and/or western property lines. Details of the infiltration system were not available at the time of this preliminary report but are anticipated to consist of near-surface infiltration via bio-swales or shallow retention basins.

Our preliminary recommendations are based on the above structural and grading information. We should be notified if the project plans change significantly from the above. GPI should review the project plans prior to finalizing the project design.

1.3 PURPOSE OF INVESTIGATION

The primary purpose of this investigation and report is to provide a preliminary evaluation of the existing geotechnical and geologic conditions at the site as they relate to the design and construction of the proposed development. More specifically, this investigation was aimed at providing preliminary geotechnical recommendations for earthwork, and design of foundations, floor slabs, retaining walls, infiltration systems, and pavements.

1.4 PRIOR SITE WORK

Several prior geotechnical investigations have been performed by GPI and others on portions of the subject site. The scope of work for the subject preliminary investigation included a review of these investigations.

GPI previously performed geotechnical investigations at the subject site for the new Warm Springs Parkway (to replace the vacated section of old Antelope Road) and a proposed commercial/retail development located to the east of the new road (GPI, 2009). Although not developed, the previously proposed project is similar to the type of development that is the subject of this report. The findings from our prior investigations were used in our current evaluation. The explorations performed for the prior investigations, as well as the lab testing on samples obtained from these explorations, have been incorporated into this report.

As part of our past work at the site, we were provided with prior investigations performed by others. These investigations included a preliminary geotechnical investigation for a proposed commercial development (EcoTech, 2004), a rock hardness/rippability study of the onsite materials to evaluate the site as a proposed borrow source (GeoSoils, 2005), and another preliminary geotechnical investigation for a proposed commercial/retail development and realignment of Antelope Road (T.H.E. Soils Co., 2008). The pertinent information within these reports was considered in preparing our preliminary report.

2.0 SCOPE OF WORK

Our scope of work for this investigation consisted of a review of existing data, field exploration, limited field infiltration testing, laboratory testing, engineering analysis, and the preparation of this report.

As part of a prior geotechnical investigation at the subject site, we performed 17 hollow-stem auger borings (B-1 to B-17) located along the new Warm Springs Parkway and within the portion of the site between the Warm Springs Parkway alignment and the existing residential developments to the east (GPI, 2009). These exploratory borings were performed to depths ranging from 11 to 26 feet below the site grades at the time of the investigation. Refusal in the dense, less weathered bedrock occurred in two of the borings prior to reaching their planned depths, likely due to the presence of cobbles and boulders.

The new fieldwork performed for this investigation included three hollow-stem auger borings (B-101 to B-103) performed to depths of 31 to 38 feet below existing grades before refusing in the dense, less weathered bedrock. A description of field procedures and logs of the current and prior explorations are presented in Appendix A. The locations of the subsurface explorations are shown on the Site Plan, Figure 2.

Laboratory tests were performed on selected representative samples as an aid in soil classification and to evaluate the engineering properties of the soils. The geotechnical laboratory testing program included determinations of moisture content and dry density, gradation, fines content, Atterberg Limits, direct shear, maximum dry density and optimum moisture content, R-value, and soil corrosivity.

R-value and soil corrosivity testing was performed by GeoLogic, Inc. and Schiff Associates (currently HDR), respectively, under subcontract to GPI. Laboratory testing procedures and results are summarized in Appendix B.

Engineering evaluations were performed to provide preliminary earthwork criteria, foundation and slab design parameters, preliminary pavement sections and assessments of seismic hazards. The results of our evaluations are presented in the remainder of the report.

3.0 SITE CONDITIONS

3.1 SITE HISTORY

Our understanding of the site history is based on a review of the prior investigations by GPI and others and a review of historical aerial photographs (Historic Aerials) dating back to 1938. Up to 1967, the site and surrounding area appears to be largely undeveloped except for a highway (known then as US 395) located west of the subject site area. In 1978, the highway (I-215) is shown being widened and the northbound on-ramp and Antelope Road appear to be under construction. The remaining majority of the site remains undeveloped. Between 1978 and 2002, the site appears to remain unchanged. In 2005, the residential developments to the east of the subject site are now in-place. Sometime between early 2007 and mid-2009, mining and exporting operations began on the northern half of the subject site and the portion of Antelope Road located along the west side of the subject site was closed. Since 2009, the site has remained relatively unchanged apart from the continued mining operations.

3.2 SURFACE CONDITIONS

The site is approximately 46 acres of vacant land and bounded by Clinton Keith Road to the south, the on-ramp to the northbound I-215 freeway to the west, vacant land followed by Cape Aire Way to the north, and a residential development to the east. Excluding the Costco Warehouse parcel, which is being evaluated by others, the portion of the site covered by our evaluation is approximately 28 acres in size.

Ground surface elevations across the site vary from about +1505 feet in the southeast corner to an approximate peak of +1610 feet in the northern middle portion of the site. Due to the ongoing mining and exporting activities, the grades are continuing to be lowered across a majority of the site. Existing slopes are highly variable, with localized areas exhibiting grades as steep as 2.5:1 (horizontal to vertical).

The existing alignment of Antelope Road, located in the western portion of the site, is currently abandoned. Ground surface elevations for Antelope Road vary from approximately +1546 in the north to +1526 in the south at the intersection with Clinton Keith Road. Pavement sections for the road were not determined.

3.3 SUBSURFACE SOILS

Our field investigations disclosed a subsurface profile consisting of topsoil and weathered organic bedrock over less weathered organic bedrock. It should be noted that these descriptions reflect the subsurface profiles at the location and time our explorations were performed. Borings B-1 through B-17 were performed in 2009 and were limited to the eastern half of the subject site. Borings B-101 through B-103 were performed as part of the current investigation and are located along the existing vacated portion of Antelope Road. Since 2009, the ongoing mining/exporting operations have resulted in significant cuts and the potential for significant fills throughout the site. Detailed descriptions of the conditions encountered are shown on the Logs of Borings in Appendix A.

The near-surface soil materials consisted primarily of weathered granitic bedrock (silty sands, sands, and sandy silts) with varying amounts of gravel. Drilling was performed with small diameter equipment, making it difficult to identify potential cobbles and boulders. However, based on our experience (cobbles and boulders were noted within the excavation being performed by others to the northwest of the site) and our observation of surficial conditions, localized boulders and cobbles should be anticipated. During our 2009 investigation, we observed exposed outcrops of boulders on the site that included material greater than 5 feet in diameter. However, the majority of the oversized material exposed in adjacent excavations at the time appeared to be between 2 and 3 feet in diameter. Based on information provided by the site developer, some onsite boulders are as large as 15 feet in diameter.

Based on blowcounts, the native soils generally range from very loose to very dense, with density increasing abruptly with depth and reduction in weathering of the granitic materials. The soils encountered within select 2009 explorations (Borings B-8, 12, 13, 15, 16, and 17) appeared to encounter the loose topsoil to deeper extents than other areas of the site. The moisture contents ranged from dry to moist, generally decreasing in content with depth.

As part of the ongoing mining activities at the site, some fill has been placed in select areas of the site for stockpiling purposes. Based on discussions with the site developer, decomposed granitic bedrock (planned for sale and export) has been placed in two separate stockpiles up to 10 feet in height. The complete extent of stockpile generation as a result of the mining activities is expected to vary with the export operations.

We did not note fill in our explorations performed as part of the current field investigation.

3.4 GROUNDWATER AND CAVING

Groundwater was not encountered in our borings to the maximum explored depth of 38 feet below existing ground surface. State historical records for groundwater were not available in the area of the subject site.

Caving was not noted within the relatively small-diameter hollow-stem auger borings. Localized caving may be encountered in the upper loose and dry sandy deposits; however, the general increase in density and cementation of the granular materials with depth make caving of the deeper materials unlikely if the excavation guidelines presented herein are followed.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on the results of our investigation, it is our opinion that from a geotechnical viewpoint it is feasible to develop the site as proposed. The proposed structures can be supported on shallow foundations following remedial grading to mitigate the geotechnical constraints discussed below. The most significant geotechnical issues that will affect the design and construction of the proposed structures are as follows:

- The site contains topsoil/weathered bedrock over very dense granitic bedrock. The topsoil and upper portions weathered bedrock within the upper 2 to 7 feet below existing site grades is generally loose and not considered suitable for subsequent fill placement or uniform support of footings, slab-on-grade floors, and pavements. We recommend that the topsoil and low density weathered bedrock be removed prior to subsequent fill placement to design grades.

- To provide uniform support for the planned buildings in fill areas, the soils should be removed to a depth of at least 5 feet below the planned finished pad grade or 2 feet below the base of foundations, whichever is greater. Foundations for minor structures, such as site walls, may be supported in the undisturbed bedrock or be underlain by at least 2 feet of properly compacted fill. Deeper overexcavation in the area of deep cuts into the bedrock may be desirable in utility corridors to aid in excavating with conventional equipment.

- The undisturbed bedrock is suitable for direct support of building foundations but may be undesirable for future excavations of footings or utility trenches. To aid in excavation with conventional equipment, we recommend the removals for building pads established entirely in bedrock materials extend to a depth of at least 3 feet below the finished pad grade.

- Based on the variable depths to bedrock across the site, select building footprints may span bedrock and compacted fill. Such buildings will have an increased potential for differential settlement. For these transition pads, we recommend removal depths of at least 3 feet beneath the base of footings across the entire bedrock portion of the pad.

- The required grading will involve excavation into very dense granitic bedrock materials. Heavy ripping and relatively slow earthwork operations are anticipated in most of the deeper cuts. Some difficulty will be encountered in ripping the granitic materials and drilling and shooting or blasting may aid in grading production rates. Large floating boulders will be encountered during grading. Earthwork subcontractor should discuss the subsurface conditions with the site owner and personnel performing the recent mining operations to better understand the excavation issues anticipated.

- Because of the nature of the site bedrock and likely presence of boulders and cobbles, oversize material (greater than 12-inches in maximum dimension) should be anticipated to be generated from site cuts. Special handling and/or crushing of oversize material will be required in order to use the material as compacted fill. On-site placement of the boulders in compacted fills will depend on the final site grades.

- Bedding sand can be generated at the site for use in utility trenches and under slabs.

- Class II Aggregate Base, for use as backfill in onsite and offsite street work and other paved areas, may be generated from onsite crushed materials. Prior to the use of these materials, they should be properly graded and pass the necessary hardness tests in accordance with the Greenbook requirements.

Our recommendations related to the geotechnical aspects of the development of the site are presented in the subsequent sections of this report.

4.2 SEISMIC CONSIDERATIONS

4.2.1 General

The site is located in a seismically active area typical of Southern California and is likely to be subjected to strong ground shaking due to earthquakes on nearby faults.

We assume that seismic design of the proposed development will be in accordance with 2016 California Building Code (CBC) criteria. For the 2016 CBC, Site Class C may be used. The seismic code values can be obtained directly from the tables in the building code using the above value and appropriate United States Geological Survey web site (earthquake.usgs.gov). The Project Structural Engineer should determine the seismic design method.

4.2.2 Strong Ground Motion Potential

Based on published information (geohazards.usgs.gov), the most significant fault in the proximity of the site is the Elsinore Fault, which is located about 3.4 miles to the southwest.

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on the USGS website (earthquake.usgs.gov), we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 0.70g for a magnitude 7.0 earthquake. This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from ASCE 7-10 (ASCE, 2010) and a site coefficient (F_{PGA}) based on site class. The predominant earthquake magnitude was determined using a 2-percent probability of exceedance in a 50-year period, or an average return period of 2,475 years. The structural design will need to incorporate measures to mitigate the effects of strong ground motion.

4.2.3 Potential for Ground Rupture

The site is not located within an Alquist-Priolo Special Studies Zone and there are no known faults crossing or projecting toward the site. Therefore, ground rupture due to faulting is considered unlikely at this site.

4.2.4 Liquefaction and Dynamic Settlement

Liquefaction is a phenomenon in which saturated cohesionless soils undergo a temporary loss of strength during severe ground shaking and acquire a degree of mobility sufficient to permit ground deformation. In extreme cases, the soil particles can become suspended in groundwater, resulting in the soil deposit becoming mobile and fluid-like. Liquefaction is generally considered to occur primarily in loose to medium dense deposits of saturated sandy soils. Thus, three conditions are required for liquefaction to occur: (1) a sandy soil of loose to medium density; (2) saturated conditions; and (3) rapid, large strain, cyclic loading, normally provided by earthquake motions.

The site is not located within an area mapped by the State of California as having a potential for soil liquefaction, in accordance with the Seismic Hazards Mapping Act as shown in the Murrieta Quadrangle (CGS, 2007). Groundwater was not encountered in our borings to a depth of 38 feet. Historical high groundwater data is not available in the vicinity of the site (CGS, 2007). Excluding the site from a potential liquefaction zone appears to have been based primarily on the depth to groundwater and the dense to very dense subsurface soils and the presence of weathered granitic bedrock.

Based on our findings, the potential for soil liquefaction to adversely affect the planned project is considered to be very low

4.2.5 Seismic Ground Subsidence

Seismic ground subsidence (not related to liquefaction induced settlements), occurs when loose, granular (sandy) soils above the groundwater are densified during strong earthquake shaking. Earthquake-induced seismic subsidence during a strong earthquake is not anticipated to adversely affect the planned project because of the dense to very dense nature of the near surface sandy soils and the presence of bedrock materials underlying those soils. Based on our analysis, potential earthquake-induced seismic subsidence during a strong earthquake is estimated to be less than ¼-inch.

4.3 EARTHWORK

The earthwork at the subject site is anticipated to consist of clearing and grubbing, significant cuts (up to 20 feet) to finished grade, overexcavation of disturbed soils under buildings, in utility corridors, and under settlement sensitive hardscape, subgrade preparation, and the placement and compaction of fills.

Significant ripping, oversized materials, and slow earthwork operations should be anticipated. Based on our explorations and observations of grading operations at nearby project sites, the upper weathered granite will break down under the ripping and compaction equipment to the consistency of a granular soil with some cemented larger particles. Localized clusters of less weathered rock are exposed at the ground surface, were encountered at depth in our explorations, and have been encountered in the adjacent on-going excavations. Some large boulders encountered on-site may be used in landscaping. The remaining oversized materials can be placed on-site in the compacted fill as recommended in a following section, if access is available.

4.3.1 Clearing and Grubbing

Prior to grading, the areas to be developed should be stripped of vegetation and cleared of debris. Buried obstructions, such as footings, utilities and tree roots, should be removed. Deleterious material generated during the clearing operation should be removed from the site. Although not anticipated in great extent because the site is undeveloped, and if acceptable to the regulatory agencies, on-site inert demolition debris, such as concrete and asphalt, may be crushed for re-use in engineered fills in accordance with criteria presented in the "Material for Fill" section of this report.

Although not encountered during our investigations, cesspools or septic systems encountered within building areas during grading should be removed in their entirety. The resulting excavation should be backfilled as recommended in the "Subgrade Preparation" and "Placement and Compaction of Fill" sections of this report. As an alternative, cesspools can be backfilled with lean sand-cement slurry. At the conclusion of the clearing operations, a representative of the personnel from GPI should observe and accept the site prior to further grading.

Two wells were observed on-site during our 2009 investigation. Details regarding the wells in regards to their current status and/or operation were not available. If the wells are still in place and are not planned to be used for production, they should be properly abandoned in accordance with City and County guidelines.

4.3.2 Excavations

Excavations at this site will include cut of existing materials to design grades, removals of loose topsoil and portions of the weathered bedrock, footing excavations, and trenching for proposed utility lines.

Prior to placing fills or the construction of the proposed buildings or other foundation supported improvements, the loose or disturbed upper soils within the proposed building areas should be removed and replaced as properly compacted fill. Although not encountered in our explorations, existing undocumented fills should also be removed below building pads or foundation supported improvements.

Based on our explorations, the depth of topsoil and low density highly weathered bedrock ranges from about 2 to 7 feet below existing site grades, with an average depth of about 4 feet below existing site grades. Where not removed by planned cuts, these materials should be removed and replaced with properly compacted fill.

The recommended earthwork will depend on the area of the site (bedrock or topsoil) as well as the planned improvement and cut/fill conditions. The actual depths of removal should be determined in the field during grading by a representative of GPI. For planning purposes, we recommend the following overexcavation depths:

- At least 2 feet below the existing ground surface in areas to receive fill to remove and recompact the near-surface disturbed natural soils.
- At least 5 feet below the planned finish pad grade or 2 feet below the base of foundations, whichever is greater, for buildings and other major foundation supported improvements in fill areas.
- At least 3 feet beneath the base of footings for buildings spanning across bedrock and compacted fill (transition pad).
- At least 2 feet below the base of foundations for minor foundation supported improvements such as short retaining walls or screen walls, unless the foundations will be established in the undisturbed bedrock.
- At least 2 feet below the existing grade within planned drives and parking areas (may be replaced with scarification, moisture-conditioning, and compaction in areas of cut greater than 2 feet).

The undisturbed bedrock is suitable for direct support of building foundations. Conventional excavating equipment (e.g. backhoe) may have difficulty excavating footings or pad utility trenches in the undisturbed bedrock. As such, we recommend the removals for building pads established entirely in bedrock materials extend to a depth of at least 3 feet below the finished pad grade. Deeper removals may be beneficial if deep footings or pad utilities are planned. Also, overexcavation and recompaction for site utility corridors may be beneficial in areas of deep cut where less weathered bedrock is anticipated so that conventional installation of utilities is possible.

The Project Surveyor should accurately stake the corners of the areas to be overexcavated in the field. The base of the excavations should extend laterally at least 10 feet beyond the outside edge of the foundations or a minimum distance equal to the depth of overexcavation/compaction below finish grade (i.e., a 1:1 projection below the top edge of footings), whichever is greater. This includes the footprint of the building, hardscape along the building, and other foundation supported improvements, such as site walls, canopies, equipment pads, and enclosures.

Where not removed by the aforementioned excavations, existing utility trench backfill within building areas should be removed and replaced as properly compacted fill. This is especially important for deeper fills such as existing sewers and storm drains. For planning purposes, removals over the utilities should extend to within 1-foot of the top of the pipe. For utilities that are 3 feet or shallower, the removal should extend laterally 1-foot beyond both sides of the pipe. For deeper utilities, the removals should include a zone defined by a 1:1 projection upward (and away from the pipe) from each side of the pipe. The actual limits of removal will be confirmed in the field. We recommend that known utilities be shown on the grading plan. It is our understanding that existing utilities under the vacated portion of Antelope Road will be left-in-place and should not be disturbed.

Temporary construction excavations may be made vertically without shoring to a depth of 5 feet below adjacent grade. For deeper cuts up to 20 feet, the slopes should be properly shored or sloped back at least 1:1 or flatter in the moderately to highly weathered bedrock and ¾:1 (horizontal:vertical) in the less weathered granitic bedrock. The allowable temporary slope inclination is measured from the toe to top of slope. As such, if a vertical cut is incorporated into the slope, the remaining portion must be flattened to achieve the recommended inclination. Some raveling should be anticipated at the slope inclinations recommended if sandy deposits are exposed. If raveling cannot be tolerated, flatter slope inclinations should be considered. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing.

In areas where removals are performed adjacent to property lines, existing streets, or other improvements where temporary slopes are not feasible, temporary shoring or “ABC” slot cuts should be utilized. The slots should be no wider than 8 feet and no deeper than 8 feet, and should be backfilled <u>immediately</u> to finish grade prior to excavation of the adjacent two slots. If localized dry, clean sand deposits are encountered, narrower slots may be required. We should review the plans for excavation adjacent to property lines and existing improvements when they are developed.

Surcharge loads should not be permitted within a horizontal distance equal to the height of cut from the top of the excavation or 5 feet from the top of the slopes, whichever is greater, unless the cut is properly shored. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site facilities should be properly shored to maintain support of adjacent elements. Excavations and shoring systems should meet the minimum requirements given in the most current State of California Occupational Safety and Health Standards. If cuts greater than 20 feet are required, we should be contacted to provide an allowable slope inclination.

In general, the excavations will extend into both loose topsoil/weathered bedrock and very dense bedrock material. Conventional soil excavation equipment such as backhoes, loaders, scrapers, or dozers is being used for the on-going excavations on the adjacent site, but may have difficulty excavating into the dense bedrock. Relatively slow earthwork operations should be anticipated in most of the deeper cuts. Some difficulty will be encountered in ripping the dense granitic materials and drilling and shooting or blasting may aid in increasing earthwork production in deep cut areas.

4.3.3 Subgrade Preparation

Prior to placing fills or construction of the proposed structures, the subgrade soils should be scarified to a depth of 8 inches, moisture-conditioned, and compacted to at least 90 percent of the maximum dry density in accordance with ASTM D 1557. Subgrade processing should be omitted where the exposed soils are well over the optimum moisture content or if moist, undisturbed bedrock is exposed.

4.3.4 Material for Fill

The on-site soils are, in general, suitable for use as compacted fill within the building pad and pavement areas. Particles larger than 12 inches in diameter should be placed at least 2 feet below the finished subgrade in parking and landscape area fills. The location selected for the rock placement should avoid future underground work, such as utility installation. The larger rock should be placed to avoid “nesting.” Cobbles less than 12 inches in diameter may be placed in building pad fills, with the exception of the upper 3 to 4 feet (at least 1-foot below the planned footing depths), which should consist of cleaner sandy soils (particles sizes of less than 3-inch in diameter and not more than 25 percent of the material larger than 1-inch in diameter) to facilitate footing excavations.

Although not encountered in our explorations, clayey soils should not be placed within 2 feet of the building pads or within retaining wall backfill. Soils used as retaining wall backfill should be predominately granular (contain no more than 40 percent fines - portion passing No. 200 sieve) and non-expansive (E.I. less than 20). Such materials are anticipated to be available on-site.

If required, imported fill material should be predominately granular (contain no more than 40 percent fines - portion passing No. 200 sieve) and non-expansive (E.I. less than 20). The import should also exhibit an R-value of at least 50 if used in proposed paved areas. GPI should be provided with a sample (at least 50 pounds) and notified of the location of soils proposed for import at least 72 hours in prior to importing. Each proposed import source should be sampled, tested and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by GPI may be rejected if not suitable. Imported materials larger than 6 inches in diameter should not be used in the compacted fills.

Although not anticipated in significant quantities and if acceptable to the City and the owner, the on-site inert debris, such as concrete, asphalt, or brick, can be used in planned fills if it is adequately crushed and mixed with the on-site soils. For general fills, the material should be crushed to the consistency of aggregate base (3-inch minus) and mixed with at least three parts soil. Such material should not be placed in proposed landscape areas or beneath planned buildings.

4.3.5 Placement and Compaction of Fills

Fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to at least 90 percent of the maximum dry density in accordance with ASTM D 1557. Fill soils placed within 12 inches of the finished pavement subgrade or deeper than 5 feet below finished grade should be compacted to at least 95 percent. The optimum lift thickness will depend on the compaction equipment used and can best be determined in the field. The following uncompacted lift thickness can be used as preliminary guidelines.

Plate compactors	4-6 inches
Small vibratory or static rollers (5-ton)	6-8 inches
Scrapers, heavy loaders, and large vibratory rollers	8-12 inches

The maximum lift thickness should not be greater than 12 inches and each lift should be thoroughly compacted and accepted prior to subsequent lifts. Earthwork contractors should consider additional equipment utilizing a sheepsfoot compactor to readily breakdown oversized bedrock material from the cut during compactions.

The moisture contents of the on-site soils are widely variable (dry to moist), such that mixing and moisture conditioning during grading will be required. The earthwork contractors should allow for the variable conditions in their bids. The moisture content of the on-site materials should be between 1 to 3 percent over the optimum moisture conditions at the time of compaction. Care should be taken by the Contractor to maintain the moisture content of the soils exposed at finished grade in slab, hardscape, and pavement areas until the soils are covered. If the soils are allowed to dry out, additional processing will be required.

During backfill of excavations, the fill should be properly benched into the construction slopes as it is placed in lifts.

If the boulders encountered in the on-site cuts are to be placed in the parking lot or landscape area fills, the materials should be disposed of in windrows or pits to reduce the potential for voids. Boulders up to 36 inches in diameter should be disposed of in windrows. The windrows should be constructed in a trench condition (engineered fill brought up at a higher elevation on both sides) to allow for the voids between the boulders to be filled with sands and flooded or jetted. The particles should be placed far enough apart to prevent nesting and to permit placement and flooding/jetting of sand around each particle to avoid permanent voids. The sands used for the jetting should consist of the on-site sands or imported materials with no more than 35 percent by weight retained on the No. 200 sieve and no particles larger than 1-inch in diameter. Upon jetting, the windrows can be covered with one lift of soil before mechanically compacting with a vibratory compactor (minimum 40,000 pounds dynamic force) or large static compaction equipment (Caterpillar 824 or equivalent). A horizontal spacing of at least 20 feet should be provided between windrows. Vertically, the bottom of a new windrow should be spaced at least 5 feet from the top of the old windrow and should be offset such that the windrows are not constructed directly over the underlying windrow. The windrows should be constructed parallel with any existing permanent fill slopes and should be maintained at a horizontal distance of at least 15 feet from the face of slopes.

Boulders larger than 36 inches in diameter should be broken to achieve smaller sizes (less than 36 inches). A less desirable but acceptable alternative would be to dispose of larger boulders in pits. The pits should be constructed similarly to the windrows discussed above but only one boulder should be placed in each pit. The pit should be backfilled with sand to a distance of 36 inches from the base of the pit and flooded or jetted. Above 36 inches, the backfill should be mechanically compacted in lifts.

The locations of the windrows or pits should be coordinated by the contractor to avoid conflicts with other planned improvements, such as utilities or parking lot light standards. Alternative methods for disposal of oversized materials proposed by the contractor can be evaluated.

4.3.6 Shrinkage and Subsidence

Shrinkage is the loss of soil volume caused by compaction of fills to a higher density than before grading. Subsidence is the settlement of in-place subgrade soils caused by loads generated by large earthmoving equipment. For earthwork volume estimating purposes, an average shrinkage value of 5 to 10 percent for the surficial soils (generally in the upper 2 to 5 feet) and a bulking value of 0 to 5 percent in the deeper materials (cut) may be assumed. A subsidence of 0.1 feet may be assumed for the surficial natural soils in planned pavement areas where the surficial soils will be scarified and compacted in place. These values are estimates only and exclude losses due to removal of vegetation or debris. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be determined during grading.

4.3.7 Trench/Wall Backfill

Utility trench and wall backfill consisting of the on-site material or imported sand should be mechanically compacted in lifts. Soils placed as retaining wall backfill should be granular and relatively non-expansive, as previously defined. Lift thickness should not exceed those values given in the "Placement and Compaction of Fills" section of this report. Moisture conditioning of the on-site soils will be required prior to re-use as backfill. Jetting or flooding of backfill materials should not be permitted with the exception of disposing oversized materials. A representative of GPI should observe and test trench and wall backfills as they are placed.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, sand-cement slurry may be substituted for compacted backfill. The slurry should contain 1½ sacks of cement per cubic yard and have a maximum slump of 5 inches. When set, such a mix typically has the consistency of compacted soil.

4.3.8 Observation and Testing

A representative of GPI should observe excavations, subgrade preparation, and fill placement activities. Sufficient in-place field density tests should be performed during fill placement and in-place compaction to evaluate the overall compaction of the soils. Soils that do not meet minimum compaction requirements should be reworked and retested prior to placement of additional fill.

4.4 FOUNDATIONS

4.4.1 General

The proposed structures may be supported on conventional isolated and/or continuous shallow footings, provided the subsurface soils are prepared in accordance with the recommendations given in this report. Footings should be supported on properly compacted fill.

The onsite sandy soils will dry out relatively quickly if left exposed. Immediately prior to placing reinforcing steel and concrete, we recommend the base of the footings be wetted and surficially compacted using a hand compactor.

4.4.2 Allowable Bearing Pressures

Based on the shear strength and elastic settlement characteristics of the recompacted on-site soils, a static allowable bearing pressure of 4,000 pounds per square foot (psf) may be used for both continuous footings and isolated column footings. These bearing pressures are for dead-plus-live-loads, and may be increased one-third for short-term, transient, wind and seismic loading. The actual bearing pressure used may be less than the value presented above and can be based on economics and structural loads to determine the minimum width for footings as discussed below. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values.

4.4.3 Minimum Footing Width and Embedment

The following minimum footing widths and embedments are recommended for the corresponding allowable bearing pressure.

STATIC BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (inches)	MINIMUM FOOTING* EMBEDMENT (inches)
4,000	48	24
3,000	24	24
2,500	18	24
2,000	18	18

* Refers to minimum depth below lowest adjacent grade.

A minimum footing width of 18 inches should be used even if the actual bearing pressure is less than 2,000 psf.

4.4.4 Estimated Settlements

Maximum total static settlement of the more heavily loaded column footings (assumed to 300 kips) is expected to be less than 1-inch. Maximum total static settlement of the moderately loaded column footings (assumed to be 50 to 100 kips) is expected to be less than ½-inch. Maximum differential settlements between similarly loaded adjacent footings are expected to be less than ½-inch and ¼-inch for heavily loaded and moderately loaded column footings, respectively.

The above estimates are based on the assumption that the recommended earthwork will be performed and the footings will be sized in accordance with our recommendations.

4.4.5 Lateral Load Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of footings and underlying soils and by passive soil pressures acting against the embedded sides of the footings. For frictional resistance, a coefficient of friction of 0.35 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weighing of 300 pounds per cubic foot, may be used, provided the footings are poured tight against compacted fill soils. These values may be used in combination without reduction.

4.4.6 Foundation Concrete

Laboratory testing by Schiff and Associates (currently HDR, Appendix B) indicates a soluble sulfate content of 8 mg/kg (less than 0.01 percent by weight). For the 2016 CBC, Foundation concrete should conform to the requirements outlined in ACI 318, Section 4.3, for negligible levels of soluble sulfate exposure from the on-site soil.

4.4.7 Foundation Observation

Prior to placement of concrete and steel, a representative of GPI should observe and approve foundation excavation.

4.4.8 Light Standard Bases

The design of light standard bases is generally governed by lateral force considerations. For design by the simplified pole formula presented in Section 1807A.3.2.1 of the 2016 California Building Code, a unit passive resistance of 300 pounds per square foot per foot (to a maximum of 3,000 pounds per square foot) may be used for the piles with level ground in lieu of the presumptive lateral bearing values presented in Table 1806A.2. As stated in the code, a passive resistance of 600 pounds per square foot per foot (to a maximum of 6,000 pounds per square foot) may be used for isolated piles as determined by the Project Structural Engineer. This value incorporates the allowable increase stated in Section 1806A.3.4 of the code for single poles that can tolerate ½-inch of deflection under short-term loads.

A pile designed for adequate embedment to resist the anticipated lateral loads should have adequate axial capacity to support the anticipated vertical loads. The net allowable vertical compressive capacity can be conservatively calculated based on a unit side friction of 500 pounds per square foot, neglecting any end bearing contribution. We recommend that the upper 1-foot of the subgrade soils be ignored in determining the required depth of embedment to allow for future surface disturbance adjacent to the pile.

4.5 BUILDING FLOOR SLABS

Slab-on-grade floors should be supported on granular (sandy), non-expansive, compacted soils as discussed in the “Material for Fill” section. The on-site sands and silty sands encountered are anticipated to be suitable for direct support of the slabs.

A vapor/moisture retarder should be placed under slabs that are to be covered with moisture-sensitive floor coverings (parquet, vinyl, tile, etc.). Polyolefin in 15-mil thickness should be covered by a layer of clean sand (less than 5 percent by weight passing the No. 200 sieve) having a minimum thickness of 2 inches. Based on our explorations and laboratory testing, the soils at the site are not suitable for this purpose. The function of the sand layer is to protect the vapor retarder during construction and to aid in the uniform curing of the concrete. This layer should be nominally compacted using light equipment. The sand placed over the vapor retarder should only be slightly moist. If the sand gets wet (for example because of rainfall or excessive moistening) it must be allowed to dry prior to placing concrete. Care should be taken to avoid infiltration of water into the sand layer after placement of the concrete slab, such as at slab cut-outs and other exposures. Placement of the sand over the retarder is a construction related issue. If the Contractor desires to omit the sand layer and takes the necessary steps to protect the vapor retarder and properly cure the concrete slab (prevent curling), we take no exception.

It should be noted that the material used as a vapor retarder is only one of several factors affecting the prevention of moisture accumulation under floor coverings. Other factors include maintaining a low water-cement ratio for the concrete used for the floor slab, effective sealing of joints and edges (particularly at pipe penetrations) as well as excess moisture in the concrete. The manufacturer of the floor coverings should be consulted for establishing acceptable criteria for the condition of the floor surface prior to placing moisture-sensitive floor coverings.

For the elastic design of slabs supporting sustained concentrated loads, a modulus of subgrade reaction (k) of 200 pounds per cubic inch (pounds per square inch per inch of deflection) may be used. This value is for a 1-foot by 1-foot square loaded area and should be adjusted by the structural designer for the area of the proposed building slab using appropriate elastic theory.

For lateral resistance design, a coefficient of friction value of 0.35 between aggregate base or select fill and concrete may be used. For a slab on a visqueen moisture barrier, a coefficient of 0.1 should be used.

4.6 LATERAL EARTH PRESSURES

Based on information available to us at the time this report was prepared, no major retaining walls are planned for the project. The following recommendations are provided for walls less than 8 feet high at the project site. We recommend that retaining walls be backfilled with on-site or imported non-expansive granular soils.

Active earth pressures can be used for designing walls that can yield at least ½-inch laterally in 10 feet of wall height under the imposed loads. For level backfill comprised of on-site or imported granular soils (non-expansive with no more than 40 percent passing No. 200 sieve), the magnitude of active pressures are equivalent to the pressures imposed by a fluid weighing 35 pounds per cubic foot (pcf). This pressure may also be used for the design of temporary excavation support.

At-rest pressures should be used for restrained walls that remain rigid enough to be essentially non-yielding. At-rest pressures imposed by a fluid weighing 52 pounds per cubic foot should be used for drained granular backfill.

Walls subject to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge pressure for unrestrained and restrained walls, respectively.

The wall backfill should be well-drained to relieve possible hydrostatic pressure or designed to withstand these pressures. A drain consisting of perforated pipe and gravel wrapped in filter fabric should be used. One cubic foot of rock should be used for each lineal foot of pipe. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top. We prefer pipe and gravel drains to weep holes to avoid potential for constant flow of surface water in front of the wall.

The Structural Engineer should specify the use of select, granular wall backfill on the plans. Wall footings should be designed as discussed in the "Foundations" section.

4.7 CORROSIVITY

Based on results of laboratory testing (Appendix B), the on-site soils are moderately corrosive to ferrous metals. GPI does not practice corrosion engineering. Should the use of buried pipe be proposed, a corrosion engineer such as HDR should be contacted to provide recommendations to protect these elements from corrosion.

4.8 DRAINAGE

Positive surface gradients should be provided adjacent to structures so as to direct surface water run-off and roof drainage away from foundations and slabs toward suitable discharge facilities. The introduction of water into uncompacted natural soils can result in subsidence due to densification of the collapsible soils. Long-term ponding of surface water should not be allowed on pavements or adjacent to buildings. We recommend that landscape planters be avoided immediately adjacent to the building. If planters are required, they should be provided with surface drains and planted with drought tolerant plants to reduce the potential for the infiltration of surface water beneath the building foundations and floor slabs.

4.9 EXTERIOR CONCRETE AND MASONRY FLATWORK

Exterior concrete and masonry flatwork should be supported on granular, non-expansive (EI not greater than 20) compacted fill. Prior to placement of concrete, the subgrade should be prepared as recommended in "Subgrade Preparation" section. It should be noted that the sidewalk in front of the buildings should be considered as part of the building pad for earthwork purposes.

4.10 PAVED AREAS

R-value test results indicate that the subgrade soils have an R-value of 63. Preliminary pavement design has been based on an R-value of 50. The California Division of Highways Design Method was used for design of the recommended preliminary asphalt concrete pavement sections. These recommendations are based on the assumption that the pavement subgrades will consist of existing near surface soils. Final pavement design should be based on R-value testing performed near the conclusion of rough grading. The following pavement sections are recommended for planning purposes only.

PAVEMENT SECTIONS

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		ASPHALT/PORTLAND CONCRETE	AGGREGATE BASE COURSE
Asphalt Concrete			
Automobile Parking	4.0	3.0	4.0
Automobile Drives	5.0	3.0	5.0
Truck/Bus Drives	6.0	3.5	5.0
Portland Cement Concrete			
Automobile Parking	4.0	6.0	4.0
Automobile Drives	5.0	6.0	4.0
Truck/Bus Drives	6.0	6.5	4.0

The pavement subgrade underlying the aggregate base should be properly prepared and compacted in accordance with the recommendations outlined under "Subgrade Preparation" or “Placement and Compaction of Fills”.

The pavement base course should be compacted to at least 95 percent of maximum density (ASTM D 1557). Aggregate base should conform to the requirements of Section 26 of the California Department of Transportation Standard Specifications for Class II aggregate base (three-quarter inch maximum) or Section 200-2 of the Standard Specifications for Public Works Construction (Green Book) for untreated base materials (except processed miscellaneous base).

The concrete used for paving should have a modulus of rupture of at least 550 psi (equivalent to an approximate compressive strength of 3,700 psi at the time the pavement is subjected to traffic). The upper 12 inches of the subgrade soils should be compacted to at least 95 percent of the maximum dry density (ASTM D 1557).

The above recommendations are based on the assumption that the base course and compacted subgrade will be properly drained. The design of paved areas should incorporate measures to prevent moisture build-up within the base course, which can otherwise lead to premature pavement failure. For example, curbing adjacent to landscaped areas should be deep enough to act as a barrier to infiltration of irrigation water into the adjacent base course.

4.11 SLOPES

New slopes or modifications to existing slopes up to 15 feet in height may be constructed at inclinations of 2:1 (horizontal:vertical) or flatter. For higher slopes, GPI should be provided with the details to evaluate the recommended inclination.

Achieving good compaction at the face of the slope is important in reducing the potential for erosion. Modifications to the existing slopes should be overbuilt by at least 3 feet during rough grading and trimmed back to a hard and unyielding surface. The fill should be properly benched into the existing slopes as it is placed in lifts. Slope rolling to achieve a finished compacted surface should not be used if surficial erosion is not tolerable.

Slopes should be seeded or planted as soon as possible to reduce the erosion potential. Other protective measures, such as installing drains and grading the top of the slope to drain should be performed. We recommend that the condition of the existing slopes on-site be periodically reviewed to confirm that the erosion protection measures are in good repair.

Setbacks of structures from the top and toe of the existing and planned slopes should be maintained in accordance with the County of Riverside standards.

4.12 SUBSURFACE INFILTRATION

To provide a preliminary evaluation of the infiltration characteristics for a proposed stormwater bio-infiltration system, we performed two near-surface field infiltration tests. Details of the infiltration system have not been established, but based on the request of the Project Team, we have performed infiltration tests along Clinton Keith Road at the existing alignment of Antelope Road and along Antelope Road between our borings B-101 and B-102. Both tests were performed at approximate depths of 1-foot below the current ground surface. The tests were performed in general accordance with the County of Riverside methods (County of Riverside, 2011).

The tests were performed in shallow borings drilled with an 8-inch hollow stem auger. The test wells were filled with approximately 1-foot of water at the initiation of the test. During the initial soak, the wells exhibited two consecutive measurements of greater than 6 inches of water surface drop in less than 25 minutes, meeting the criteria for Sandy Soils as detailed in the County guidelines. As such, we performed the subsequent infiltration tests over 10-minute intervals. The tests were repeated until 8 readings were obtained. The pre-adjusted infiltration rate was calculated as the depth of water drained over the associated time interval.

Once the pre-adjusted infiltration rate was determined, we corrected the results using the Porchet Method as outlined by the County. We determined the following stabilized and corrected rates:

- P-101 – 2.2 inches per hour
- P-102 – 2.1 inches per hour

Significantly lower infiltration rates, approaching zero, are anticipated in the less weathered bedrock materials. Additional testing should be performed when finished grades are established. Additional factors of safety in computing the design infiltration rate of the proposed infiltration BMP should be determined by the project Civil Engineer. The results of the infiltration tests are presented in Table 1, Borehole Infiltration Test Results.

It should be noted that the volume of water applied during our test was relatively low compared to the planned system. Due to the dense nature of the near surface and subsurface soils and the presence of shallow weathered bedrock, infiltration of large volumes of water into the near surface soils may result in limited percolation rates and the potential for long-term mounding or perched conditions. The Civil Engineer should evaluate the feasibility of subsurface infiltration using the rates provided.

The testing was performed with clean, clear water, and the results do not include effects of sediments, fines, dissolved solids, or other debris, as these will significantly reduce the percolation rates of the subsurface soils. The infiltration system should include processes to clean the inflow of sediments or other deleterious materials to reduce the potential for clogging and reduced infiltration rates.

4.13 GEOTECHNICAL OBSERVATION AND TESTING

We recommend that a representative of GPI observe earthwork during construction to confirm that the recommendations provided in our report are applicable during construction. The earthwork activities include grading, compaction of fills, subgrade preparation, backfill of utility trenches and retaining walls, pavement construction and foundation excavations. If conditions are different than expected, we should be afforded the opportunity to provide an alternative recommendation based on the actual conditions encountered.

5.0 LIMITATIONS

The report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for use by CK 17 LP, Felicita Inc., and their consultants in designing the proposed development. If relying on this report, those entities besides CK 17 LP are bound by the terms and conditions in our contract with CK 17 LP (a copy can be provided upon request). The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on any project other than the currently proposed development, as it may not contain sufficient or appropriate information for such uses. If this report or portions of this report are provided to contractors or included in specifications, it should be understood that they are provided for information only. This report cannot be utilized by another entity without the express written permission of GPI. This report is an instrument of our services and remains the property of GPI.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations. While we cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data obtained in the field and laboratory are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

Furthermore, our recommendations were developed with the assumption that a proper level of field observation and construction review will be provided during grading, excavation, and foundation construction by GPI. If field conditions during construction appear to be different than is indicated in this report, we should be notified immediately so that we may assess the impact of such conditions on our recommendations. If construction phase services are performed by others they must accept full responsibility (as Project Geotechnical Engineer) for the geotechnical aspects of the project, including this report.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable Geotechnical Engineers practicing in this area. No other representation, either expressed or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals Inc.

Dylan J. Boyle, P.E.
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
DYLAN J. BOYLE
No. 80638
CIVIL
STATE OF CALIFORNIA

Paul R. Schade, G.E.
Principal

REGISTERED PROFESSIONAL ENGINEER
PAUL R. SCHADE
No. 2371
Exp. 9/30/18
GEOTECHNICAL
STATE OF CALIFORNIA

DJB/PRS:sph

REFERENCES

American Society of Civil Engineers (2010), "Minimum Design Loads for Buildings and Other Structures," ASCE/SEI 7-10.

California Department of Conservation, Division of Mines and Geology (2007), "Earthquake Zones of Required Investigation, Murrieta Quadrangle", December 5, 2007.

California Department of Conservation, Division of Mines and Geology (1997), "Special Publications 117: Guidelines for Evaluating and Mitigating Seismic Hazards in California."

California Department of Conservation, Division of Mines and Geology (2007), "Seismic Hazard Zone Report for the Murrieta 7.5-Minute Quadrangle, Riverside County, California, Seismic Hazard Zone Report 115".

County of Riverside, "Infiltration Testing, Appendix A, Low Impact Development BMP Design Handbook," revised September 2011.

Geotechnical Professionals Inc. (2009), "Geotechnical Investigation, Proposed Retail Center Project, Clinton Keith and Antelope Roads, Murrieta, California," GPI Project No. 2261.I, dated March 5, 2009 (for CK 17 LLC)

Geotechnical Professionals Inc. (2009), "Limited Geotechnical Evaluation, New Antelope Road, Clinton Keith and Antelope Roads, Murrieta, California," GPI Project No. 2261.I, dated February 6, 2009 (for CK 17 LLC)

T.H.E Soils Co., Inc. (2008), "Preliminary Geotechnical Investigation, Proposed Realignment of Antelope Road & Commercial/Retail Development, 38.18-Acres, APN: 392-290-025, -026, -028, -029, -038 & 392-270-018, Northeast Corner of Clinton Keith Road and Interstate I-215, City of Murrieta, Riverside County, California," February 20, 2008, Work Order No. 1003701.00 (for Unique Lighting Systems)

GeoSoils, Inc. (2005), "Rock Hardness/Rippability and Slope Stability Study, Proposed Borrow Site, Parcels 1 through 4 of Parcel Map 16499, City of Murrieta, Riverside County, California," October 3, 2005, Work Order No. 4919-A-SC (for Markham Development Management Group, Inc.)

EcoTech Remediation, Inc. (2004), "Preliminary Geotechnical Investigation, Assessors Parcel Numbers: 359-500-011, 012, 013, 014, Antelope Road, City of Murrieta, Riverside County, California," July 26, 2004, Project No. 969-01 (for Bill Rawlings)

NETR Online, Historical Aerials, www.historicalaerials.com

United States Geological Survey (2014), U.S. Seismic Design Maps Website, http://earthquake.usgs.gov/designmaps/us/application.php/

United States Geological Survey (2014), 2008 National Seismic Hazard Maps, Source Parameters, http://geohazards.usgs.gov/

BASE MAP REPRODUCED FROM MICROSOFT STREETS AND TRIPS © 2008

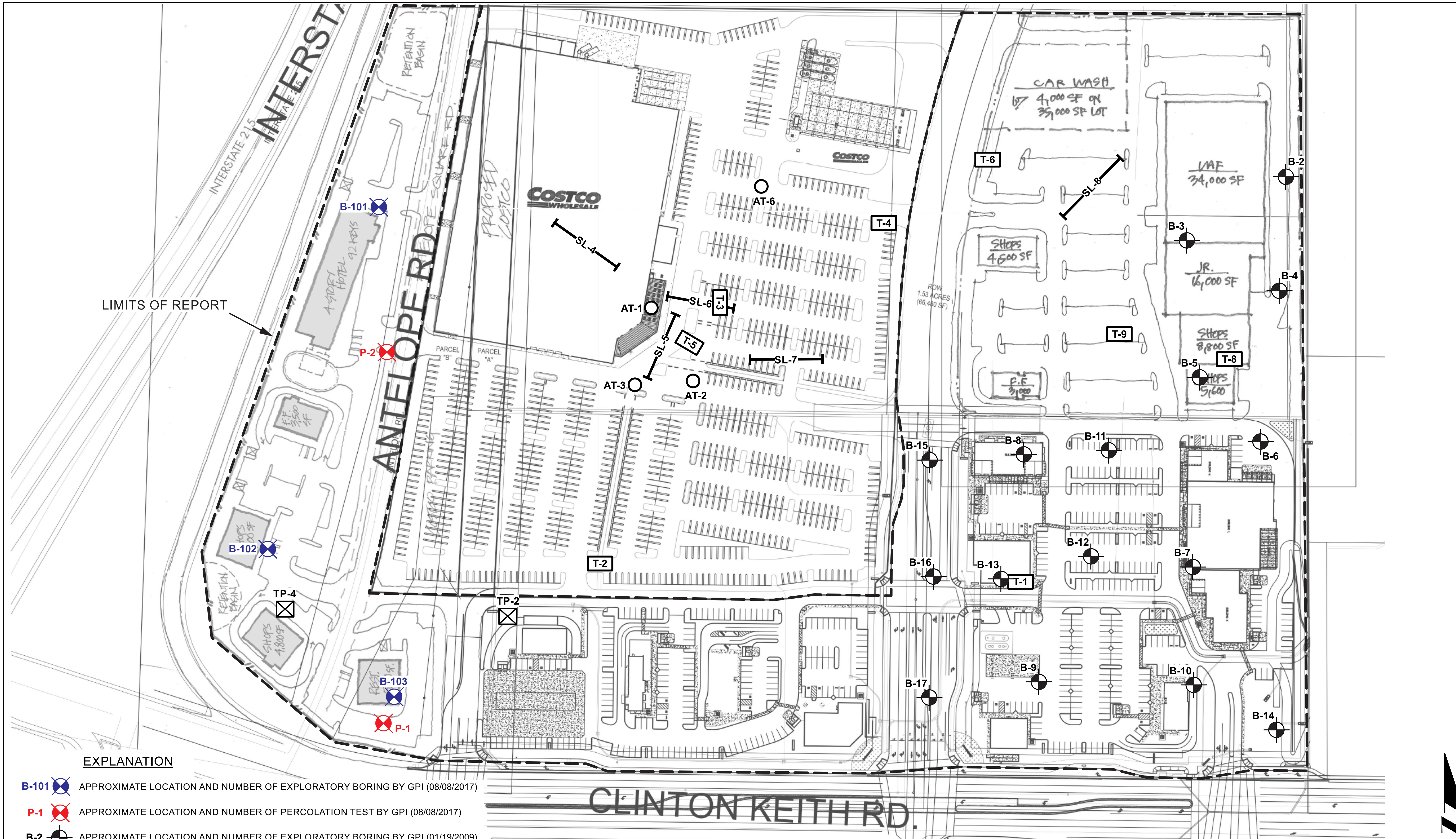
GPI GEOTECHNICAL PROFESSIONALS, INC.

VINEYARD / VAL VISTA CENTER

GPI PROJECT NO.: 2833.I | SCALE: 1" = 2000'

SITE LOCATION MAP

FIGURE 1



EXPLANATION

B-101 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING BY GPI (08/08/2017)

P-1 APPROXIMATE LOCATION AND NUMBER OF PERCOLATION TEST BY GPI (08/08/2017)

B-2 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING BY GPI (01/19/2009)

AT-3 APPROXIMATE LOCATION AND NUMBER OF AIR TRACK BORINGS BY GEOTEK (2004)

T-1 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY TRENCHES BY THE SOILS CO. (3/15/2007)

TP-4 APPROXIMATE LOCATION AND NUMBER OF TEST PIT BY ECOTECH (7/26/2004)

SL-5 APPROXIMATE LOCATION AND NUMBER OF SEISMIC LINES BY GEOSOILS, INC (10/3/05)

0 150 300 FEET

BASE PLAN REPRODUCED FROM CONCEPTUAL PLAN PROVIDED BY CK-17, LP: (UNDATED)

GEOTECHNICAL PROFESSIONALS, INC.

VINEYARD / VAL VISTA CENTER

GPI PROPOSAL NO.: 2833.I

SCALE: 1" = 150'

SITE PLAN

FIGURE 2

TABLE 1

BOREHOLE INFILTRATION TEST RESULTS (corrected with Porchet Method)

Riverside County Method-TGD, 2011

Project No. 2833.I Project Name: Vineyard CK17 Date: 8/18/2017

Test Date 8/8/2017 NOTE: Slowest rate from percolation testing used to calculate infiltration rate

Test Well	Test Duration (min) Δt	Water Depth Initial (ft) D_o	Water Depth Final (ft) D_f	Total Depth of Test Hole (ft) D_T	Hole Diameter (inches)	Initial Water Height (ft) H_o	Final Water Height (ft) H_f	Change in Height of Water (ft) $\Delta H=\Delta D$	Average Height of Water (ft) H_{avg}	Infiltration Rate (in/hr) I_t
P-101	10	0.00	0.28	1.13	8	1.13	0.85	0.28	0.99	2.9
P-101	10	0.00	0.26	1.13	8	1.13	0.87	0.26	1.00	2.7
P-101	10	0.00	0.25	1.13	8	1.13	0.88	0.25	1.00	2.6
P-101	10	0.00	0.24	1.13	8	1.13	0.89	0.24	1.01	2.5
P-101	10	0.00	0.24	1.13	8	1.13	0.89	0.24	1.01	2.5
P-101	10	0.00	0.23	1.13	8	1.13	0.90	0.23	1.01	2.3
P-101	10	0.00	0.22	1.13	8	1.13	0.91	0.22	1.02	2.2
P-101	10	0.00	0.22	1.13	8	1.13	0.91	0.22	1.02	2.2

Test Date 8/8/2017 NOTE: Slowest rate from percolation testing used to calculate infiltration rate

Test Well	Test Duration (min) Δt	Water Depth Initial (ft) D_o	Water Depth Final (ft) D_f	Total Depth of Test Hole (ft) D_T	Hole Diameter (inches)	Initial Water Height (ft) H_o	Final Water Height (ft) H_f	Change in Height of Water (ft) $\Delta H=\Delta D$	Average Height of Water (ft) H_{avg}	Infiltration Rate (in/hr) I_t
P-102	10	0.08	0.33	1.08	8	1.00	0.75	0.25	0.88	2.9
P-102	10	0.08	0.32	1.08	8	1.00	0.76	0.24	0.88	2.8
P-102	10	0.08	0.32	1.08	8	1.00	0.76	0.24	0.88	2.8
P-102	10	0.08	0.30	1.08	8	1.00	0.78	0.22	0.89	2.5
P-102	10	0.08	0.30	1.08	8	1.00	0.78	0.22	0.89	2.5
P-102	10	0.08	0.29	1.08	8	1.00	0.79	0.21	0.90	2.4
P-102	10	0.08	0.27	1.08	8	1.00	0.81	0.19	0.91	2.1
P-102	10	0.08	0.27	1.08	8	1.00	0.81	0.19	0.91	2.1

APPENDIX A

APPENDIX A

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling 20 exploratory borings over two separate field investigations at the subject site. A total of 17 borings (B-1 through B-17), located on the eastern half of the site, were performed in January 2009. Three borings (B-101 through B-103) were performed as part of the current investigation. The borings were advanced to depths ranging from 11 to 39 feet below the existing ground surface (at the time of drilling). Several explorations were terminated prior to the planned depths after meeting practical refusal on dense native soils or possible due to the presence of cobbles or boulders. The locations of the explorations from both investigations are shown on the Site Plan, Figure 2.

The exploratory boring was drilled using truck-mounted hollow-stem auger drill equipment. Relatively undisturbed samples were obtained using a brass ring lined sampler (ASTM D 3550). The brass rings have an inside diameter of 2.42 inches. The ring samples were driven into the soil by a 140-pound hammer dropping 30 inches. The number of blows needed to drive the sampler into the soil was recorded as the penetration resistance.

At selected locations, disturbed samples were obtained using a split-spoon sampler by means of the Standard Penetration Test (SPT, ASTM D 6066). The spoon sampler was driven into the soil by a 140-pound hammer dropping 30 inches, employing the “free-fall” hammer described above. After an initial seating drive of 6 inches, the number of blows needed to drive the sampler into the soil a depth of 12 inches was recorded as the penetration resistance. These values are the raw uncorrected blowcounts.

The field explorations for the investigation were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The site contains topsoil/weathered bedrock over bedrock materials. However, the materials encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings are presented in Figures A-1 to A-20 in this appendix.

The boring locations were laid out in the field based on the available site plans, topographic information, and by measuring from existing site features. Ground surface elevations for the explorations from our 2009 investigation were obtained from a topographic map. Elevations for explorations from the current investigation were estimated from internet sources. In both cases, the ground surface elevations should be considered approximate.

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, moist	
10.3	122	25	D		@ 2 ft, medium dense	
5.8		50	S	5	Moderately Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, moist, very dense	1515
3.8	110	50/4"	D		@ 7 ft, dry	
2.3		50/5"	S	10		1510
17.8		50/3"	D	15	@ 15 ft, light brown, very moist, increase of fines and trace of gravel	1505
2.8		50/5"	S	20	@ 20 ft, brown, dry	1500
4.6		50/5"	S	25		1495
					Total Depth 26 ft	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
1-19-09
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2833.I
VINEYARD/VAL VISTA

LOG OF BORING NO. B-1

FIGURE A-1

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				B	0	Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, moist	
	5.4	127	75/10"	D		Moderately Weathered Granitic Bedrock: **SILTY SAND (SM)** slightly moist, light brown to brown, very dense	1510
	3.3		50/4"	D	5	@ 5 ft, brown, dry, with gravel	
	2.2		50/3"	D			1505
	2.0		50/3"	D	10		1500
			50/2"	D	15	@ 15 ft, no sample recovery	
	2.0		50/3"	S			
						Total Depth 17 ft	

SAMPLE TYPES
- [C] Rock Core
- [S] Standard Split Spoon
- [D] Drive Sample
- [B] Bulk Sample
- [T] Tube Sample

DATE DRILLED: 1-19-09

EQUIPMENT USED: 8 " Hollow Stem

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2833.I

VINEYARD/VAL VISTA

LOG OF BORING NO. B-2

FIGURE A-2

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
					0	Weathered Granitic Bedrock: **SAND (SP)** brown, slightly moist, with gravel	
	3.4	127	50/6"	D		Moderately Weathered Granitic Bedrock: **SAND (SP)** brown, trace of silt, dry, very dense, with gravel	
	2.3		50/4"	D	5		1515
	7.6		50/5"	S		**SILTY SAND (SM)** brown, slightly moist, very dense, with gravel	
	24.9		36	S	10	**SANDY SILT (ML)** light brown, very moist, very stiff, with gravel (highly weathered)	1510
	18.2		16	S	15	@ 15 ft, stiff	1505
	3.1		50/3"	D	20	**SILTY CLAY (CL)** brown, dry, hard, with gravel	1500
						Total Depth 21 ft	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
1-19-09
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2833.I
VINEYARD/VAL VISTA

LOG OF BORING NO. B-3

FIGURE A-3

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
			B	0	Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, moist	
6.9	135	59	D		@ 2 ft, slightly moist, dense, trace of gravel	
3.5		50/6"	S	5	Moderately Weathered Granitic Bedrock: **SAND WITH SILT (SP-SM)** brown, dry, very dense, with gravel	1510
3.2		50/5"	S		**SILTY SAND (SM)** brown, dry, very dense, trace of gravel	
2.4		50/5"	S	10		1505
2.2		50/4"	S	15	**SAND WITH SILT (SP-SM)** brown, dry, very dense, with gravel	1500
2.1		50/4"	S	20	**SILTY SAND (SM)** brown, dry, very dense, with gravel	1495
1.8		50/2"	S	25	**SAND WITH SILT (SP-SM)** brown, dry, very dense, with gravel	1490
					Total Depth 26 ft	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
1-19-09

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2833.I

VINEYARD/VAL VISTA

LOG OF BORING NO. B-4

FIGURE A-4

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	Weathered Granitic Bedrock: **SILTY SAND (SM)** light brown, moist, with gravel	
8.6	117	83/11"	D		Moderately Weathered Granitic Bedrock: **SILTY SAND (SM)** light brown, moist, very dense, with gravel	1510
7.9	112	50/5"	D	5		
4.3	128	52	D		@ 7 ft, light brown to brown, dry, dense	1505
		50/3"	D	10	@ 10 ft, no sample recovery	
						1500
1.9		50/3"	D	15	**SAND WITH SILT (SP-SM)** brown, dry, very dense, with gravel	
					Total Depth 16 ft	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
1-19-09

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2833.I
VINEYARD/VAL VISTA

LOG OF BORING NO. B-5

FIGURE A-5

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, moist	
	13.0	116	80/9"	D		Moderately Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, moist, very dense, with gravel and trace clay	1505
	6.7	100	80/9"	D	5	**SAND WITH SILT (SP-SM)** light brown, slightly moist, very dense	
	8.6	113	50/5"	D		**SILTY SAND (SM)** light brown, moist, very dense, with gravel	1500
	13.0		50/5"	S	10	**SANDY SILT (ML)** light brown, moist, hard, with gravel	
							1495
						Auger Refusal at 13 ft, likely the presence of boulders/cobbles	

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
1-19-09

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2833.I
VINEYARD/VAL VISTA

LOG OF BORING NO. B-6

FIGURE A-6

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					0	Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, moist	
	3.4	144	88	D		Moderately Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, dry, very dense, with gravel	1505
	2.8	119	50/6"	D	5	**SAND WITH SILT (SW-SM)** brown, dry, very dense	
	1.6		50/5"	D		@ 7 ft, with gravel	1500
	2.5		50/2"	S	10		
							1495
	4.6		50/5"	D	15	@ 15 ft, slightly moist	
						Auger Refusal at 16 ft, likely the presence of boulders/cobbles	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
1-19-09

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2833.I

VINEYARD/VAL VISTA

LOG OF BORING NO. B-7

FIGURE A-7

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	Natural Soil: **SILTY SAND (SM)**brown, moist	
8.7	106	8	D		@ 2 ft, loose, trace of porosity	
						1515
2.8	104	31	D	5	Weathered Granitic Bedrock: **SILTY SAND (SM)** reddish brown, dry, medium dense	
3.4	116	63	D		Moderately Weathered Granitic Bedrock: **SAND WITH SILT(SP-SM)**red to brown, dry, dense, trace of gravel	
1.2		50/4"	S	10	@ 10 ft, very dense, with gravel	1510
1.7		50/4"	D	15	@ 15 ft, greyish brown	1505
					Total Depth 16 ft	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
1-19-09
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2833.I
VINEYARD/VAL VISTA

LOG OF BORING NO. B-8

FIGURE A-8

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
					0	Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, moist	1515
	10.1	125	58	D		@ 2 ft, dense, with gravel	
	2.9		50/5"	D	5	Moderately Weathered Granitic Bedrock: **SAND WITH SILT(SP-SM)** light brown, dry, very dense, with gravel	1510
	1.6		50/4"	S		**SILTY SAND (SM)** light brown, dry, with trace of gravel	
	1.4		50/3"	S	10	**SAND WITH SILT (SP-SM)** light brown, dry, very dense, with gravel	1505
	1.7		50/3"	S	15		1500
						Total Depth 16 ft	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
1-19-09

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2833.I

VINEYARD/VAL VISTA

LOG OF BORING NO. B-9

FIGURE A-9

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, moist	
	6.2	136	80/10"	D		Moderately Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, slightly moist, very dense, with gravel	1505
	1.3		50/3"	D	5	**SAND WITH SILT (SP-SM)** with gravel, brown, dry, very dense	
	4.6		50/1"	D		**SILTY SAND (SM)** with gravel, light brown, slightly moist, very dense	1500
			50/5"	D	10	@ 10 ft, no sample recovery	
							1495
	1.0		50/4"	D	15	**SAND WITH SILT (SP-SM)** grey, dry, very dense, with gravel	
						Total Depth 16 ft	

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 1-19-09

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2833.I

VINEYARD/VAL VISTA

LOG OF BORING NO. B-10

FIGURE A-10

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					0	Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, moist	
	9.3	119	25	D		@ 2 ft, light brown, medium dense, with gravel	
	1.4		50/5"	D	5	Moderately Weathered Granitic Bedrock: **SAND (SP)** light brown, dry, very dense	1510
	1.8	128	50/5"	D		**SAND WITH SILT (SP-SM)** light brown, dry, very dense, with gravel	
	1.4		50/3"	S	10		1505
						Total Depth 11 ft	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 1-19-09

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2833.I

VINEYARD/VAL VISTA

LOG OF BORING NO. B-11

FIGURE A-11

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Natural Soil: **SILTY SAND (SM)**brown, moist	
	9.1	109	6	D		@ 2 ft, trace of porosity, very loose to loose	
			75/10"	D	5	Moderately Weathered Granitic Bedrock: **SILTY SAND (SM)** light brown, dry, very dense, with trace of gravel	1510
			50/3"	S			
						@ 7 ft, no sample recovery	
	2.1		50/1"	S	10		1505
						Total Depth 11 ft	

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
1-19-09

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not encountered

GPI

PROJECT NO.: 2833.I

VINEYARD/VAL VISTA

LOG OF BORING NO. B-12

FIGURE A-12

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
					0	Natural Soil: **SILTY SAND (SM)**brown, moist, very loose to loose	
	9.7	111	6	D			1515
	2.5	123	72	D	5	Moderately Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, dry, trace of porosity, very dense, with gravel	
	2.0		50/5"	S		**SAND WITH SILT (SP-SM)** light brown, dry, very dense, with trace of gravel	1510
	2.5		50/5"	S	10	@ 10 ft, with gravel	
						Total Depth 11 ft	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
1-19-09

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not encountered

GPI

PROJECT NO.: 2833.I
VINEYARD/VAL VISTA

LOG OF BORING NO. B-13

FIGURE A-13

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				B	0	Natural Soil: **SANDY SILT (ML)**brown, moist, trace of clay	
	12.0	125	34	D		Weathered Granitic Bedrock: **SILTY SAND (SM)**light brown, moist, medium dense	1500
	1.9		50/4"	D	5	Moderately Weathered Granitic Bedrock: **SAND WITH SILT (SP-SM)**brown, dry, very dense, with gravel	
	2.2		50/4"	S			1495
	2.2		50/3"	S	10		
							1490
	7.0		73	S	15	**SILTY SAND (SM)**grey to brown, slightly moist, very dense, with gravel	
						Total Depth 16 ft	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
1-19-09
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not encountered

GPI

PROJECT NO.: 2833.I
VINEYARD/VAL VISTA

LOG OF BORING NO. B-14

FIGURE A-14

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				B	0	Natural Soil: **SILTY SAND (SM)**brown, moist	1525
	9.3	109	6	D		@ 2 ft, trace of porosity, very loose to loose	
	4.0	108	39	D	5	Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, dry, medium dense, trace gravel	1520
	2.3		50/4"	S		Moderately Weathered Granitic Bedrock: **SILTY SAND (SM)** brownish grey, very dense	
	2.2		50/6"	S	10	**SAND WITH SILT (SP-SM)**light brown, dry, very dense, trace of gravel	1515
			50/1"	S	15	@ 15 ft, no sample recovery	1510
						Total Depth 15 ft	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
1-20-09

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not encountered

GPI

PROJECT NO.: 2833.I

VINEYARD/VAL VISTA

LOG OF BORING NO. B-15

FIGURE A-15

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
			B	0	Natural Soil: **SILTY SAND (SM)**brown, moist, loose	
8.5	109	8	D			
4.6	111	39	D	5	Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, slightly moist, medium dense, trace gravel	1515
4.4	134	80/11"	D		Moderately Weathered Granitic Bedrock: **SILTY SAND (SM)** light brown, very dense	
1.8		50/2"	S	10	@ 10 ft, dry, with gravel	1510
		50/1"	S	15		1505
					Total Depth 16 ft	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
1-20-09
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not encountered

GPI

PROJECT NO.: 2833.I
VINEYARD/VAL VISTA

LOG OF BORING NO. B-16

FIGURE A-16

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				B	0	Natural Soil: **SILTY SAND (SM)**brown, moist	
	5.5	92	11	D		**SANDY SILT (ML)**light brown, slightly moist, firm, with gravel	1515
	1.9	133	80/11"	D	5	Moderately Weathered Granitic Bedrock: **SILTY SAND (SM)** light brown, dry, very dense	
			50/5"	S		@ 7 ft, no sample recovery	1510
	2.4		50/1"	S	10	@ 10 ft, grey, with gravel	
						Total Depth 11 ft	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
1-20-09

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not encountered

GPI

PROJECT NO.: 2833.I

VINEYARD/VAL VISTA

LOG OF BORING NO. B-17

FIGURE A-17

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
			B	0	Natural: **SANDY SILT (ML)** brown, dry to slightly moist	
10.1	106	64	D		Weathered Granitic Bedrock: **SAND (SP)** red brown, moist, dense	1545
3.8	121	85/11"	D	5	Moderately Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, slightly moist, very dense	
3.7	108	50/5"	D			1540
3.7	110	50/5"	D	10	@ 10 feet, trace gravel	1535
4.1		77/11"	S	15	@ 15 feet, grey	1530
3.0		50/5"	S	20		1525
3.3		50/4"	S	25		1520
2.8		50/3	S	30		1515
2.9		50/1.5'	S	35		
2.7		50/2"	S			
					Refusal @ 38 feet	

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 8-8-17

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2833.I

VINEYARD/VAL VISTA

LOG OF BORING NO. B-101

FIGURE A-18

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
			B	0	Natural: **SILTY SAND (SM)** dry to slightly moist, brown, very dense, with gravel	
6.1	114	79	D			1540
5.1	103	50/4"	D	5	Weathered Granitic Bedrock: **SAND with SILT (SP-SM) brown, slightly moist, very dense**	
8.0	118	50/5"	D		Moderately Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, moist, very dense, with gravel	1535
5.3		72/9"	S	10	@ 10 feet, slightly moist, trace gravel	
						1530
1.7		50/2"	D	15	**SAND with SILT (SP-SM)** brown, dry, very dense, with gravel	
						1525
1.5		50/2"	S	20	**SAND (SP)** grey, dry, very dense	
						1520
1.8		50/3"	S	25		
						1515
		50/1"	S	30	@ 10 feet, no recovery	
					Refusal @ 31 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
8-8-17

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2833.I

VINEYARD/VAL VISTA

LOG OF BORING NO. B-102

FIGURE A-19

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					0	Natural: **SILTY SAND (SM)** brown, dry to slightly moist, dense to very dense with gravel	
	3.8	123	75	D		@ 2 feet, moist	
							1540
	3.6	104	50/4"	D	5	Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, slightly moist, very dense	
	4.4 3.9	113	50/2"	D			
							1535
	3.3		50/3"	D	10	Moderately Weathered Granitic Bedrock: **SILTY SAND (SM)** brown, slightly moist, very dense, with gravel	
							1530
	2.3		50/1.5'	S	15		
							1525
	2.0		50/1.5'	D	20		
							1520
	2.5		50/2"	S	25		
							1515
	6.5		50/1.5'	D	30		
			50/1"	S			
						Refusal @ 33 feet	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 8-8-17

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2833.I

VINEYARD/VAL VISTA

LOG OF BORING NO. B-103

FIGURE A-20

APPENDIX B

APPENDIX B

LABORATORY TESTS

INTRODUCTION

Representative undisturbed soil samples and bulk samples were carefully packaged in the field and sealed to prevent moisture loss. The samples were then transported to our Cypress office for examination and testing assignments. Laboratory tests were performed on selected representative samples as an aid in classifying the soils and to evaluate the physical properties of the soils affecting foundation design and construction procedures. Detailed descriptions of the laboratory tests are presented below under the appropriate test headings. Test results are presented in the figures that follow.

MOISTURE CONTENT AND DRY DENSITY

Moisture content and dry density were determined from a number of the samples. The samples were first trimmed to obtain volume, weighed to determined the wet weight, and then were dried in accordance with ASTM D 2216. After drying, the weight of each sample was measured, and moisture content and dry density were calculated. Moisture content and dry density values are presented on the boring logs in Appendix A.

GRAIN SIZE DISTRIBUTION

Multiple soil samples were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. That portion of the material retained on the No. 200 sieve was oven-dried and weighed to determine the percentage of the material passing the No. 200 sieve. For select samples, the retained material was then run through a standard set of sieves in accordance with ASTM D 422 to classify the coarse fraction of representative sample. A summary of the percentages passing the No. 200 sieve is presented below. The grain size distribution data obtained from the full sieve analyses are presented in Figures B-1 to B-3.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-2	2	Silty Sand (SM)	25
B-6	5	Sand with Silt (SP-SM)	8
B-8	5	Silty Sand (SM)	27
B-11	7	Sand with Silt (SP-SM)	10
B-12	5	Silty Sand (SM)	21
B-14	0 – 5	Silty Sand (SM)	38
B-15	0 – 5	Silty Sand (SM)	32
B-15	2	Silty Sand (SM)	30

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-15	5	Silty Sand (SM)	28
B-15	10	Sand with Silt (SP-SM)	11
B-16	2	Silty Sand (SM)	27
B-16	5	Silty Sand (SM)	31
B-16	7	Silty Sand (SM)	17
B-16	10	Silty Sand (SM)	13
B-17	0 – 5	Silty Sand (SM)	35
B-17	2	Sandy Silt (ML)	53
B-17	10	Silty Sand (SM)	16
B-101	2	Sandy Silt (ML)	53
B-101	7	Silty Sand (SM)	17
B-102	5	Sand with Silt (SP-SM)	11
B-102	10	Silty Sand (SM)	21
B-103	5	Silty Sand (SM)	16
B-103	10	Silty Sand (SM)	13

ATTERBERG LIMITS

Liquid and plastic limits were determined for select samples of cohesive material in accordance with ASTM D 4318. Results of the Atterberg Limits tests are summarized on Figure B-4.

DIRECT SHEAR

Direct shear tests were performed on undisturbed and remolded bulk samples in accordance with ASTM D 3080. The bulk sample was remolded to approximately 90 percent of maximum density (ASTM D1557). The samples were placed in the shear machine, and a normal load comparable to the in-situ overburden stress was applied. The samples were inundated, allowed to consolidate, and then were sheared to failure. The tests were repeated on additional test specimens under increased normal loads. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear tests are presented in Figures B-5 to B-7.

COMPACTION TEST

Maximum dry density/optimum moisture tests were performed in accordance with ASTM D 1557 on select, representative bulk samples of the site soils. The test results are as follows:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	OPIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)
B-14	0 – 5	Silty Sand (SM)	9.0	133
B-16	0 – 5	Silty Sand (SM)	8.0	132

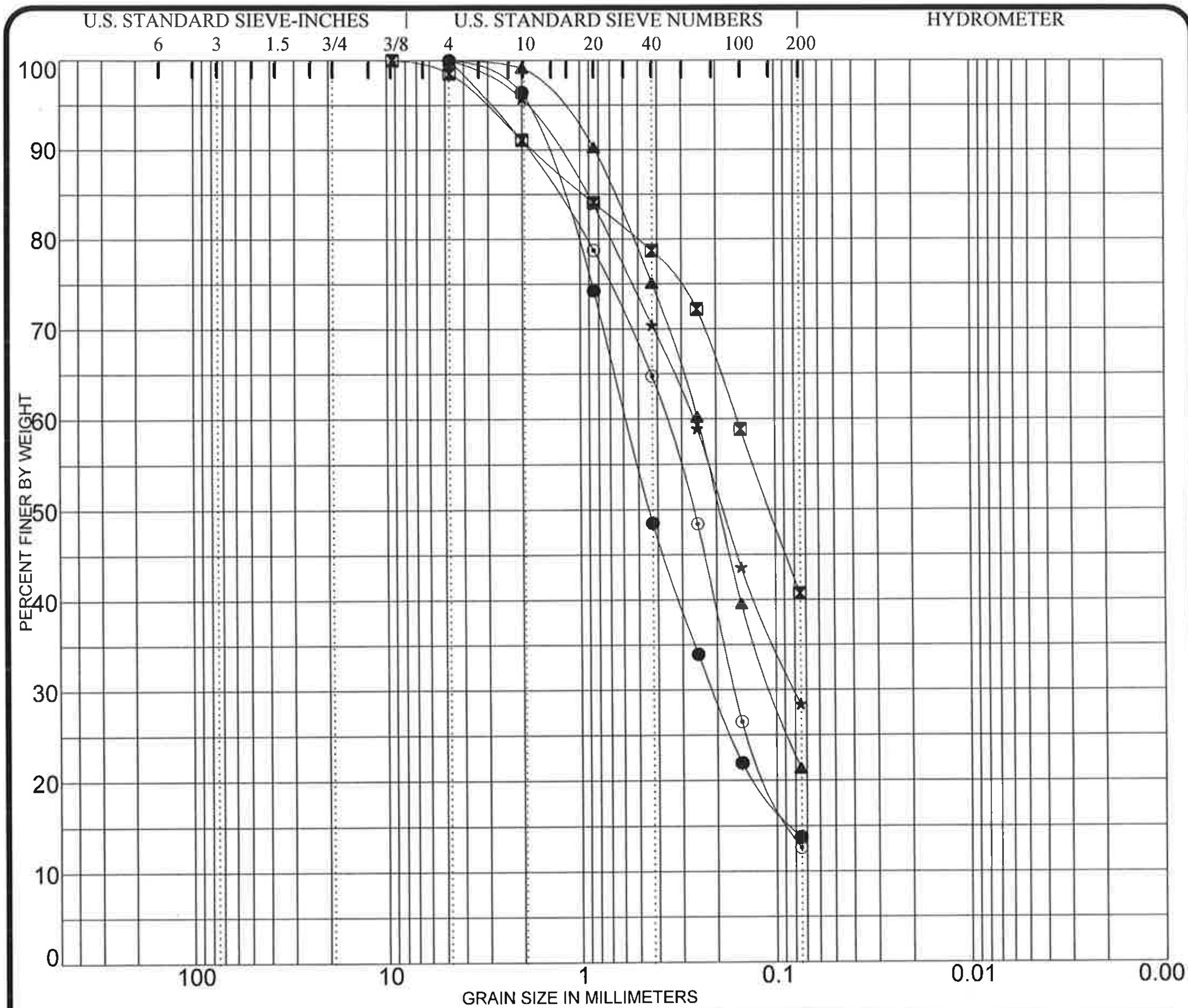
R-VALUE

Suitability of the near-surface soils for pavement was evaluated by conducting an R-value test. The test was performed in accordance with ASTM D 2844 by GeoLogic Associates (GLA) under subcontract to GPI. The result of the test is as follows:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	R-VALUE BY EXPANSION
B-16	0 – 5	Silty Sand (SM)	63

CORROSIVITY

Soil corrosivity testing was performed by Schiff and Associates (currently HDR) on a soil samples provided by GPI. The test results are summarized in Table 1 of this Appendix.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample Location		Classification	MC%	LL	PL	PI	Cc	Cu
●	B-1	5.0	SILTY SAND (SM)						
⊠	B-1	15.0	SILTY SAND (SM)						
▲	B-15	7.0	SILTY SAND (SM)						
★	B-16	0.0	SILTY SAND (SM)						
⊙	B-17	5.0	SILTY SAND (SM)						

	Sample Location		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-1	5.0	4.75	0.58	0.211		0.0	86.2	13.8	
⊠	B-1	15.0	9.35	0.16			1.5	57.8	40.7	
▲	B-15	7.0	4.75	0.25	0.104		0.0	78.7	21.3	
★	B-16	0.0	4.75	0.26	0.081		0.0	71.6	28.4	
⊙	B-17	5.0	4.75	0.36	0.163		0.0	87.4	12.6	

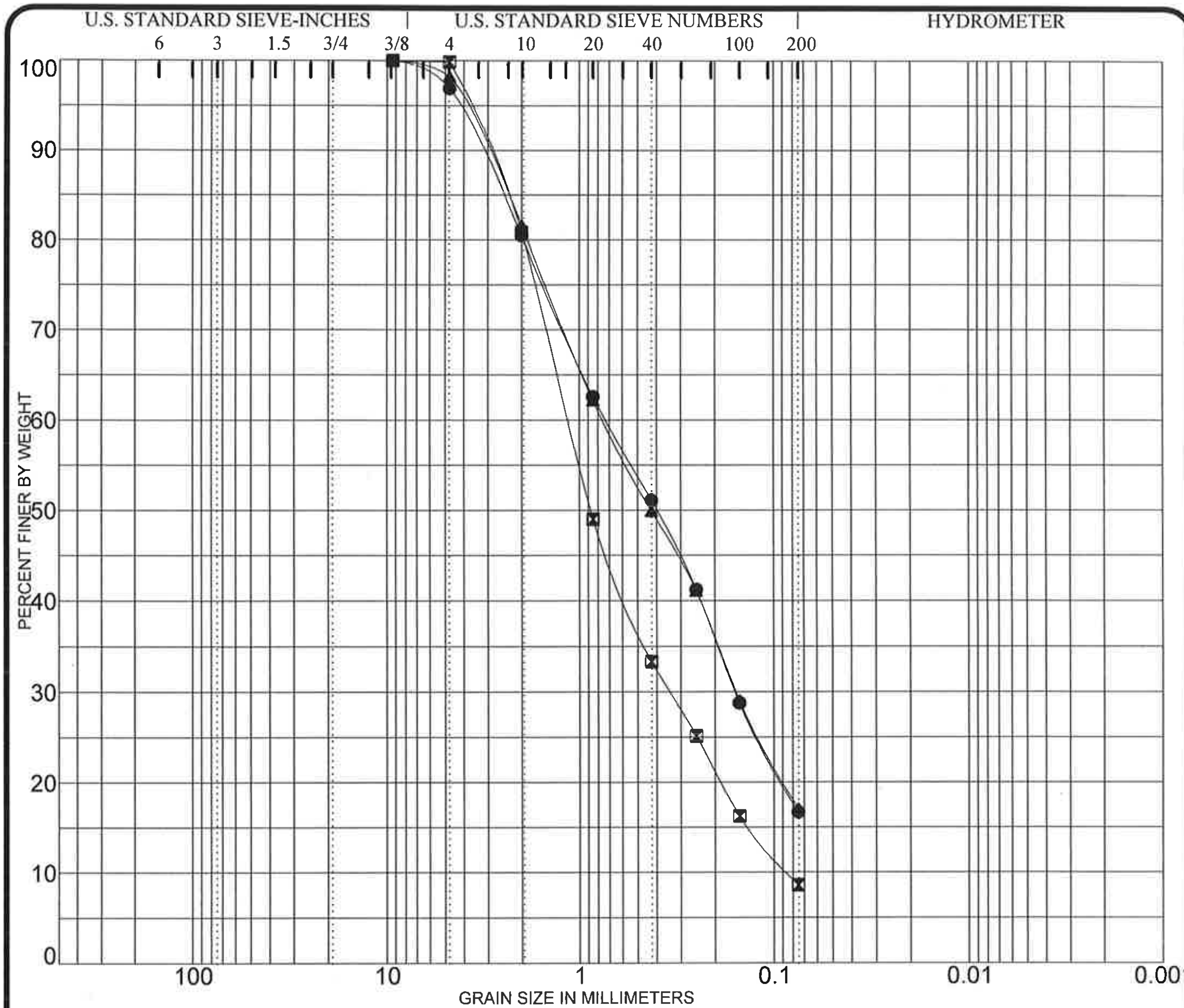
PROJECT: VINEYARD/VAL VISTA

PROJECT NO. 2833.I

GPI

GRAIN SIZE DISTRIBUTION

FIGURE B-1



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample Location		Classification	MC%	LL	PL	PI	Cc	Cu
●	B-4	10.0	SILTY SAND (SM)						
☒	B-7	5.0	SAND with SILT (SW-SM)					1.21	13.4
▲	B-9	7.0	SILTY SAND (SM)						

	Sample Location		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-4	10.0	9.35	0.73	0.158		3.1	80.2	16.7	
☒	B-7	5.0	9.35	1.14	0.343	0.0853	0.1	91.3	8.6	
▲	B-9	7.0	9.35	0.75	0.156		1.8	81.2	17.1	

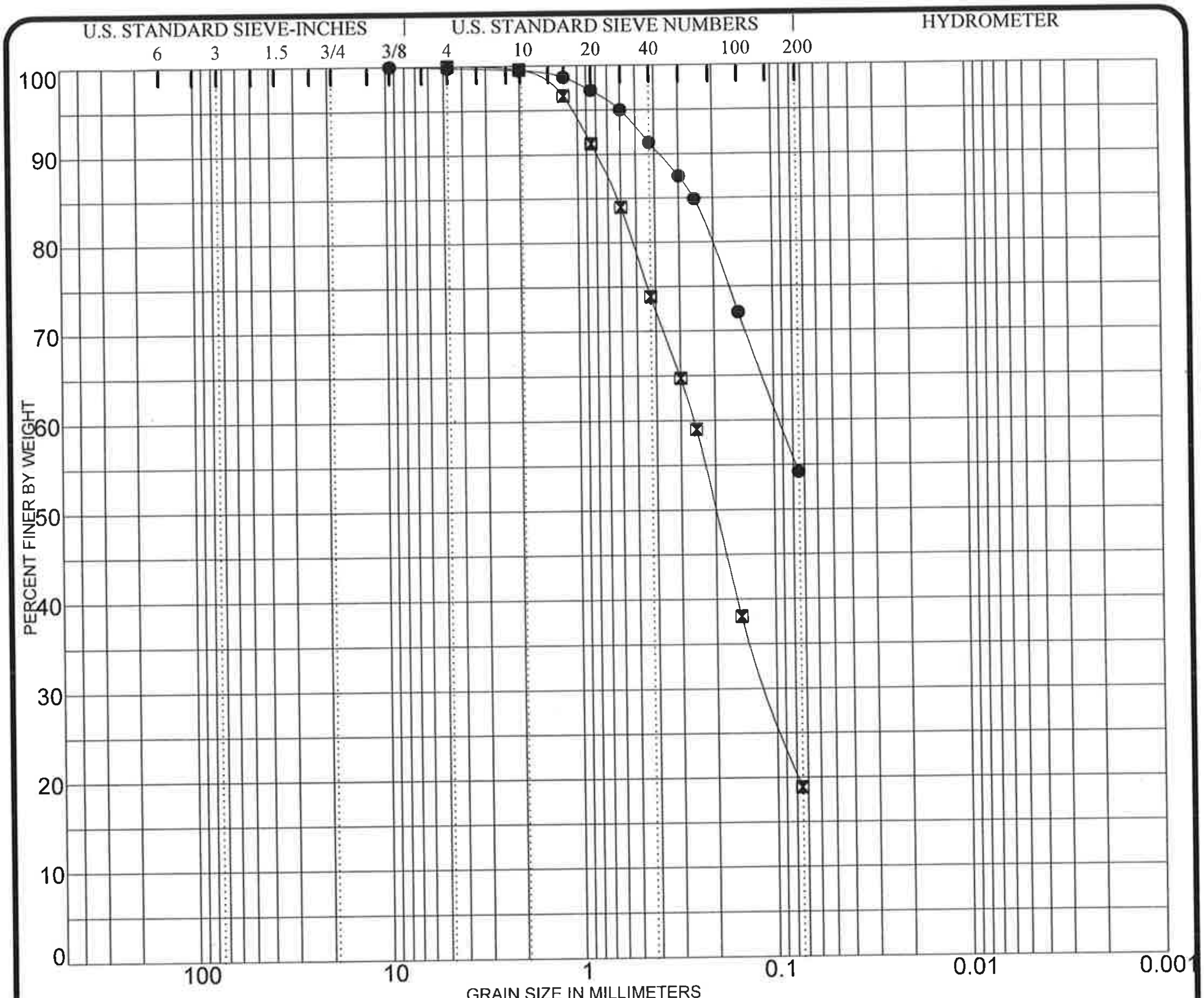
PROJECT: VINEYARD/VAL VISTA

PROJECT NO. 2833.I

GPI

GRAIN SIZE DISTRIBUTION

FIGURE B-2



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Sample Location		Classification	MC%	LL	PL	PI	Cc	Cu
●	B-101	2.0	SANDY SILT (ML)						
⊠	B-101	7.0	SILTY SAND (SM)						

	Sample Location		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-101	2.0	9.50	0.09			0.2	45.6	54.2	
⊠	B-101	7.0	4.75	0.26	0.112		0.0	81.1	18.9	

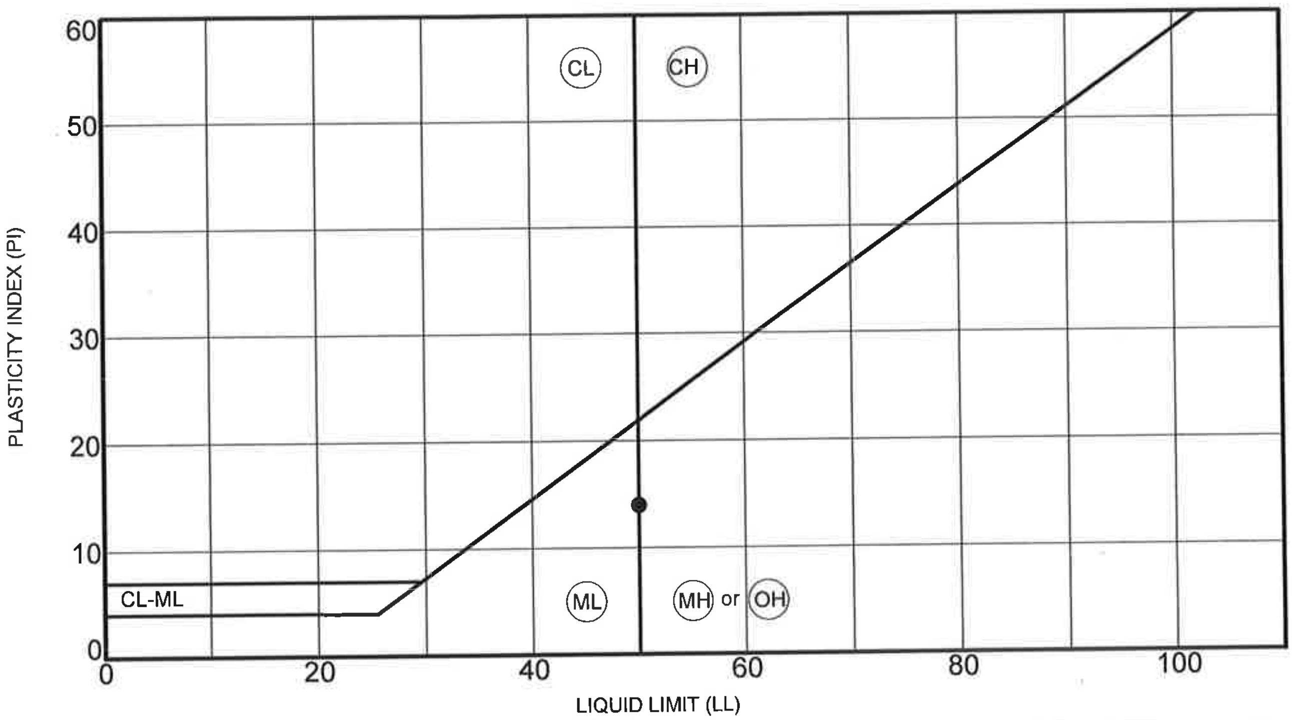
PROJECT: VINEYARD/VAL VISTA

PROJECT NO. 2833.I

GPI

GRAIN SIZE DISTRIBUTION

FIGURE B-3



	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
●	B-3	15.0	50	36	14		SANDY SILT (ML/MH)

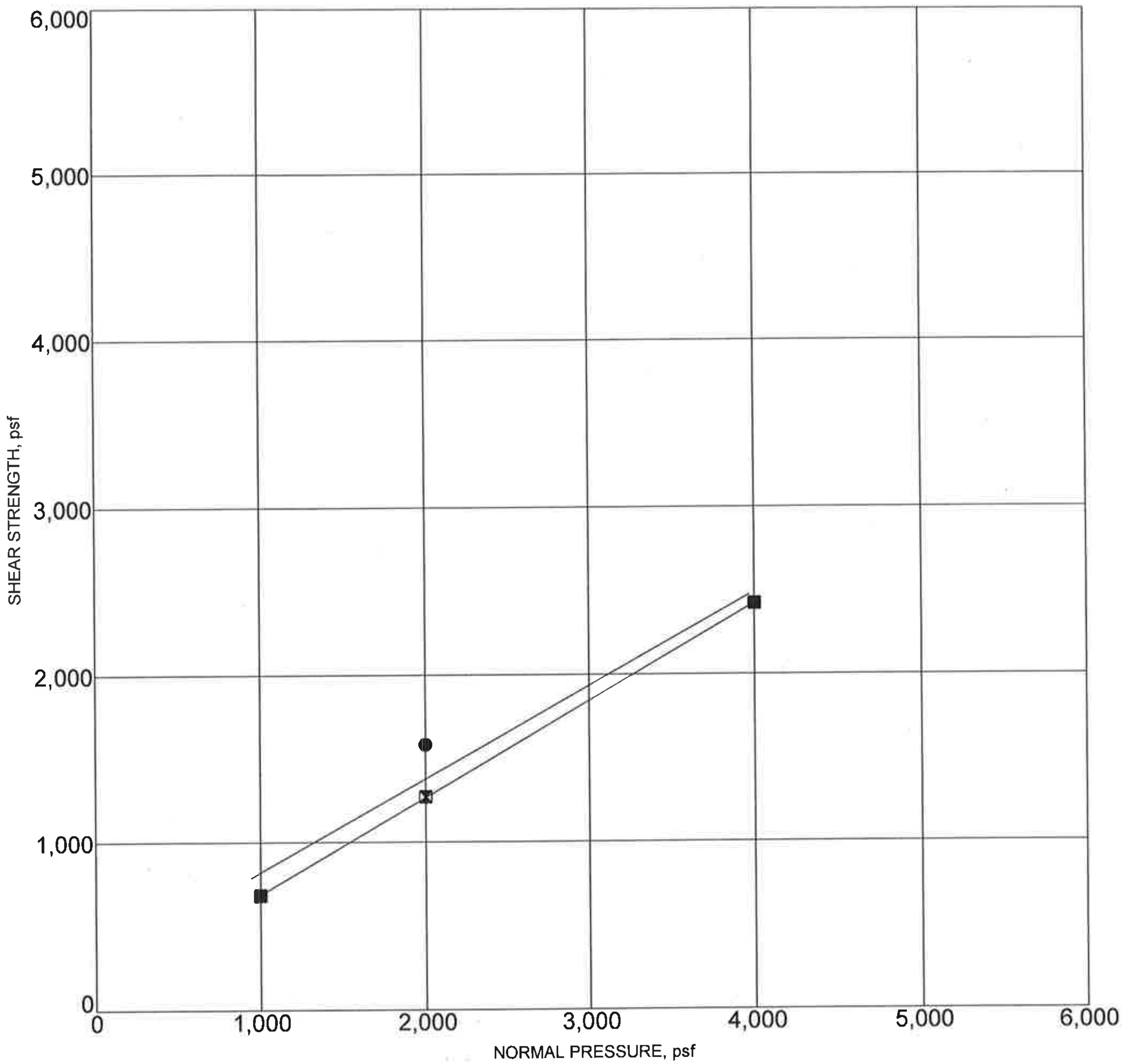
PROJECT: VINEYARD/VAL VISTA

PROJECT NO. 2833.I

GPI

ATTERBERG LIMITS TEST RESULTS

FIGURE B-4



● **PEAK STRENGTH**
Friction Angle= 29 degrees
Cohesion= 264 psf

ULTIMATE STRENGTH
Friction Angle= 30 degrees
Cohesion= 108 psf

Sample Location		Classification	DD,pcf	MC,%
B-8	5.0	SILTY SAND (SM)	104	2.8

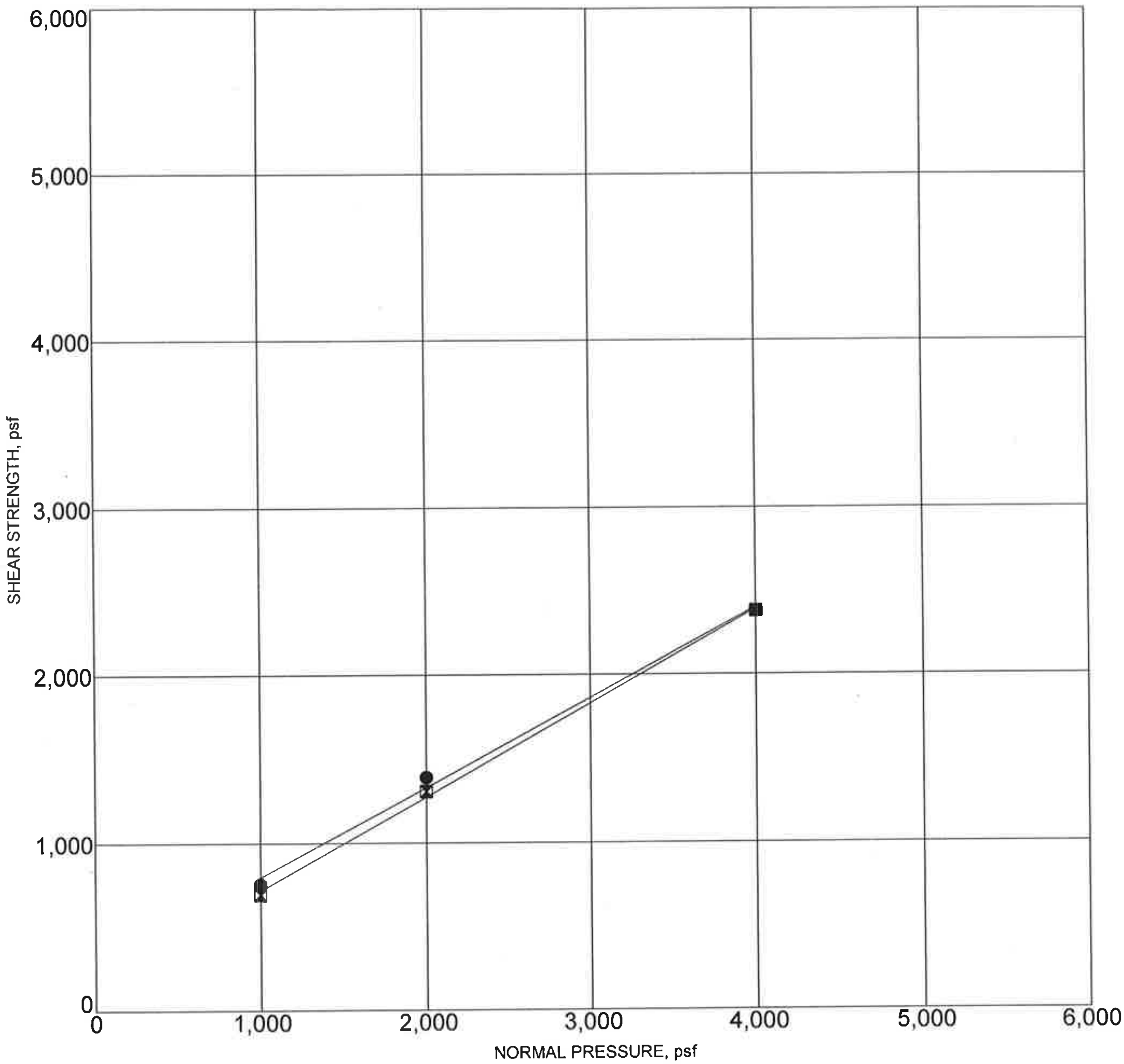
PROJECT: VINEYARD/VAL VISTA

PROJECT NO.: 2833.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-5



● **PEAK STRENGTH**
Friction Angle= 28 degrees
Cohesion= 264 psf

☒ **ULTIMATE STRENGTH**
Friction Angle= 29 degrees
Cohesion= 162 psf

Note: Samples remolded to 90% of maximum dry density

Sample Location		Classification	DD,pcf	MC,%
B-14	0-5	SILTY SAND (SM)	120	9.5

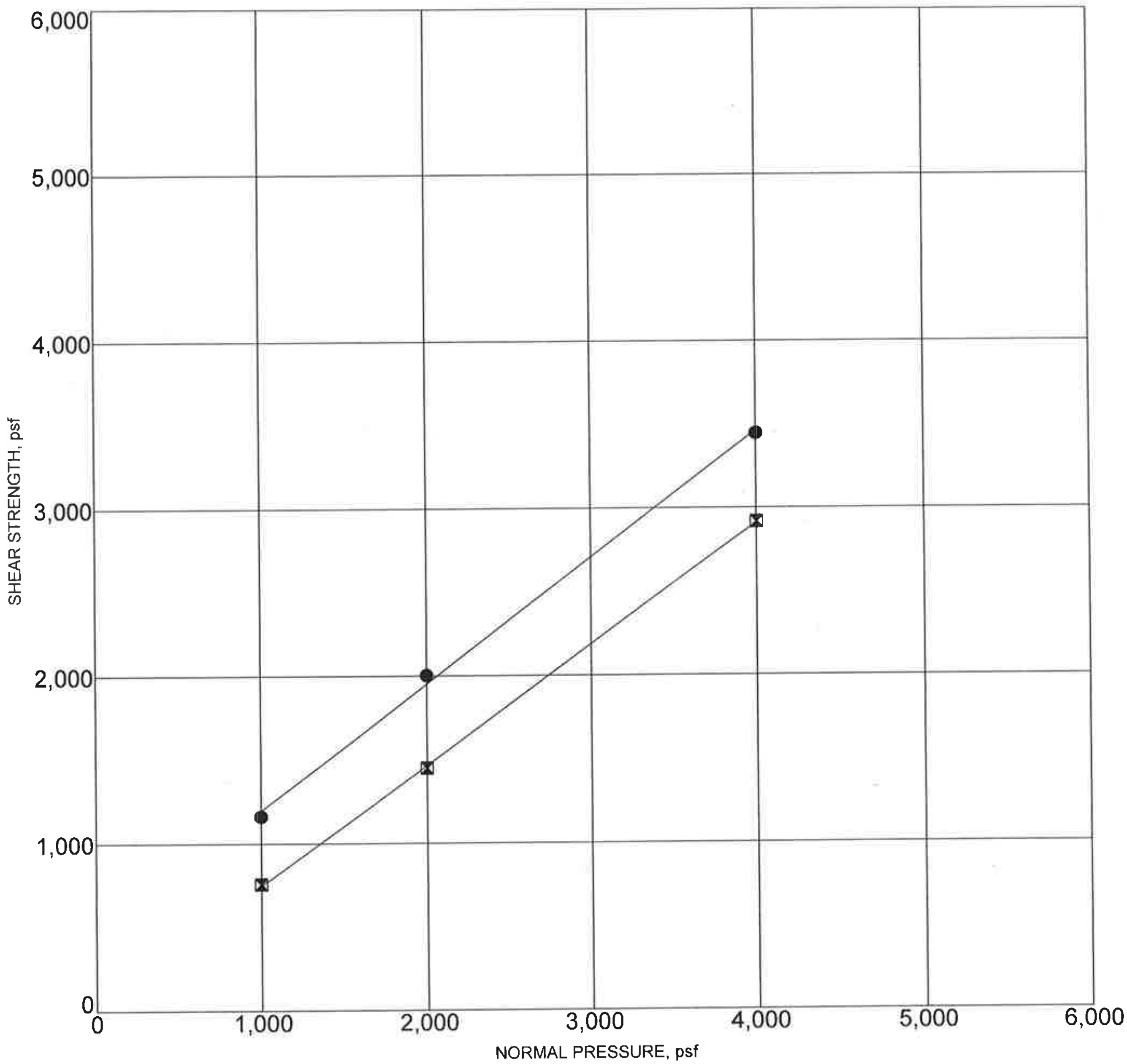
PROJECT: VINEYARD/VAL VISTA

PROJECT NO.: 2833.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-6



● **PEAK STRENGTH**
Friction Angle= 37 degrees
Cohesion= 444 psf

⊠ **ULTIMATE STRENGTH**
Friction Angle= 36 degrees
Cohesion= 24 psf

Sample Location		Classification	DD,pcf	MC,%
B-101	5.0	WEATHERED BEDROCK: SILTY SAND (SM)	121	3.8

PROJECT: VINEYARD/VAL VISTA

PROJECT NO.: 2833.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-7