Appendix B. Geotechnical Investigation

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GEOTECHNICAL PROFESSIONALS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED VIVANTE NEWPORT COAST
850 SAN CLEMENTE DRIVE
NEWPORT BEACH, CALIFORNIA

Prepared for:
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Project No. 2870.I

November 13, 2018

GEOTECHNICAL PROFESSIONALS INC.

November 13, 2018

Vivante Newport Center, LLC
c/o Nexus Companies
1 MacArthur Place, Suite 300
Santa Ana, California 92707

Attention: Mr. Brad Cameron, S.E.
Technical Director

Subject: Report of Geotechnical Investigation
Proposed Vivante Newport Coast
850 San Clemente Drive
Newport Beach, California
GPI Project No. 2870.I

Dear Mr. Cameron:

Transmitted herewith is an electronic copy of our report of geotechnical investigation for the subject project. The report presents our evaluation of the foundation conditions at the site and recommendations for design and construction. Hard copies can be provided as needed.

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

cc: Doug Burroughs, Nexus Companies (email)
Stephen Scanlon, Nexus Companies (email)
Stephen Harris, S.E., Simpson Gumpertz & Heger (email)
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TABLE OF CONTENTS

LIST OF FIGURES

APPENDIX A

APPENDIX B

APPENDIX B

1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for the proposed Vivante Newport Coast project in Newport Beach, California. The site location is shown on the Site Location Map, Figure 1.

1.2 PROJECT DESCRIPTION

The proposed development will consist of a 7-story (6 levels above grade, one-level below grade) senior assisted living facility. The overall site is approximately 2.9 acres in plan and currently consists of two parcels. Based on plans provided by the Project Architect and Civil Engineer, the proposed building will be about 26,500 square feet in plan, with the majority underlain by a single subterranean level. The subterranean level is approximately 24,500 square feet in plan with a finished floor at an approximate elevation of +169 feet. The subterranean level also includes a pool area that extends to the northwest, outside of the above-grade portion of the proposed structure. The basement pool area will be overlain by an outdoor amenity deck. The northeast portion of the proposed building will be supported at-grade at an approximate elevation of +181 feet.

The project will also include new pavement and hardscape, landscape, minor retaining walls, bio-infiltration basins, and a new drive entrance. The existing site configuration is shown on the Existing Site Plan, Figure 2. The proposed site configuration, including the basement limits, is shown on the Proposed Site Plan, Figure 3.

It is our understanding that the proposed structure will be of Type 1 construction. Based on information provided by SGH, the Project Structural Engineers, column loads are anticipated to range from a minimum of 120 kips to a maximum of 800 kips, with an average load of 400 kips. Mat foundation pressures for the subterranean portion of the building are anticipated to range from about 1,000 to 2,000 pounds per square foot (psf). Preliminary grading plans provided by Tait & Associates the Project Civil Engineer indicate potential cuts of up to 11 feet within the basement level of the proposed structure and fills of up to 10 feet outside the building footprint. The preliminary grading plans indicate a potential net import (fill) of approximately 6,000 cubic yards of material. A subsurface section showing potential cut and fill areas is shown on the Subsurface Cross Section, Figure 4.

Our recommendations are based upon the above structural and grading information. We should be notified if the actual loads and/or grades change during the project design to either confirm or modify our recommendations. Also, when additional project grading and structural plans become available, we should be provided with a copy for review and comment.

1.3 PURPOSE OF INVESTIGATION

The primary purpose of this investigation and report is to provide an evaluation of the existing geotechnical and seismic conditions at the site, as they relate to the design and construction of the proposed development. More specifically, this investigation was aimed at providing geotechnical recommendations for earthwork, and design of foundations, walls below grade, and pavements.

1.4 PRIOR SITE WORK

GPI was provided with a prior, feasibility-level geotechnical investigation by others for a proposed high-rise residential structure at the subject site (Group Delta, 2015). The prior investigation included three exploratory borings and laboratory testing consisting of fines content analyses, Atterberg limits, consolidation, and corrosivity. The prior investigation was referenced when planning the current investigation and preparing this report. We have used select prior geotechnical data for this study where relevant, and assume the liability of relying on prior data by others.

2.0 SCOPE OF WORK

Our scope of work for this investigation consisted of a review of historical aerial photographs, review of prior investigations, geotechnical field exploration, laboratory testing, engineering analysis, and the preparation of this report.

Our geotechnical field exploration program consisted of four Cone Penetration Tests (CPT's) and two exploratory borings. The CPT's were advanced to depths ranging from 50 to 60 feet below existing site grades. The exploratory borings were drilled to depths of 21 to 46 feet below existing site grades. Details of the field procedures and logs of the CPT's and Borings are presented in Appendices A and B, respectively. The locations of the subsurface explorations are shown on Figures 2 and 3.

Laboratory tests were performed on selected representative soil samples as an aid in soil classification and to evaluate the engineering properties of the soils. The geotechnical laboratory testing included determinations of moisture content and dry density, fines content, Atterberg limits, shear strength (direct shear), consolidation, compaction (maximum density/optimum moisture), and expansion index. We have also incorporated prior test results at the site by others where appropriate. Laboratory testing procedures and results are summarized in Appendix C.

Engineering evaluations were performed to provide earthwork criteria, foundation, retaining wall, 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 development history of the site is based on review of historical aerial photographs (Historic Aerials). In reviewing aerial photographs of the site, we noted that the site appears to have remained in its approximate current configuration since at least 1980. In aerial photographs from 1972 and prior, the site and surrounding area are undeveloped. The existing properties to the north (residential apartments) and east (multi-story parking garage) of the subject site were redeveloped after 2014.

3.2 SURFACE CONDITIONS

The subject site is bounded by San Clemente Drive to the south, office/commercial development to the west, Civic Plaza and a residential apartment development to the north, and an existing parking structure to the east. The site is currently occupied by two vacant, single-story buildings that used to house the Orange County Museum of Art. The remaining portions of the site are occupied by asphalt pavements (parking and drive aisles), concrete hardscape, and landscaping.

Topography in the site vicinity generally slopes down to the west and north. Within the subject site limits, the ground surface slopes down to the southwest at an approximate 3 to 4 percent grade. A topographic plan provided by the Project Civil Engineer indicates current ground surface elevations across the site ranging from approximately +180 feet in the northeast to +164 feet in the southwest.

Existing pavement sections at our boring locations consisted of 3 to 4 inches of asphalt concrete over 3 to 6 inches of aggregate base. The existing pavement at the time of our investigation was generally in fair condition.

3.3 SUBSURFACE SOILS

Our field investigation disclosed a subsurface profile consisting of shallow undocumented fill over natural soils and weathered bedrock. Detailed descriptions of the subsurface conditions encountered are shown on the Logs of CPT's and Borings in Appendices A and B, respectively.

Fill soils, to depths of approximately 2 to 4 feet, were encountered in our exploratory borings. The fill soils consisted of moist clayey sands and sands. Documentation on the placement and compaction of the fill was not available at the time this report was prepared. Localized deeper fill may be in-place within the limits of the existing buildings.

The underlying natural soils consist primarily of marine terrace deposits overlying weathered claystone and siltstone bedrock of the Monterey Formation. The marine terrace deposits consist of sands, silty sands, and clayey sands with trace gravel extending to approximate elevations of +160 feet in the southwest portion of the site to +155 feet in the northeast portion of the site. These sandy soils were generally dense to very dense and moist, with moisture contents ranging from 6.5 to 12.2 percent, roughly near the optimum moisture content of 10 percent. The sandy soils exhibit moderate to high strength and low compressibility characteristics.

The underlying bedrock consisted of claystone and siltstone of the Monterey Formation. The bedrock nearer the ground surface and contact with the terrace deposits was weathered, becoming less weathered with depth. The moisture contents of the bedrock materials were generally wet and highly variable, ranging from 25 percent to 94 percent, with an average value of roughly 55 percent within the upper 35 feet of the subsurface profile. Sampler blow counts indicate that the bedrock material is stiff to hard, increasing in stiffness with depth, using soil consistency terminology instead of rock terminology. Atterberg limit testing indicates that the claystone is highly plastic. Although not tested, the bedrock materials are anticipated to exhibit a moderate to high potential for expansion when processed and recompacted and would be expected to shrink and swell with changes in moisture content. The bedrock materials exhibit moderate strength and compressibility characteristics.

3.4 GROUNDWATER AND CAVING

Groundwater was not encountered in our exploratory borings within the 46-foot depth explored. Groundwater was measured at depths of 27 to 50 feet in our CPT’s, corresponding to approximate elevations of +149 feet in the east to +114 feet in the west. During a prior investigation by others (Group Delta, 2015), a stabilized groundwater depth of 23 feet was measured in one of the exploratory borings.

Historical data provided by the California Geologic Survey (CGS) does not provide a clear indication of the shallowest groundwater depths in the site vicinity. The nearest groundwater level contour is located less than ½ mile to the west and indicates a shallowest depth to groundwater of 10 feet (CGS, 1997). However, ground surface elevations along this contour are, in general, at least 100 feet below the elevations across the subject site. As such, historical records may indicate shallowest groundwater depths in excess of 100 feet below the prevailing site grades.

Based on the lack of clear historical records and the variability in groundwater depths during our investigation, it is likely that the groundwater encountered in our explorations is the result of perched water conditions within the bedrock.

Caving was not encountered in our 8-inch diameter hollow-stem auger borings. Based on the fines and moisture contents of the soils encountered, the caving potential of the upper soils is considered to be low to moderate.

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 structure 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 structure are as follows:

- The undocumented fill and upper natural soils are not considered to be suitable for uniform support of new foundations or floor slabs. We recommend the existing fill and upper natural soils be removed and replaced as properly compacted fill. Details are presented in the “Earthwork” section of this report.

- There is a potential for differential settlement of footings supported at-grade and at the planned subterranean level as well as where columns with significantly different loads are located near each other. We recommend measures be taken to reduce the adverse effects of differential settlement at the transition from at-grade to below-grade foundations, such as creating joints or relief in the building exterior and flooring at the transition. We understand that a mat foundation is being considered for the subterranean portion of the building to better limit the differential settlements.

- Moisture contents of the near surface sandy soils (within 10 to 15 feet of the existing grades) are near or slightly below optimum, ranging from approximately 6 to 12 percent. The underlying claystone bedrock exhibited very high moisture contents within the upper 35 feet. Based on the provided grading plans, the claystone bedrock materials will likely not be exposed during earthwork activities and, as such, moisture conditioning (drying) of these materials will likely not be required.

- The upper sandy terrace deposits have a very low Expansion Index. Based on Atterberg limits testing, the claystone bedrock materials encountered at depth are anticipated to have moderate to high potential for expansion when processed and recompacted. As such, these materials will likely shrink and swell with changes in moisture content. These materials should not be used as compacted fill within 2 feet of finished grade in floor slab and hardscape areas.

- Retaining wall backfill should consist of granular, non-expansive fills. Based on our findings, such material is anticipated to be available on-site in significant quantities within the upper 5 to 15 feet below existing grades. The claystone bedrock materials encountered at depth should not be used as retaining wall backfill.

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 of Southern California and is likely to be subjected to strong ground shaking due to earthquakes on nearby faults.

We assume the seismic design of the proposed development will be in accordance with the 2016 California Building Code (CBC) criteria. For the 2016 CBC, a Site Class C may be used. The remaining seismic code values can be determined by the Project Structural Engineer using the value above and the pertinent United States Geological Survey (USGS) website and tables from the building code. Using the USGS website, the corresponding seismic design parameters from the CBC are as follows:

$S_S = 1.670g$	$S_{MS} = F_a * S_S = 1.670g$	$S_{DS} = 2/3 * S_{MS} = 1.114g$
$S_1 = 0.611g$	$S_{M1} = F_V * S_1 = 0.917g$	$S_{D1} = 2/3 * S_{M1} = 0.611g$

We can provide seismic parameters for the 2019 CBC if requested.

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 Newport-Inglewood Fault, which is located about 2.9 miles from the site.

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 0.68g for a magnitude 6.7 earthquake (Newport-Inglewood Fault). 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

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 shown as having a potential for soil liquefaction in accordance with the Seismic Hazards Mapping Act as shown in the Newport Beach Quadrangle (CGS, 1998). Groundwater was encountered as shallow as 27 feet below grade during our recent investigation and as shallow as 23 feet below existing grades in a prior investigation by others (Group Delta, 2015). Historical data for the site is inconclusive but based on nearby contours and regional topography, it is estimated that shallowest groundwater depths are greater than 50 feet below prevailing site grades. As such, the groundwater encountered in the recent explorations is anticipated to be the results of perched water conditions. In addition to the above, the material below elevations of +155 feet consisted of claystone bedrock, which is considered to be non-liquefiable. Excluding the site from a liquefaction hazard zone appears to be based primarily on the lack of shallow groundwater and on the near-surface presence of bedrock materials.

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.

The upper granular soils encountered in our explorations are predominantly dense to very dense sands, clayey sands, and silty sands. Based on our analyses, we computed a total potential seismic-induced subsidence of less than ¼-inch. Differential seismic settlement is estimated to be less than ¼-inch across a 40-foot span. As such, we consider the potential for seismic induced ground subsidence to adversely affect the planned project to be very low.

4.3 EARTHWORK

The earthwork anticipated at the project site will consist of clearing and grubbing, excavation of undocumented fills, loose natural soils and disturbed soils, excavations for basements, subgrade preparation, and the placement and compaction of fill.

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. Inert demolition debris, such as concrete and asphalt may be crushed for reuse in engineered fills, in accordance with the criteria presented in the “Materials for Fill” section of this report.

Although not encountered during our investigation, leach lines, cesspools or septic systems encountered 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 GPI representative should observe and accept the site prior to further grading.

4.3.2 Excavations

Excavations at this site will include the subterranean excavation, removal of undocumented fill soils and upper loose natural soils, foundation excavations, and trenching for proposed utility lines.

Prior to placing fills or construction of the proposed building, undocumented fill and loose natural soils occurring within the proposed building pad area should be removed and replaced as properly compacted fill. For planning purposes, removals for the at-grade portion of the building should extend to depths of at least 4 feet below existing grades or 2 feet below the base of the planned foundations, whichever is deeper. For the subterranean level, overexcavation of the undisturbed natural soils is not anticipated to be required (i.e., footings may be established in the undisturbed natural soils). Based on the provided plans, we anticipate that sufficient space is available for deep excavations to be accomplished using open cuts. If site access is limited, temporary shoring may be required for supporting the vertical sides of the required excavations.

For minor structures such as site walls, removals should extend at least 3 feet below the existing grade or 1 foot below the base of foundations, whichever is deeper. In proposed pavement and hardscape areas, the existing near-surface soils should be removed to a depth of 1-foot below existing grades or finished subgrade, whichever is deeper, and replaced as properly compacted fill. Remedial earthwork removals are not anticipated for the planned swimming pool if undisturbed natural soils are exposed in the excavation.

The actual depths of removals should be determined in the field during grading by a representative of GPI.

The corners of the areas to be overexcavated should be accurately staked in the field by the Project Surveyor. The base of the excavations should extend laterally at least 5 feet beyond the outside edge of the perimeter 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 and other foundation supported improvements, such as site walls and canopies.

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 all known utilities be shown on the grading plan.

Temporary construction excavations may be made vertically without shoring to a depth of 4 feet below adjacent grade for the existing natural soils. For cuts up to 12 feet deep, the entire cut should be properly shored or sloped back to at least 1:1 or flatter. For cuts up to 18 feet, the entire slope should be properly shored or sloped back at least 1¼:1 (horizontal to vertical) or flatter. Some raveling of the sandy deposits should be anticipated at the slope inclinations recommended. 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, "ABC" slot cuts may be utilized instead of shoring. The slots should be no wider than 7 feet and no deeper than 8 feet and should be backfilled immediately to finish grade prior to excavation of the adjacent two slots on each side. Where 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 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 State of California Occupational Safety and Health Standards.

4.3.3 Subgrade Preparation

After the recommended removals are complete, the exposed subgrade soils should be scarified to a depth of 12 inches, moisture-conditioned, and compacted to at least 90 percent of the maximum dry density in accordance with ASTM D 1557. If encountered, subgrade processing should be omitted in the bedrock materials and in areas where very moist to wet soils are exposed. The exposed subgrade and footings for the subterranean portion of the building should be moisture conditioned and compacted to at least 90 percent prior to covering.

4.3.4 Material for Fill

The on-site soils are, in general, suitable for use as compacted fill. Although not anticipated, clay soils should not be placed as properly compacted fill within 2 feet of the finished grade in floor slab and hardscape areas or used as retaining wall backfill. Soils placed as retaining wall backfill should be granular and non-expansive. Such soils are anticipated to be available within the required excavations.

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 25 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 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.

If open graded gravel is placed as backfill, we recommend that the gravel be placed in lifts and densified. The gravel should also be separated from the adjacent soil with a suitable filter fabric, such as Mirafi 140N or equivalent.

Both imported and existing on-site soils, to be used as fill, should be free of debris and pieces larger than 6 inches in greatest dimension.

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 within the upper 1-foot of the pavement subgrade 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.

The moisture contents of the existing near surface soils are near or slightly below optimum. The moisture content of the fill materials should be between 1 and 3 percent over optimum moisture conditions at the time of compaction. During backfill of excavations, the fill should be properly benched into the construction slopes as it is placed in lifts.

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 about 10 percent may be assumed for the near surface soils. Subsidence is expected to be less than 0.1 feet. 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 should be mechanically compacted in lifts. Lift thickness should not exceed those values given in the "Compacted Fill" section of this report. Jetting or flooding of backfill materials should not be permitted. A representative of GPI should observe and test trench and wall backfill 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. Slurry should also be used as backfill within the pipe zone for utilities that extend adjacent to and below building foundations. The slurry should contain 1½ sacks of cement per cubic yard and have a maximum slump of 5 inches.

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 tested prior to placement of additional fill.

4.4 FOUNDATIONS

4.4.1 Foundation Type

The proposed structures may be supported on conventional isolated and/or continuous shallow footings or a mat foundation, provided the subsurface soils are prepared in accordance with the recommendations given in this report. Footings for the at-grade structures should be supported on properly compacted fill. Footings or a mat foundation for the subterranean level may be established in the undisturbed natural soils or properly compacted fill. The soils exposed in the subterranean foundation excavations should be moisture conditioned and compacted to at least 90 percent prior to placement of rebar and concrete.

4.4.2 Spread Footings

Bearing Capacities

Based on the shear strength and elastic settlement characteristics of the natural and recompacted on-site soils, a static allowable net 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.

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	72	24
3,500	48	24
3,000	24	24
2,000	18	18
1,500	15	15

* Refers to minimum depth below lowest adjacent grade at the time of foundation construction.

A minimum footing width of 15 inches should be used even if the actual bearing pressure is less than 1,500 psf.

Estimated Settlements

Total static settlement of the more heavily loaded column foundations (800 kips) is expected to be on the order of 1- to 1¼-inch for footings established at the basement level and ½-inch for footings established at-grade. Maximum differential settlements between similarly loaded adjacent footings or along a 40-foot span of a continuous footing are expected to be on the order of ½- to ¾-inch. Potential seismic settlements should be added to these values when considering total settlements. As noted previously, seismic settlements are anticipated to be less than ¼ inch.

The following table presents anticipated static settlement values for near-grade and subterranean level footings and for various column loads.

COLUMN LOAD (kips)	SETTLEMENT (inch)	
	AT-GRADE FOOTINGS	SUBTERRANEAN FOOTINGS
120 (minimum)	¼ - ½	½
400 (average)	½	¾ - 1
800 (maximum)	½ - ¾	1 - 1¼

The differential settlement between the at-grade and subterranean supported portions of the building should be noted in designing settlement sensitive elements of the project, such as exterior facades and floor slabs. Structural joints/separations should be considered.

The above estimates assume that the recommended earthwork will be performed and that the footings will be sized in accordance with our recommendations.

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 weight of 275 pounds per cubic foot may be used, provided the footings are poured tight against undisturbed natural or compacted fill soils. These values may be used in combination without reduction.

4.4.3 Mat Foundation

We understand that a mat foundation is being considered for the subterranean portion of the building to limit differential settlements to within tolerable limits. The allowable bearing pressure for a mat foundation is generally not the governing geotechnical design issue as compared to the anticipated settlement. We have been provided with estimated mat foundation pressures for the shear wall cores ranging from 2,400 to 2,800 pounds per square foot.

For the elastic design of the mat foundation, a modulus of subgrade reaction (k-value) of 60 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 for the area of the mat foundation using appropriate elastic theory. Using generally accepted methods and our site-specific consolidation test results, we recommend using a value of 15 pci for the adjusted k-value in designing the mat foundation. The k-value may be increased by one-half for short-term, transient, wind and seismic loading. As previously discussed, we should be provided with the anticipated mat pressures when they are developed so that we can review and confirm the recommendations provided, as well as provide an estimate for the anticipated maximum static settlements for the mat foundations.

The allowable soil bearing pressure will be significantly greater than the average bearing pressures required for the mat foundation as discussed above. At localized areas of the mat, such as columns and point of load applications along exterior walls, a static allowable net bearing pressure of 4,000 pounds per square foot may be used. These allowable bearing pressures are for dead-plus-live loads, and may be increased one-third for short-term, transient, wind and seismic loading.

Based on a plot of the anticipated dead-plus-live load bearing pressures under the mat foundation provided by the Project Structural Engineer, we evaluated the resulting static settlements. Bearing pressures ranged from about 1,000 to 2,000 psf. We determined the resulting total static settlement of the mat to range from about ¾ inch at the perimeter to about 1¼ to 1½ inch at the center, with differential settlements of about ½ inch across spans of 40 feet.

4.4.4 Foundation Concrete

Prior testing of three samples by others at the site measured sulfate levels in the upper soils of less than 0.01 percent by weight for the upper sandy soils and 0.01 percent by weight for the deeper bedrock. In accordance with the 2016 CBC, foundation concrete should conform to the requirements outlined to the requirements outlined in ACI 318, Section 19.3 for a negligible level of soluble sulfate exposure for soil (category S0). Chloride contents were also less than 0.01 percent by weight, which is considered to be low (category C1).

4.4.5 Footing Excavation Observation

Prior to placement of steel and concrete, a representative of GPI should observe and approve foundation excavations. Footing excavations should be moistened immediately prior to concrete placement.

4.5 BUILDING FLOOR SLABS

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

A moisture vapor retarder should be placed under slabs that are to be covered with moisture-sensitive floor coverings (wood, vinyl, tile, etc.). Currently, common practice is to use a 10 or 15 mil polyethylene product such as Stego Wrap for this purpose. Whether the concrete slab is placed directly on the vapor barrier or on a clean sand layer between the slab and vapor barrier is a decision for the Project Architect and General Contractor, as it is not a geotechnical issue. If covered by sand, the sand layer should be about 2 inches thick and contain less than 5 percent by weight passing the No. 200 sieve. Based on our explorations and laboratory testing, the soils at the site are not suitable for this purpose. The sand 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 as a result 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. A sand layer is not required beneath the vapor retarder, but we take no exception if one is provided.

It should be noted that the material used as a vapor barrier is only one of several factors affecting the prevention of moisture accumulation under floor coverings. Other factors include maintaining a low water to cement ratio for the concrete used for the floor slab, effective sealing of joints and edges (particularly at pipe penetrations), and protecting the sand layer immediately under the slab from collecting water, such as through slab openings prior to construction of the roof. Ultimately, the transmission of water vapor can be reduced but not stopped completely. The manufacturer of the floor coverings should be consulted for establishing acceptable criteria for the condition of floor surface prior to placing moisture-sensitive floor coverings.

For lateral resistance design, a coefficient of friction value of 0.30 between the native sandy soils and concrete may be used.

4.6 LATERAL EARTH PRESSURES

Based on information provided, subterranean walls are planned for the majority of the proposed structure. The following recommendations are provided for retaining walls less than 15 feet in height. We recommend that walls be backfilled with non-expansive (Expansion Index of 20 of less) granular (no more than 40 percent passing No. 200 U.S. standard sieve) soils. The on-site claystone bedrock materials are not suitable for this backfill, but the on-site sands and silty sands would be suitable.

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, 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 55 pounds per cubic foot should be used for drained <u>granular</u> backfill.

To account for seismic loads, an additional lateral earth pressure equal to 20 pcf (equivalent fluid pressure distribution) should be added to the above active pressure to result in a total lateral earth pressure of 55 pcf (active plus seismic). If walls are designed using the above at-rest pressure, total (static plus seismic) lateral earth pressure may be limited to that value.

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. For the subterranean level, this drain line should be collected in a sump capable of pumping the water to a suitable discharge facility.

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.

In addition to the above active pressure, if temporary shoring is planned to consist of soldier piles and lagging, an allowable passive value of 550 pcf, to a maximum of 5,500 psf, may be used. We should review shoring plans prior to construction.

4.7 CORROSIVITY

Resistivity testing of three representative samples of the on-site soils by others indicates that they are moderately corrosive to ferrous metals. We do not practice corrosion protection engineering. If corrosion protection recommendations are required, a corrosion engineer such as HDR should be consulted.

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. Long-term ponding of surface water should not be allowed on pavements. 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 slab.

4.9 EXTERIOR CONCRETE AND MASONRY FLATWORK

Exterior concrete and masonry flatwork should be supported on non-expansive, compacted fill. Prior to placement of concrete, the subgrade should be prepared as recommended in "Subgrade Preparation" section. The use of clayey soils in the slab-subgrade should not be permitted.

4.10 PAVED AREAS

Based on the soils encountered, pavement design has been based on an assumed R-value of 25, which is consistent with the upper sandy soils encountered. R-value testing should be performed prior to construction of the pavement sections to confirm the preliminary design. The California Division of Highways Design Method was used for design of the recommended preliminary pavement sections. These recommendations are based on the assumption that the pavement subgrades will consist of existing near surface soils. The following pavement sections are recommended:

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	7.0
Truck Drives	6.0	3.5	9.0
Portland Cement Concrete			
Automobile Parking	4.0	6.0	4.0
Automobile Drives	5.0	6.5	4.0
Truck Drives	6.0	7.0	4.0

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).

If the site is base paved prior to the start of building construction, the above pavement sections should be re-evaluated based on the anticipated construction traffic loads. A significant pavement design issue with base-paving a site before building construction is that localized areas, such as construction entry drives, staging areas, and delivery areas, will experience significantly higher construction traffic loads than the typical design traffic loads during the life of the project. As such, the asphalt pavement sections should be designed for higher Traffic Indexes, where impacted by construction traffic. Identifying areas of increased traffic loads requires input from the General Contractor, and is beyond the scope of our services. As an alternative, asphalt pavement areas can be designed using the higher construction traffic loads.

We recommend the following pavement sections for the project if base paving prior to building construction is planned:

PAVEMENT AREA	TYPICAL TRAFFIC INDEX	SECTION THICKNESS (inches)	
		ASPHALT/PORTLAND CONCRETE	AGGREGATE BASE COURSE
Asphalt Concrete			
Automobile Parking	4.0	4.0	5.0
Automobile Drives	5.0	4.0	7.0
Truck Drives	6.0	4.5	9.0

In areas where very high impact construction traffic loads are planned, such as the main construction entrance/exit and locations for concrete truck delivery, the above asphalt concrete section may still experience surface distress (stops and starts or sharp turning of heavily loads vehicles will tend to shove and tear the base-paved asphalt concrete). In these localized areas, an allowance should be made to remove and replace the asphalt concrete at the completion of building construction.

The pavement subgrade underlying the aggregate base should be properly prepared and compacted in accordance with the recommendations outlined under "Subgrade Preparation".

The pavement base course should be compacted to at least 95 percent of the maximum dry 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 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 GEOTECHNICAL OBSERVATION AND TESTING

We recommend that a representative of GPI observe the 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, pavement construction and foundation excavations. If conditions are different than expected, we should be afforded the opportunity to provide an alternate 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 Nexus Companies and their consultants in designing the proposed development. 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 all 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

Paul R. Schade, G.E.
Principal

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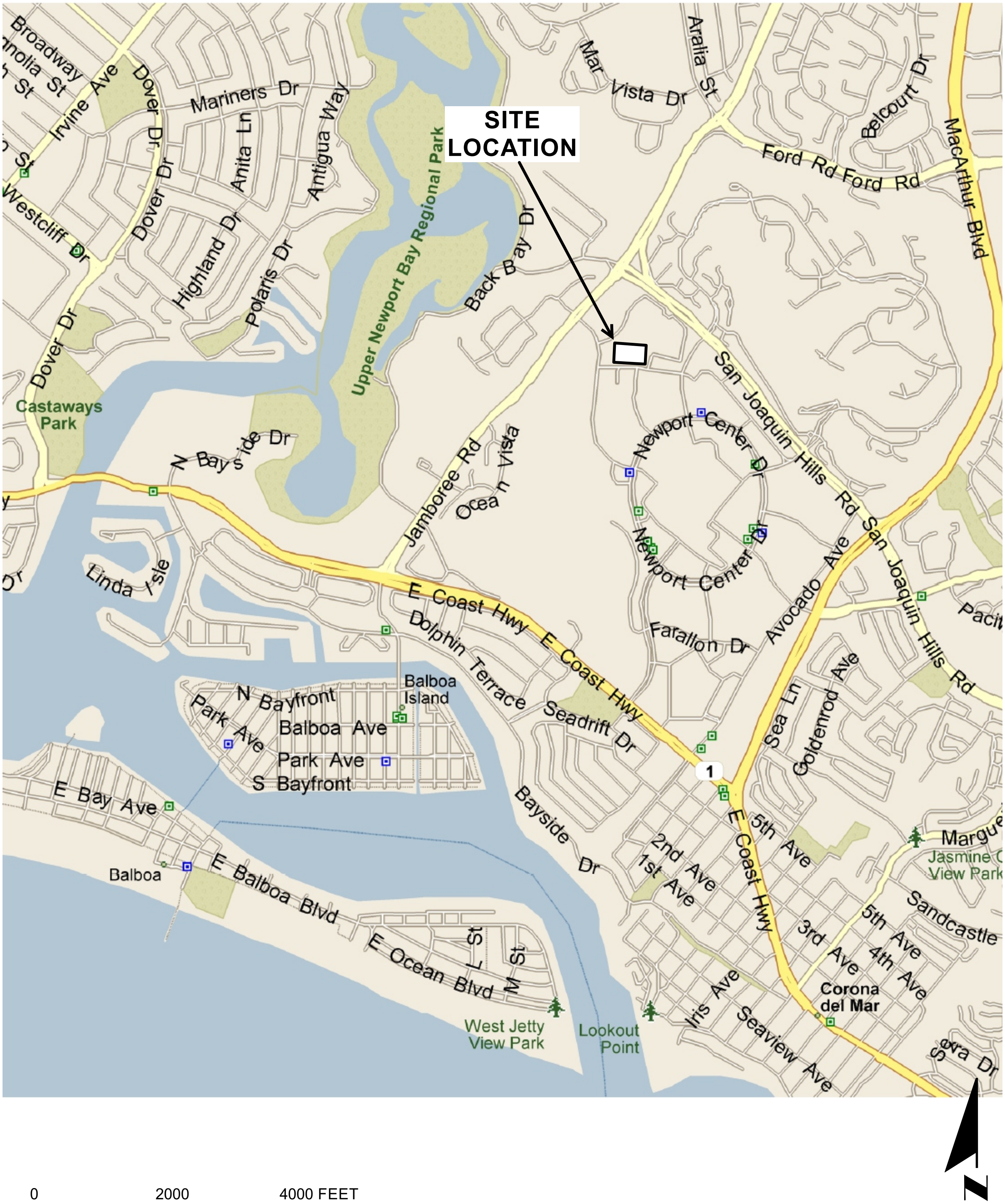

BASE MAP REPRODUCED FROM MICROSOFT STREETS AND TRIPS (C. 2008)

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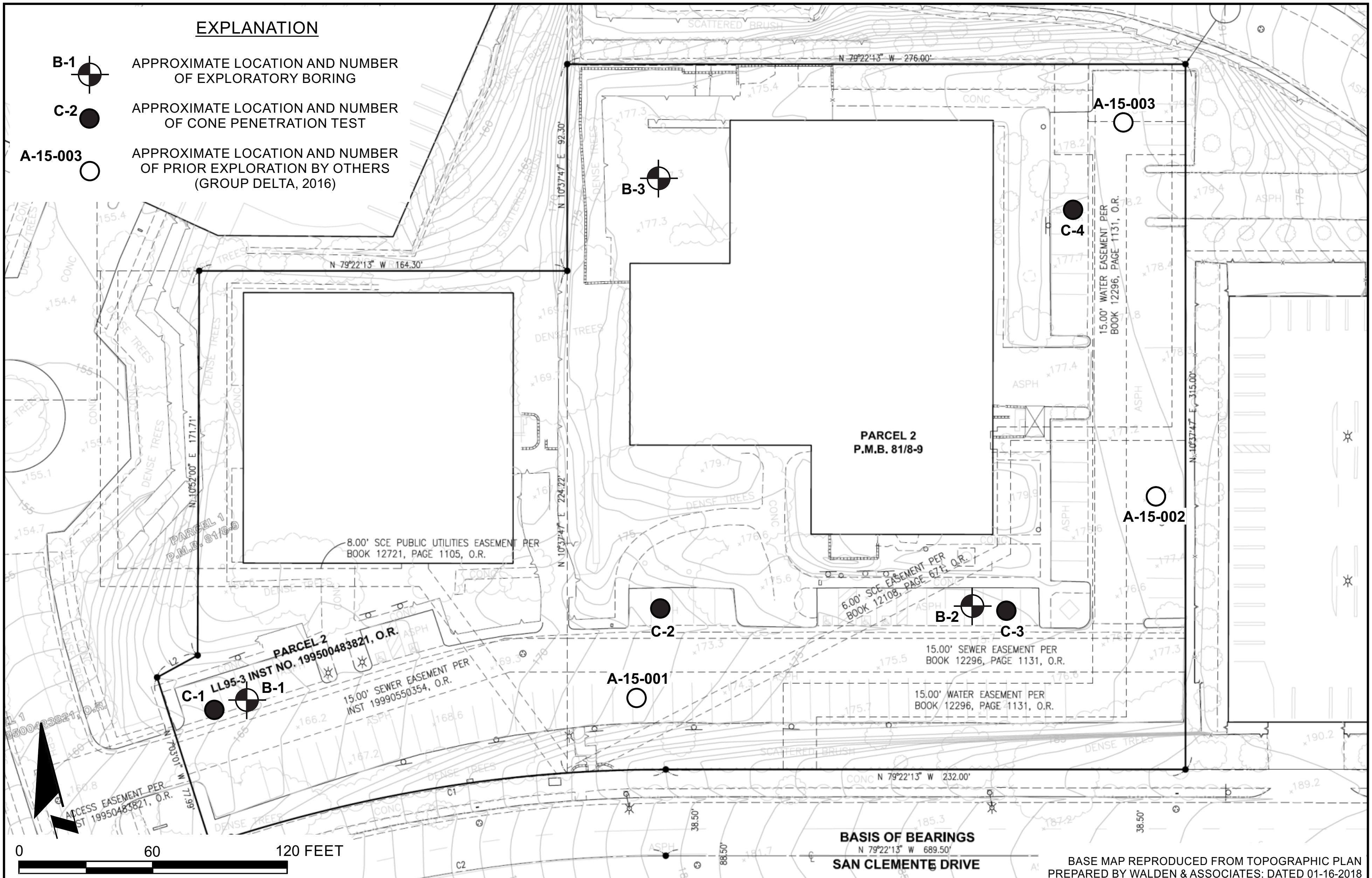
SITE LOCATION MAP

FIGURE 1



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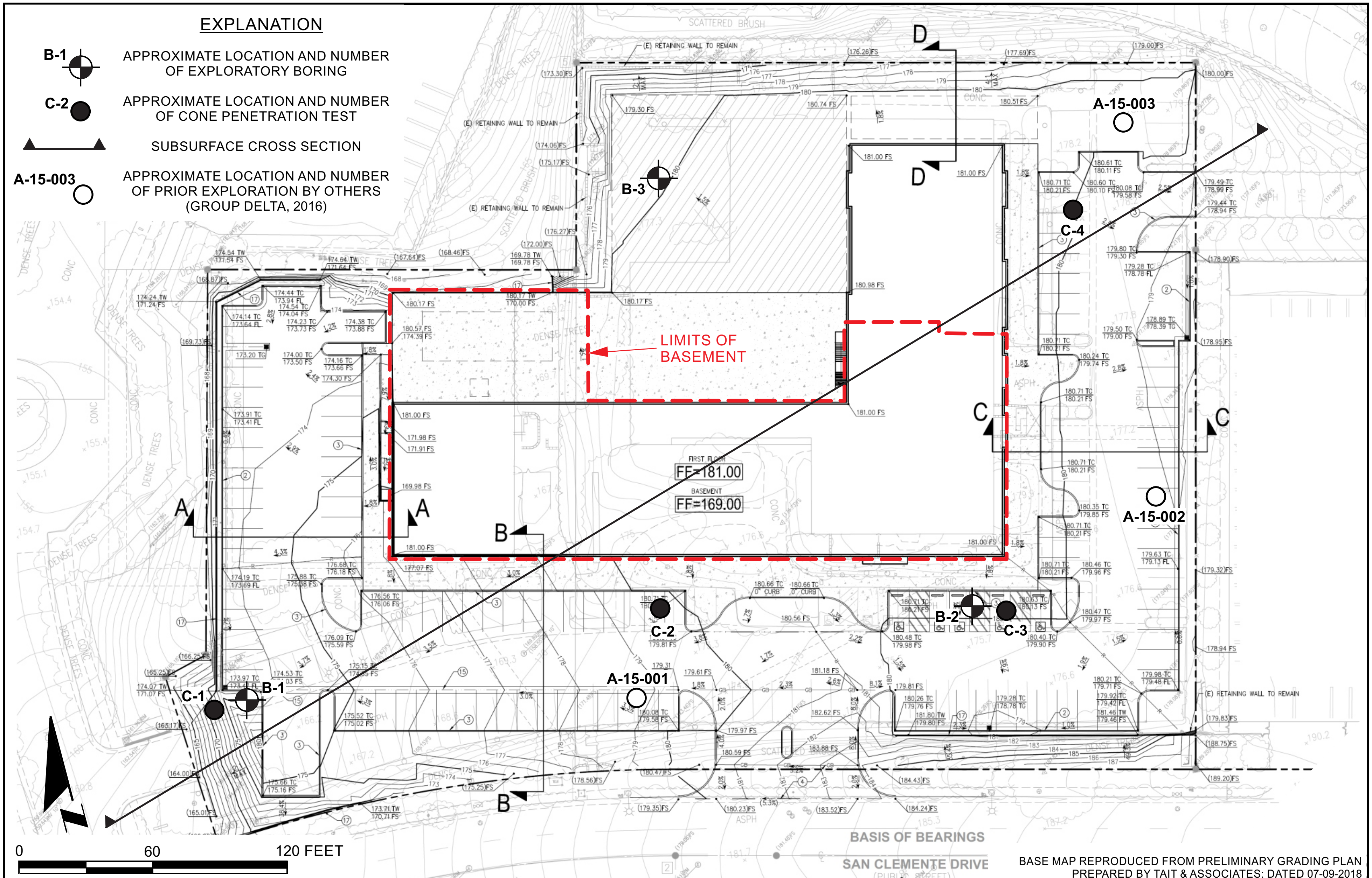
GPI PROJECT NO.: 2870.I | SCALE: 1" = 60'

EXISTING SITE PLAN

FIGURE 2



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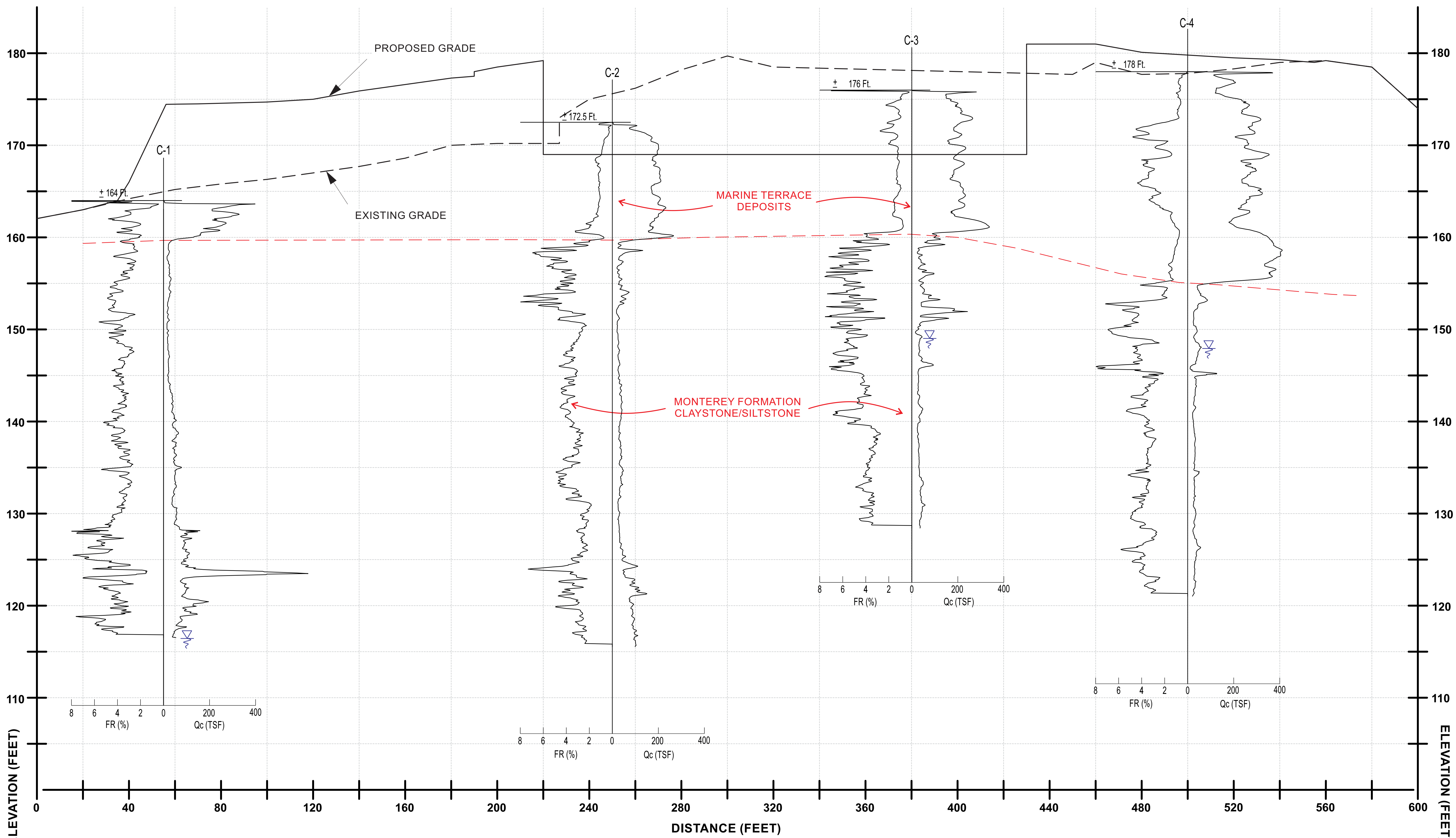
SCALE: 1" = 60'

PROPOSED SITE PLAN

FIGURE 3



Note: This section is based upon information obtained at borings and CPTs obtained during geotechnical investigation. The section is based upon limited geotechnical data and localized variations should be anticipated. This section is intended for descriptive purposes only.

HORIZONTAL: 1" = 40'
VERTICAL: 1" = 10'

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VIVANTE NEWPORT COAST

GPI PROJECT NO.: 2870.I | SCALE AS SHOWN

SUBSURFACE CROSS SECTION

FIGURE 4

APPENDIX A

APPENDIX A

CONE PENETRATION TESTS

The subsurface conditions were investigated by performing four Cone Penetration Tests (CPT's) at the site. The soundings were advanced to depths ranging from approximately 50 to 60 feet below existing grades. The locations of the CPT's are shown on the Existing and Proposed Site Plans, Figures 2 and 3.

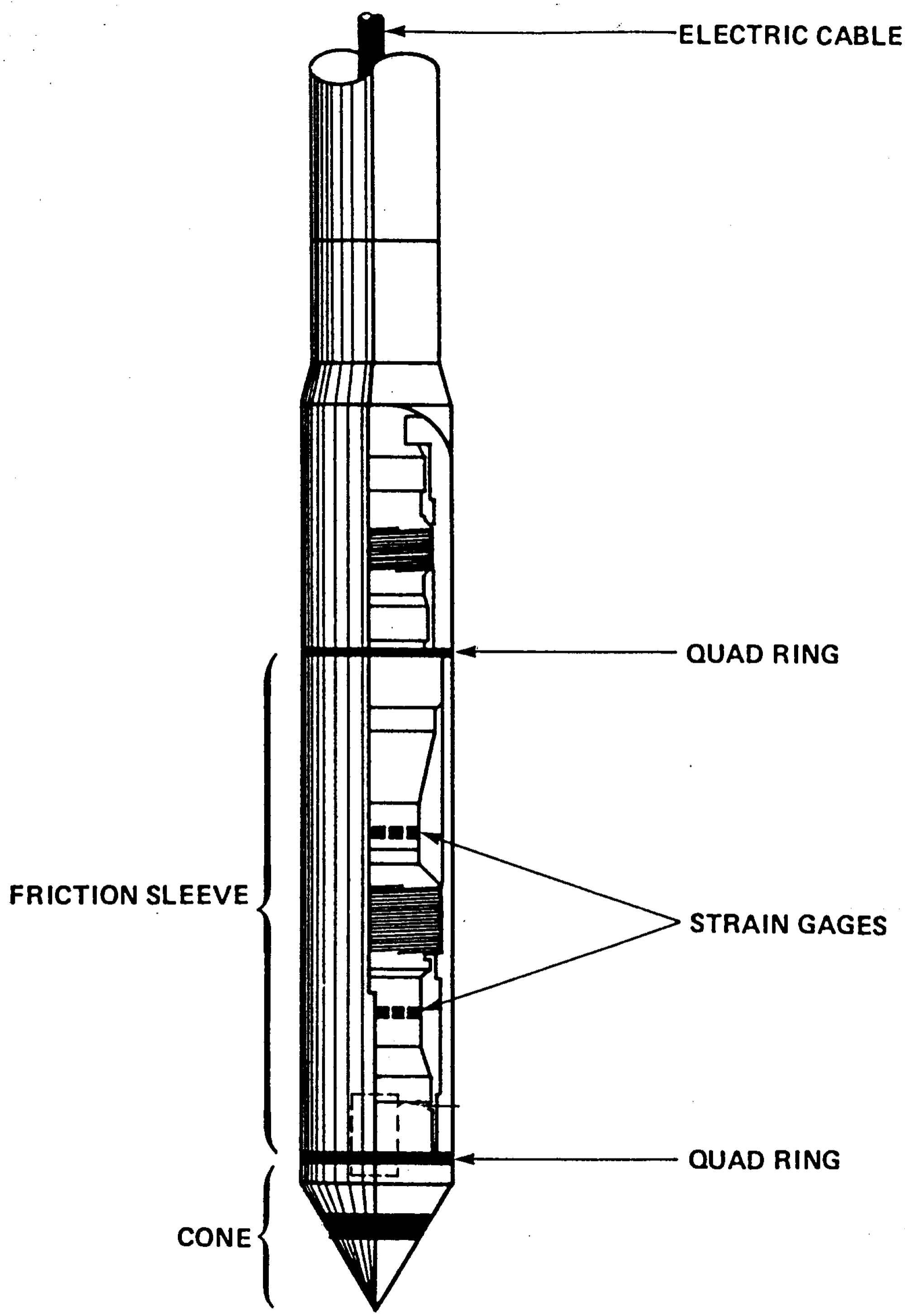
The Cone Penetration Test consists of pushing a cone-tipped probe into the soil deposit while simultaneously recording the cone tip resistance and side friction resistance of the soil to penetration (refer to Figure A-1). The CPT's described in this report were conducted in general accordance with ASTM specifications (ASTM D 5778) using an electric cone penetrometer.

The CPT equipment consists of a cone assembly mounted at the end of a series of hollow sounding rods. A set of hydraulic rams is used to push the cone and rods into the soil while a continuous record of cone and friction resistance versus depth is obtained in both analog and digital form at the ground surface. A specially designed truck is used to transport and house the test equipment and to provide a 30-ton reaction to the thrust of the hydraulic rams.

Standard data obtained during a CPT consists of continuous stratigraphic information with close vertical resolution. Stratigraphic interpretation is based on relationships between cone tip resistance and friction resistance. The calculated friction ratio (CPT friction sleeve resistance divided by cone tip resistance) is used as an indicator of soil type. Granular soils typically have low friction ratios and high cone resistance, while cohesive or organic soils have high friction ratios and low cone resistance. These stratigraphic material categories form the basis for all subsequent calculations which utilize the CPT data.

Computer plots of the reduced CPT data acquired for this investigation is presented in Figures A-2 to A-5 of this appendix. The field testing and computer processing was performed by Kehoe Testing and Engineering under subcontract to Geotechnical Professionals Inc. (GPI). The interpreted soil descriptions were prepared by GPI.

The CPT locations were laid out in the field by measuring from existing site features. Upon completion, the uncaved portions of the CPT holes were backfilled with bentonite chips. Ground surface elevations at the CPT locations were estimated from a topographic plan prepared by Walden & Associates, dated January 16, 2018.



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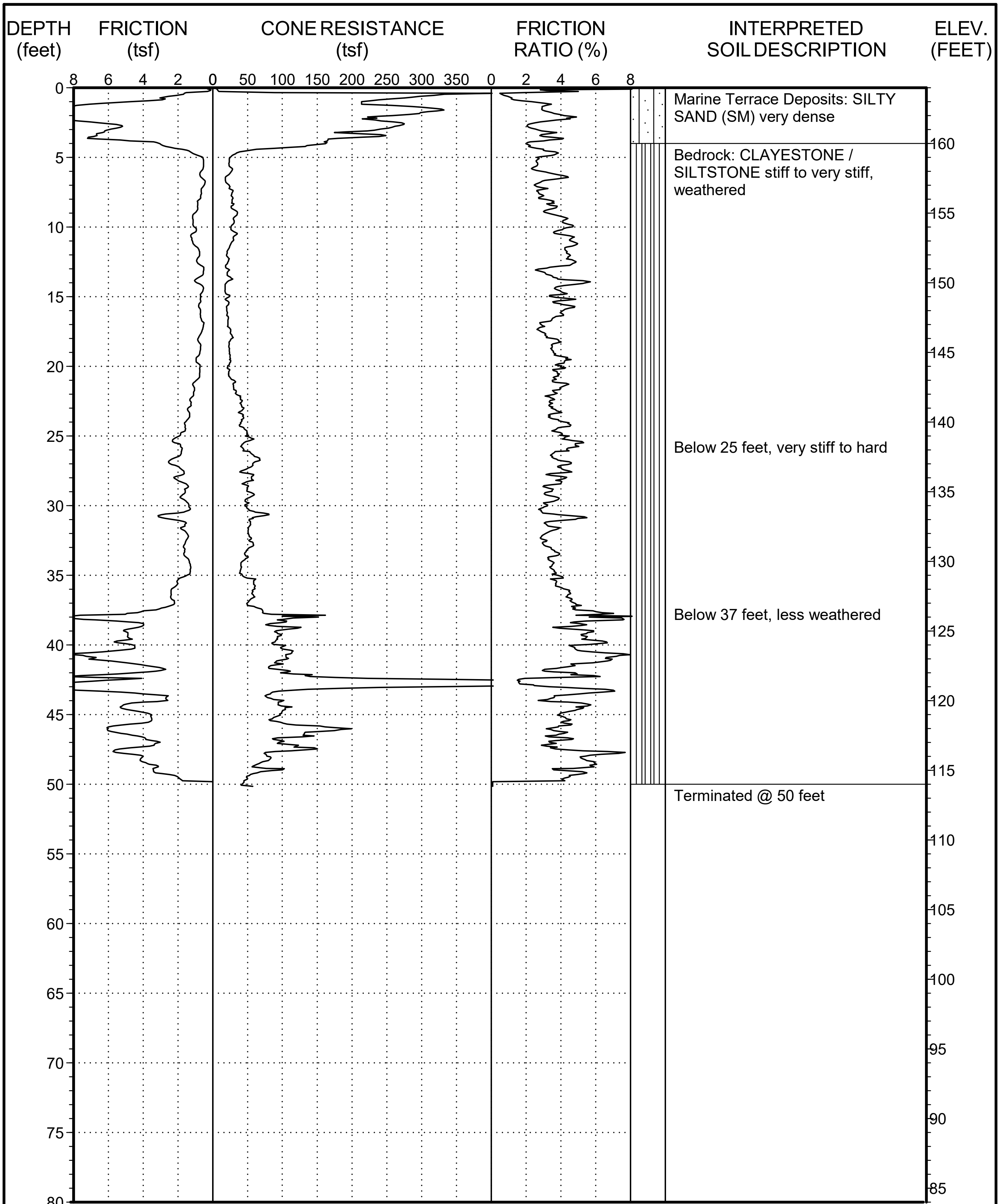
CONE PENETROMETER

FIGURE A-1



Date performed: 7-31-18

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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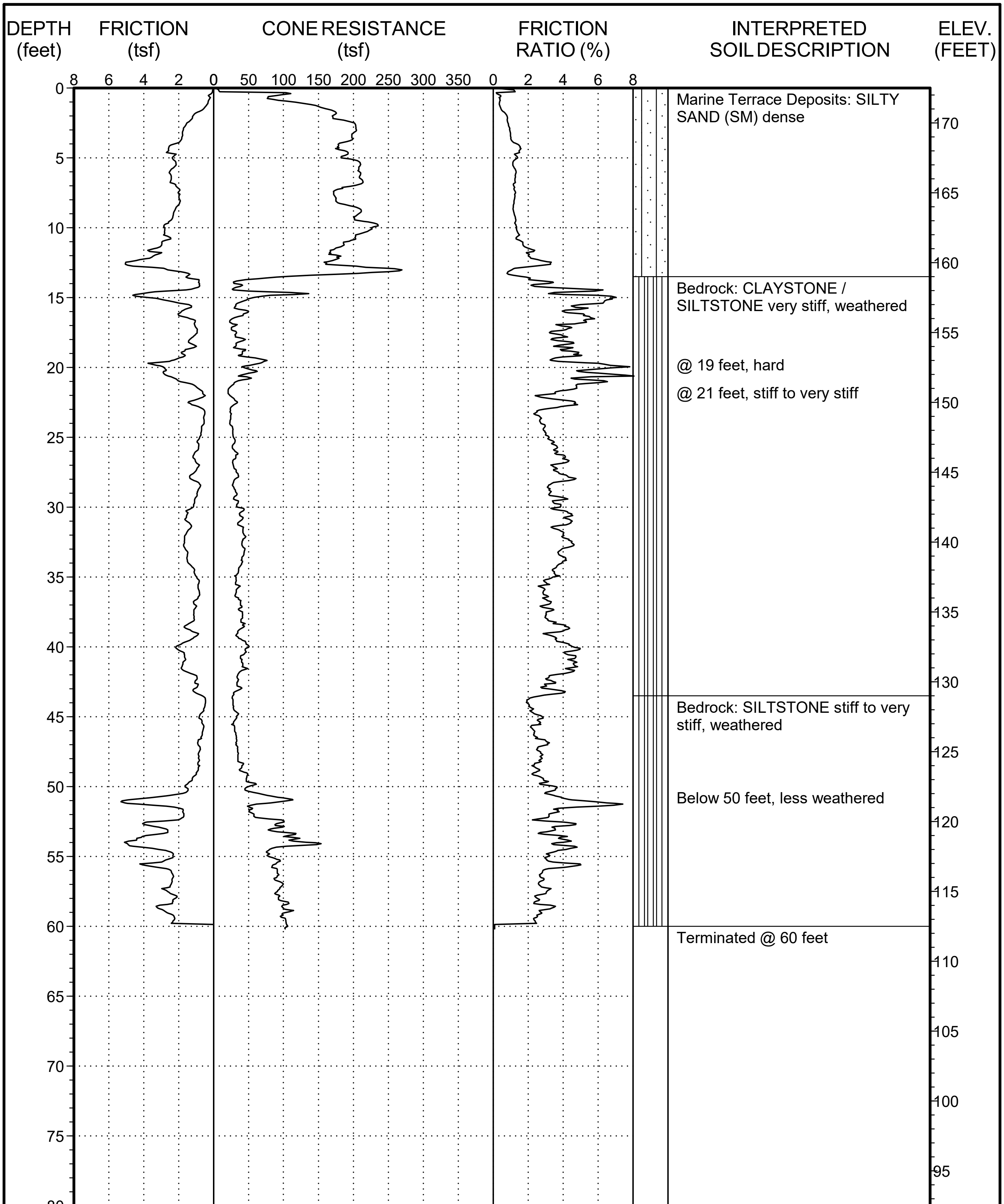
LOG OF CPT NO. C-1

FIGURE A-2



Date performed: 7-31-18

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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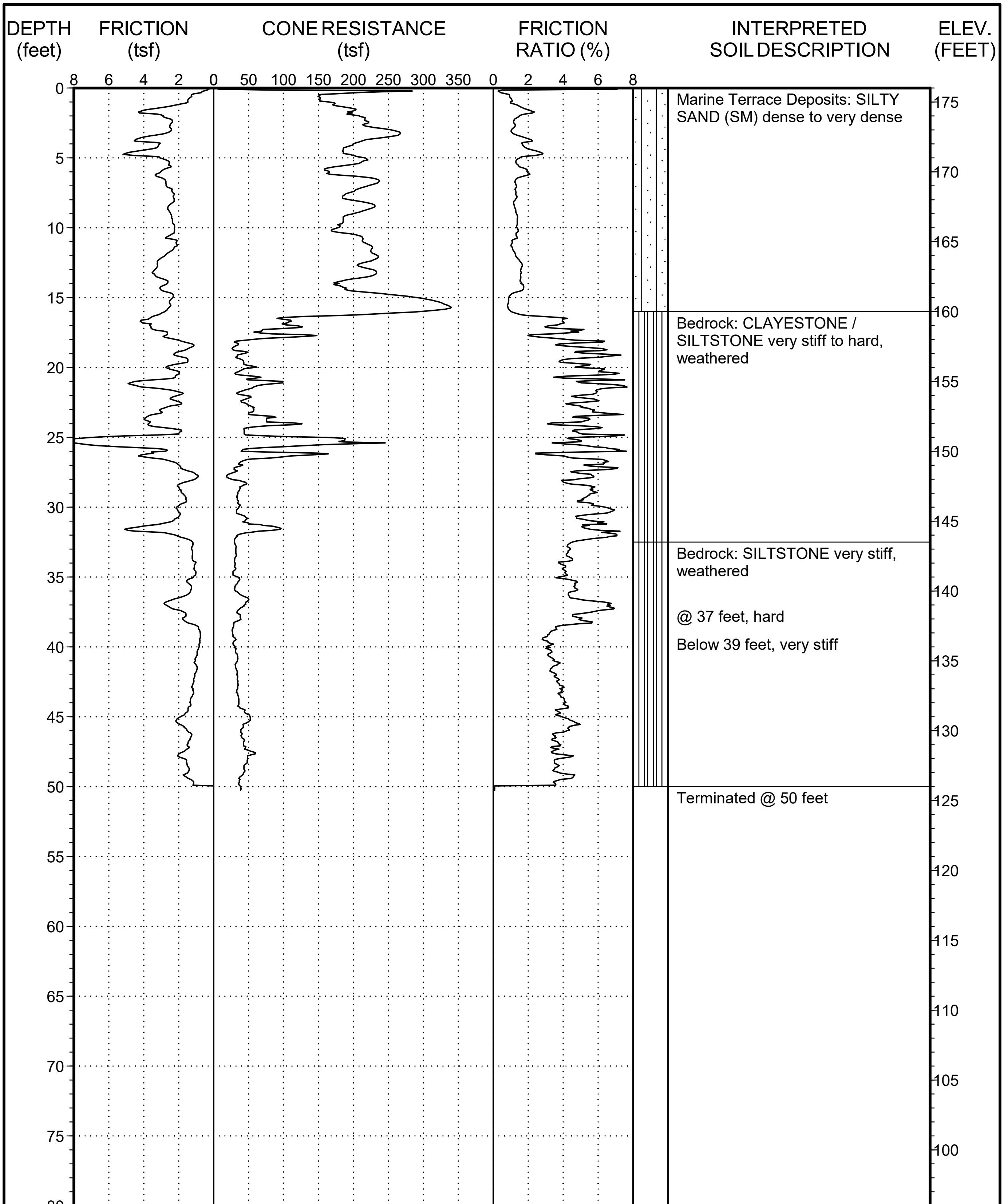
LOG OF CPT NO. C-2

FIGURE A-3



Date performed: 7-31-18

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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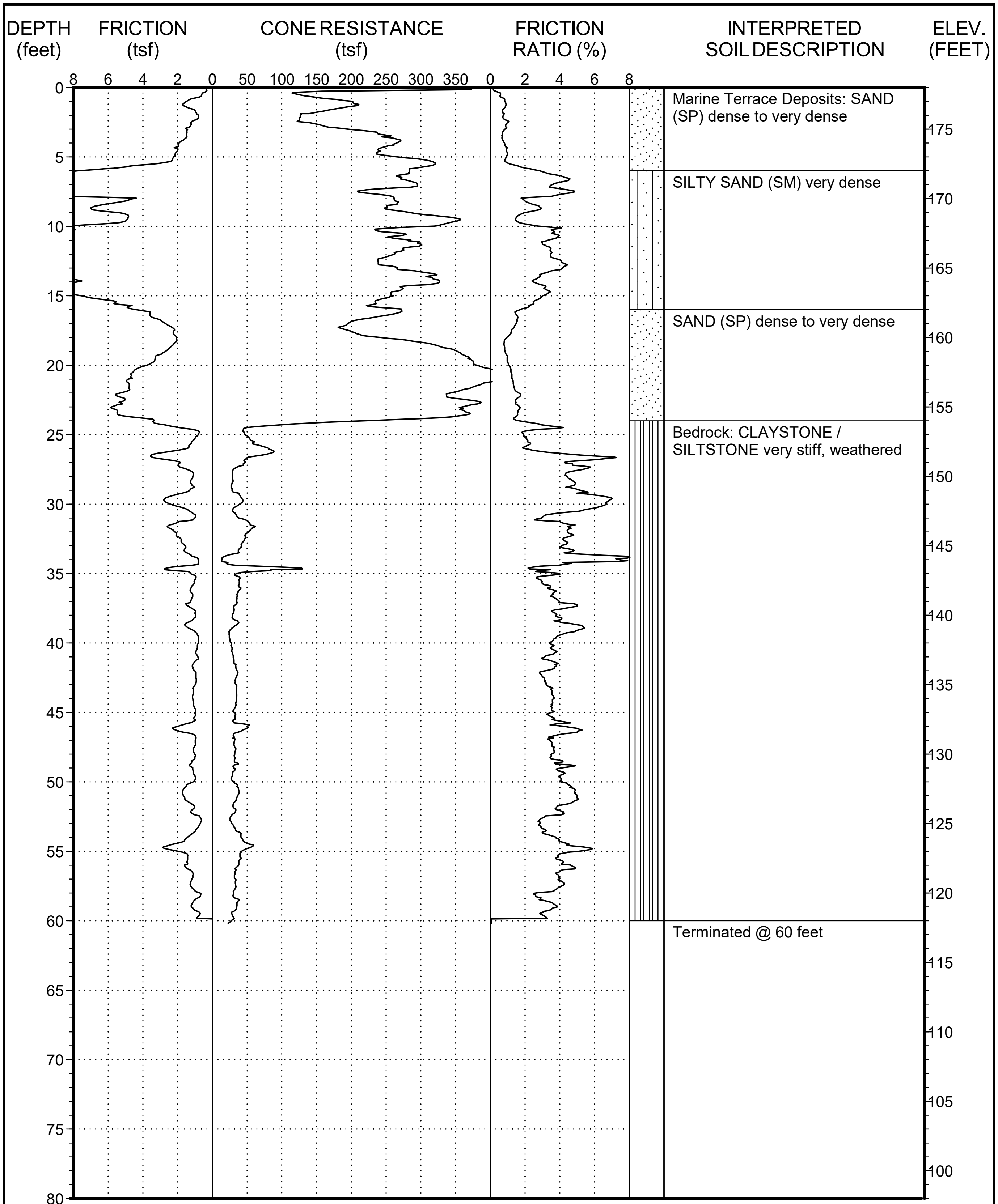
LOG OF CPT NO. C-3

FIGURE A-4



Date performed:7-31-18

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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VIVANTE NEWPORT COAST

LOG OF CPT NO. C-4

FIGURE A-5

APPENDIX B

APPENDIX B

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling two exploratory borings. The borings were advanced to depths ranging from 21 to 46 feet below the existing ground surface. The locations of the explorations are shown on the Existing and Proposed Site Plans, Figures 2 and 3.

The exploratory borings were drilled using limited access 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.

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 soils 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 B-1 to B-2 in this appendix.

The boring locations were laid out in the field by measuring from existing site features. Ground surface elevations at the exploration locations were estimated from a topographic plan prepared by Walden & Associates, dated January 16, 2018.

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	3-Inch AC over 6-Inch BASE	
					Fill: **CLAYEY SAND (SC)** reddish brown, moist, gravel	
6.5	109	81	D		Natural: **CLAYEY SAND (SC)** reddish brown, moist, very dense, marine terrace deposits	
			B			160
28.7	87	20	D	5		
					Bedrock: **Monterey Formation - CLAYSTONE** grey, wet, stiff	
48.9	68	17	D			
						155
50.4	69	20	D	10		
						150
55.4	67	23	D	15	@ 15 feet, very stiff	
						145
24.6	83	41	D	20		
						140
64.2		55	D	25	@ 25 feet, hard	
						135
88.2		47	D	30		
						130
76.1	51	58	D	35		
						125

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

DATE DRILLED: 8-9-18

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2870.I

VIVANTE NEWPORT COAST

LOG OF BORING NO. B-1

FIGURE B-1

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)
85.3	47	75/11"	D	40	Bedrock: **Monterey Formation - CLAYSTONE** grey, wet, hard	
						120
93.6	44	92/9"	D	45	@ 45 feet, brown	
					Total Depth 46 feet	

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

DATE DRILLED:
8-9-18
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2870.I
VIVANTE NEWPORT COAST

LOG OF BORING NO. B-1

FIGURE B-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
				0	4-Inch AC over 3-Inch BASE	175
					Fill: **SAND (SP)** reddish brown, moist, with clay, marine terrace deposits	
12.2	94	50	D	5	@ 5 feet, moist, dense, trace silt	
					Natural: **SAND with SILT (SP-SM)** light brown, moist, dense, marine terrace deposits	170
7.5	92	52	D	10		165
10.7	97	54	D	15	**SAND (SP)** light brown, moist to very moist, dense	160
67.3	55	20	D	20	Bedrock: **Monterey Formation - CLAYSTONE** grey, wet, stiff, weathered	155
					Total Depth 21 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:
8-9-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2870.I

VIVANTE NEWPORT COAST

LOG OF BORING NO. B-2

FIGURE B-2

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	5-inch PCC	
					Fill: **CLAYEY SAND (SC)** brown, moist	
					5-inch PCC	175
					CLAYEY SAND (SC) brown, moist	
12.3	98	18	D	5	Natural: **SILTY SAND (SM)** brown to light brown, moist, medium dense, marine terrace deposits	
						170
11.3	102	49	D	10	@ 10 feet, dense	
						165
13.0	92	37	D	15	@ 15 feet, light brown	
10.7	112	81/11"	D		**CLAYEY SAND (SC)** reddish brown, moist, very dense	160
10.2	107	66	D	20	@ 20 feet, dense	
					Bedrock: **Monterey Formation - CLAYSTONE** grey and green, wet, very stiff, weathered	155
99.4	43	44	D			
				25	@ 25 feet, hard	
108.4	41	55	D			
						150
98.8	46	32	D	30	@ 30 feet, light brown and white, very stiff	
						145
75.2	52	35	D	35		
						140

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

DATE DRILLED: 11-7-18

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2870.I

VIVANTE NEWPORT COAST

LOG OF BORING NO. B-3

FIGURE B-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)
	80.8	51	34	D	40		
							135
	91.0	46	51	D	45	@ 45 feet, hard, less weathered	
							130
	64.6	56	62	D	50		
						Total Depth 51 feet	

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

DATE DRILLED:
11-7-18
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2870.I
VIVANTE NEWPORT COAST

LOG OF BORING NO. B-3

FIGURE B-3

APPENDIX C

APPENDIX C

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 was determined from a number of the samples. The samples were weighed to determine 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 was calculated. Moisture content values are presented on the boring logs in Appendix B.

ATTERBERG LIMITS

Liquid and plastic limits were determined for selected samples in accordance with ASTM D4318. Results of the Atterberg Limits test are summarized on Figure C-1. The liquid limits and plasticity indices of samples tested during a prior investigation of the site (Group Delta, 2016) are presented in the following table:

GROUP DELTA BORING NO.	DEPTH (ft)	SOIL DESCRIPTION*	LIQUID LIMIT	PLASTICITY INDEX
A-15-001	30 - 31.5	Fat Clay (CH)	93	63
A-15-002	20 – 21.5	Silty Clay (CL)	96	37
A-15-002	25 – 26.5	Clayey Silt (ML-CL)	78	29
A-15-002	45 – 46.5	Silty Clay (CL)	93	54
A-15-003	25 – 26.5	Silty Clay (CL)	27	9

*Soil description by Group Delta, 2016

GRAIN SIZE DISTRIBUTION

Select 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. A summary of the percentages passing the No. 200 sieve is presented in the following table:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-1	2	Clayey Sand (SC)	17
B-2	15	Sand (SP)	3

CONSOLIDATION

One-dimensional consolidation tests were performed on undisturbed samples in accordance with ASTM D 2435. After trimming the ends, the samples were placed in the consolidometer and loaded to up to 0.4 ksf. Thereafter, the samples were incrementally loaded to a maximum load of up to 25.6 ksf. The samples were inundated at 1.6 ksf. Sample deformation was measured to 0.0001 inch. Rebound behavior was investigated by unloading the sample back to 0.4 ksf. Results of the consolidation tests, in the form of percent consolidation versus log pressure are presented in Figures C-2 and C-3.

DIRECT SHEAR

Direct shear tests were performed on undisturbed and remolded bulk samples in accordance with ASTM D 3080. The bulk samples were remolded to approximately 90 percent of maximum density (ASTM D1557). The samples were placed in the shear machine, and pre-selected normal loads were applied. The samples were submerged, allowed to consolidate, and then were sheared to failure. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear tests are presented in Figures C-4 to C-6.

EXPANSION INDEX

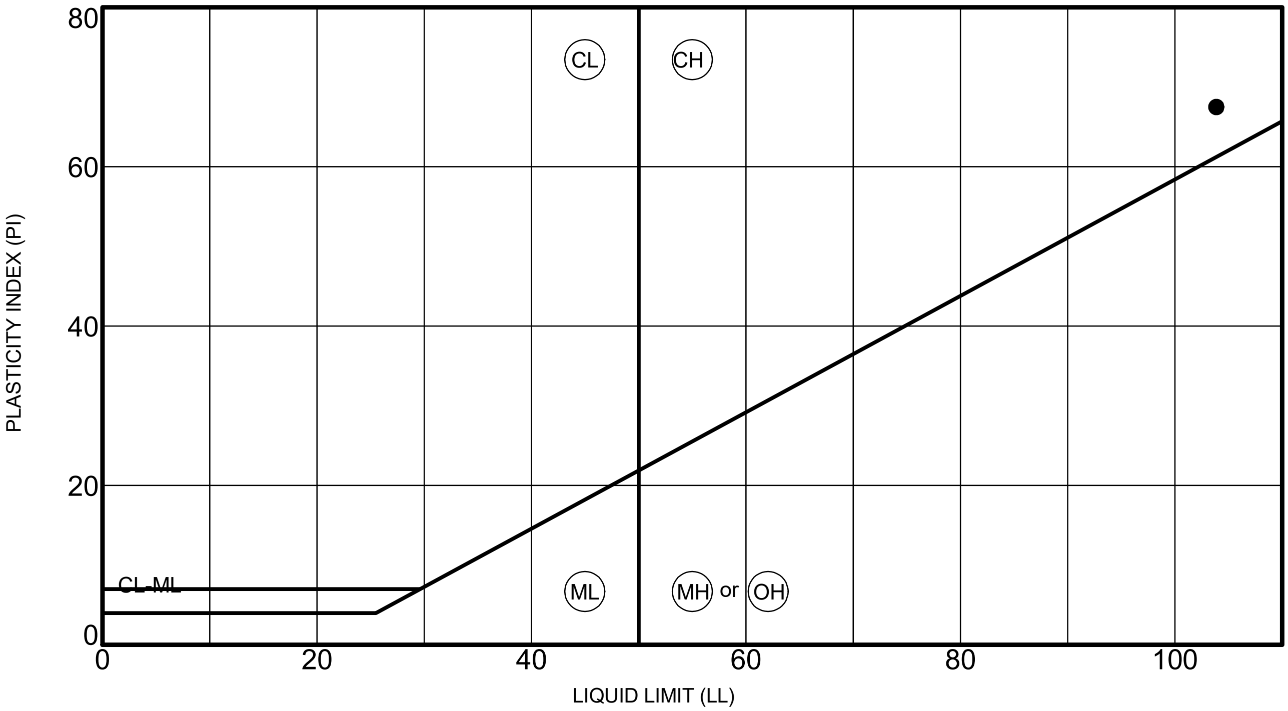
An expansion index test was performed on a bulk sample. The test was performed in accordance with ASTM 4829 to assess the expansion potential of the on-site soils. The results of the test are summarized below:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX
B-1	0 – 5	Clayey Sand (SC)	15

COMPACTION TEST

A maximum dry density/optimum moisture test was performed in accordance with ASTM D1557 on a representative bulk sample of the surficial soils. The test results are as follows.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)
B-1	0 – 5	Clayey Sand (SC) (no rock correction)	127	10.0
		Clayey Sand (SC) (w/ rock correction)	130	9.0



	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
●	B-1	7.0	104	36	67		Bedrock: CLAYSTONE

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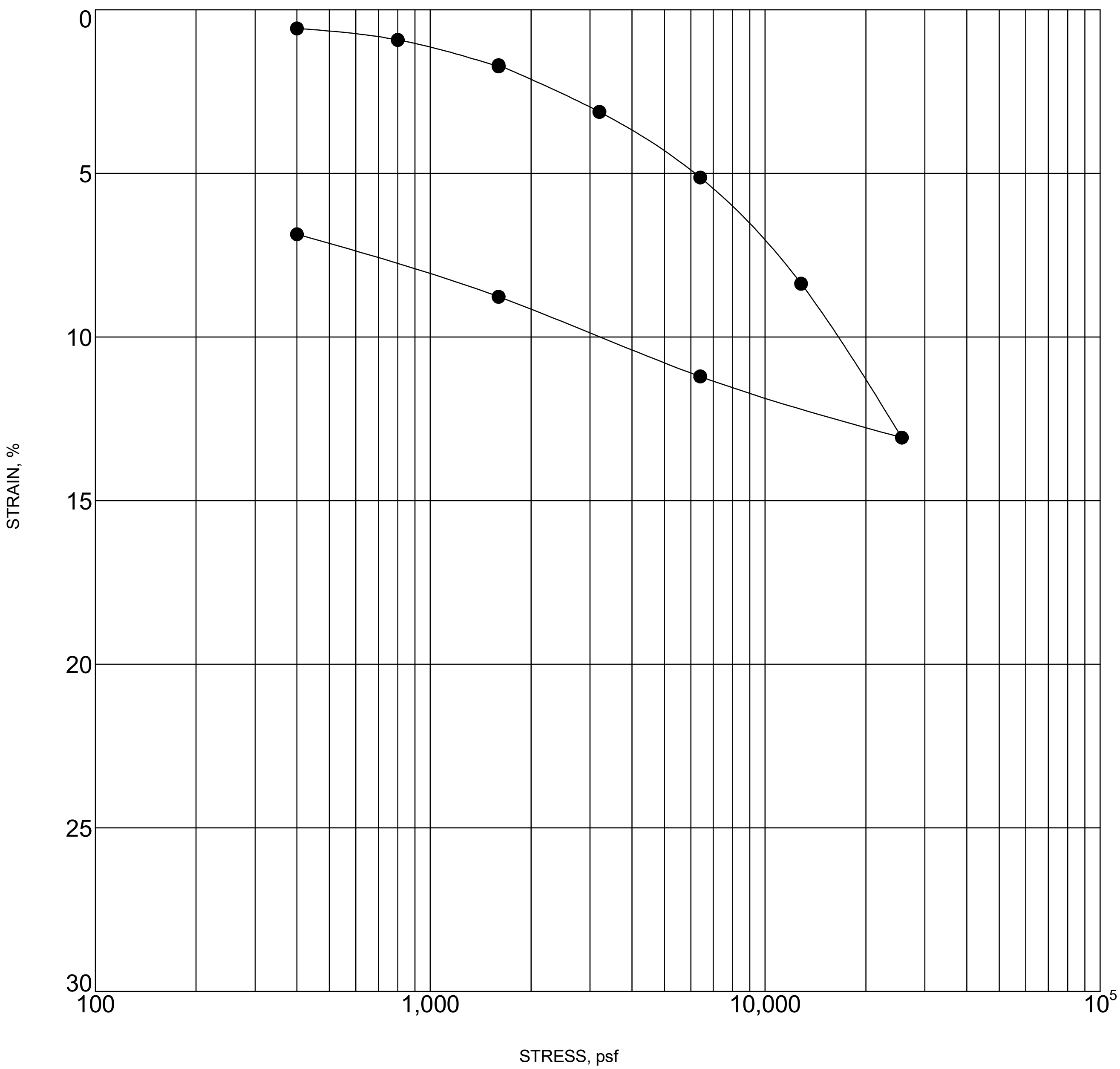
ATTERBERG LIMITS TEST RESULTS

FIGURE C-1



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-1	15.0	Bedrock: CLAYSTONE	67	55.4

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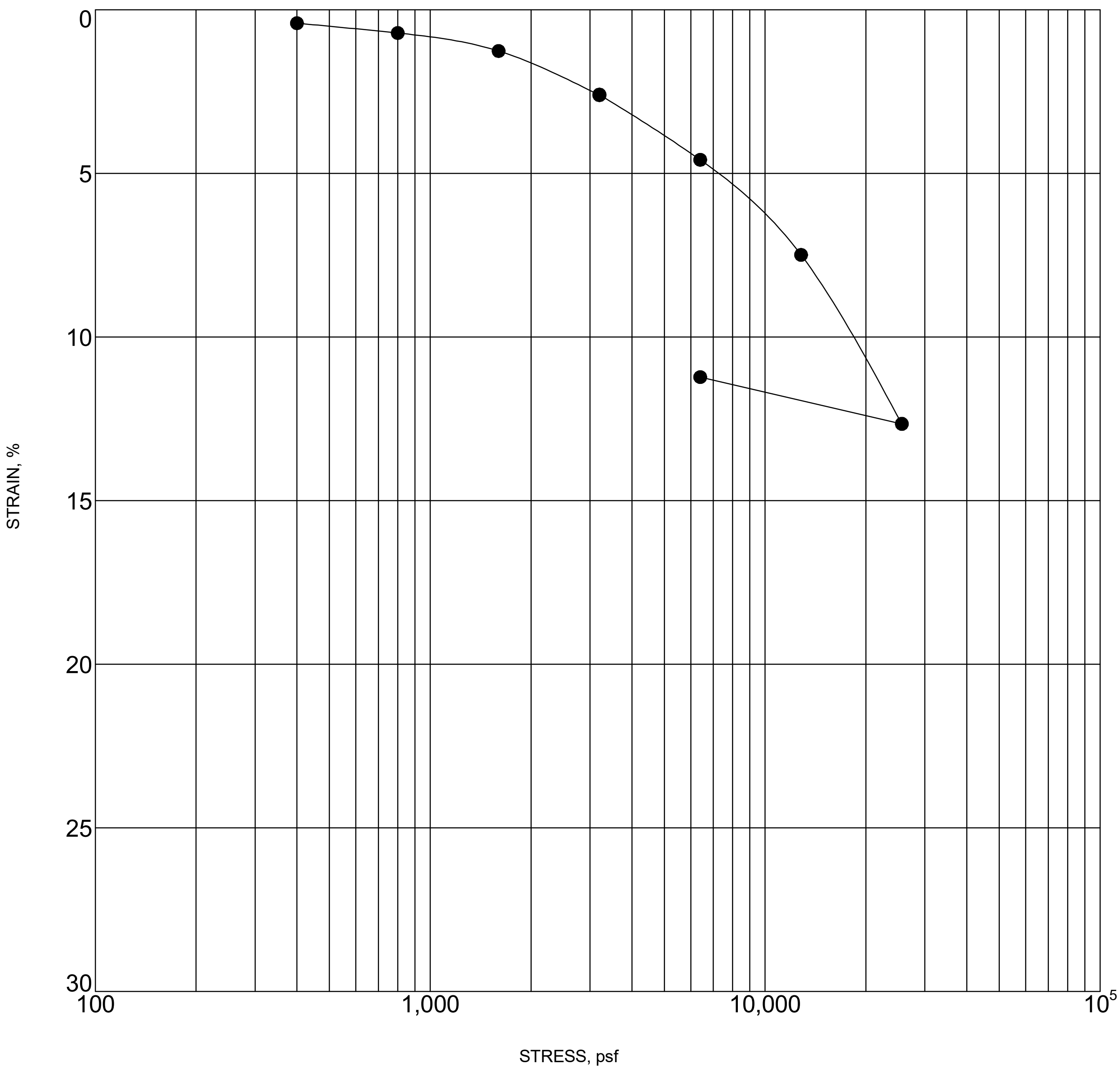
CONSOLIDATION TEST RESULTS

FIGURE C-2



Sample inundated at 3200 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-1	35.0	Bedrock: CLAYSTONE	51	76.1

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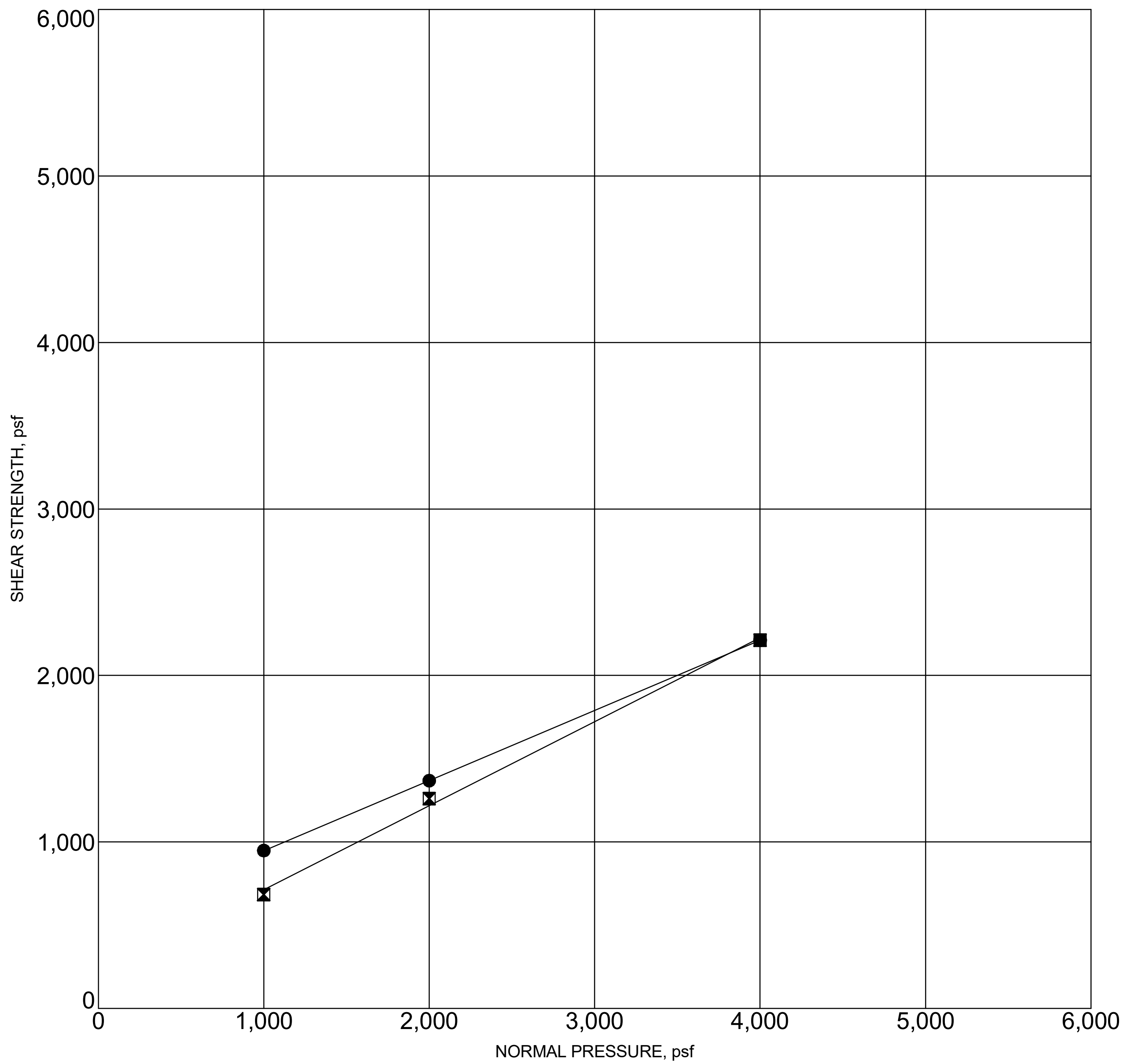
CONSOLIDATION TEST RESULTS

FIGURE C-3



● **PEAK STRENGTH**
Friction Angle= 23 degrees
Cohesion= 526 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 27 degrees
Cohesion= 208 psf

Note: Samples remolded to 90% maximum dry density

Sample Location		Classification	DD,pcf	MC,%
B-1	0-5	CLAYEY SAND (SC)	114	10.0

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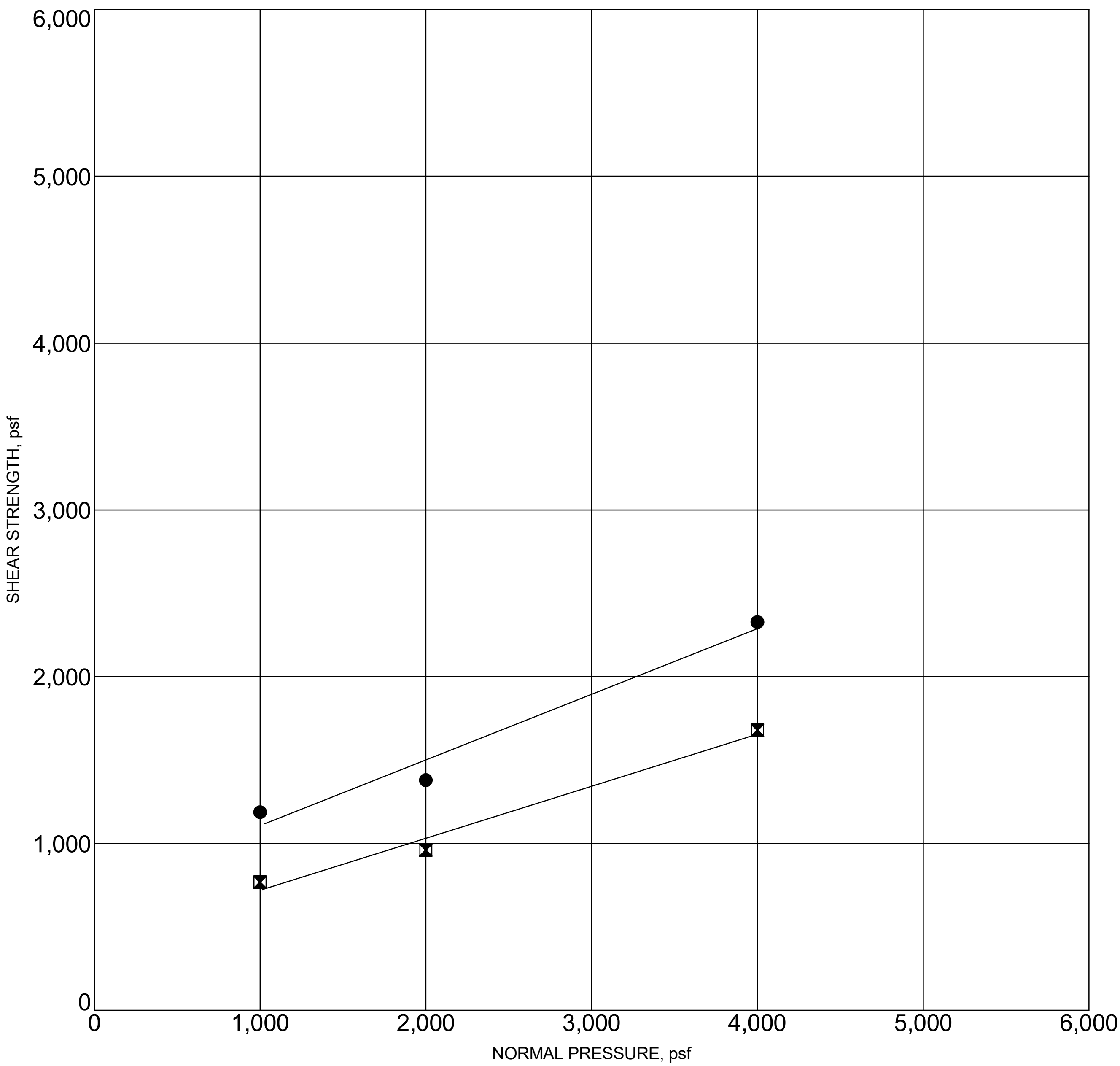
DIRECT SHEAR TEST RESULTS

FIGURE C-4



● **PEAK STRENGTH**
Friction Angle= 21 degrees
Cohesion= 714 psf

ULTIMATE STRENGTH
Friction Angle= 17 degrees
Cohesion= 408 psf

Sample Location		Classification	DD,pcf	MC,%
B-1	10.0	Bedrock: CLAYSTONE	69	50.4

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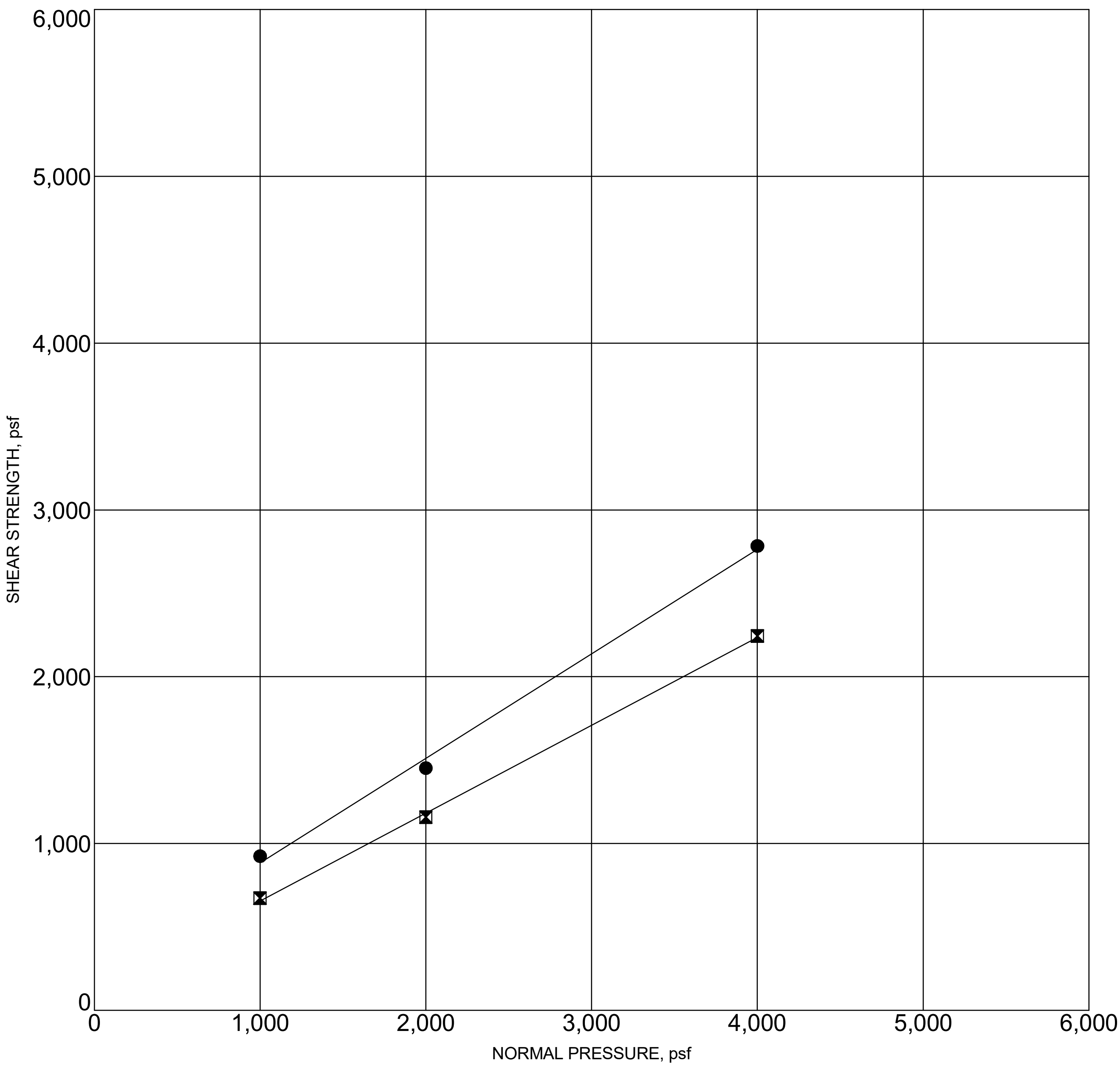
DIRECT SHEAR TEST RESULTS

FIGURE C-5



● **PEAK STRENGTH**
Friction Angle= 32 degrees
Cohesion= 258 psf

ULTIMATE STRENGTH
Friction Angle= 28 degrees
Cohesion= 129 psf

Sample Location		Classification	DD,pcf	MC,%
B-2	10.0	SAND with SILT (SP-SM)	92	7.5

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DIRECT SHEAR TEST RESULTS

FIGURE C-6