Technical Appendix G

Geotechnical Investigation Proposed Restaurant Balboa Marina Newport Beach, California
Geotechnical Professionals, Inc.
April 8, 2014

GEOTECHNICAL INVESTIGATION
PROPOSED RESTAURANT
BALBOA MARINA
NEWPORT BEACH, CALIFORNIA

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1.0 INTRODUCTION

This report presents the results of a preliminary geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for a proposed restaurant at Balboa Marina, in Newport Beach, California. The project location is shown in Figure 1.

1.1 PROJECT DESCRIPTION

At the time this report was prepared, the design of the project was in a conceptual stage. Based on information provided by Burton Landscape Architecture Studio, we understand that the proposed restaurant building will be a two-story structure to be located in the western parts of the existing parking lot of the Balboa Marina, as shown in Figure 2. The lower floor of the building will be a reinforced concrete parking structure/podium with a finish floor elevation at +9 feet (even with the lowest part of the parking lot surface). The restaurant building, on the second floor of the structure will occupy part of the total footprint, with the rest being occupied by outdoor patios, surface parking and landscaping. A conceptual design plan is presented in Figure 3.

The project will involve significant grade changes. The western edge of the site will be re-graded with a 2:1 slope cut into the existing slope, in order to accommodate the westerly expansion of the marina. Some fill will also be placed in the northeastern parts of the site to accommodate proposed grade differentials that will enable access to the second floor from the north side of the building. Up to 10 feet of fill may be placed northeasterly of the proposed building location, in order to raise grades from approximately +10 feet to +20 feet. Retaining walls will be constructed to accommodate the proposed grade differentials between the northern and southern parts of the surface parking areas.

At the time this report was prepared, the structural design was in a conceptual stage. Preliminary structural information provided by KPFF Consulting Engineers, indicates that maximum column loads are expected to be on the order of 400 kips.

1.2 PURPOSE OF THIS REPORT

The main purpose of the geotechnical investigation documented in this report was to evaluate the geotechnical conditions at the site with respect to providing adequate support for the proposed structure.

2.0 SCOPE OF WORK

The scope of this geotechnical investigation included a review of existing geotechnical information, subsurface field investigation, laboratory testing, geotechnical analyses, and preparation of this report.

Our review of existing geotechnical information included data from previous geotechnical investigations and construction monitoring by GPI, logs of borings by Caltrans for the nearby Newport Bay Bridge and published geologic information. The documents reviewed are listed under References.

The field investigation was aimed at complementing data from previous geotechnical investigations at the site and included three additional cone penetration tests (CPT's) to a depth of 50 feet, two Geoprobe borings with sampling of selected soil layers to a maximum depth of 33 feet and two hand auger borings to a depth of 7.5 feet. The CPT's were used primarily to define the subsurface layering and to obtain in-situ measurements of geotechnical properties used for evaluation of pile capacities, potential for liquefaction, seismic settlement, and seismic slope stability. The borings were located next to two of the CPT locations (C-1 and C-2) and were used to obtain soil samples of selected layers for laboratory testing and to measure groundwater levels. CPT field procedures and logs are presented in Appendix A. Field procedures and logs of borings are presented in Appendix B. The approximate locations of the subsurface explorations are presented in Figure 2.

The geotechnical laboratory testing program consisted of moisture and density determinations, grainsize analyses, Atterberg Limits, and direct shear tests. Due to the cohesionless nature of sandy soils at the site, the laboratory test data was used to complement the CPT data, which provided in-situ measurement of soil properties needed for foundation design. Laboratory test procedures and results are presented in Appendix C.

The geotechnical analyses focused on the stability of the site under seismic loading conditions, foundation analyses for piles and shallow footings, settlement analyses for areas to be filled, and lateral earth pressures for retaining wall design. The results of the geotechnical evaluations are presented in Sections 3 and 4 of this report.

3.0 SITE CONDITIONS

3.1 EXISTING FACILITIES

The site for the proposed restaurant building currently is a paved parking lot with a concrete seawall on the south side and a descending slope toward the water on the west side. The existing topographic conditions are shown in Figure 2.

The existing seawall consists of a series of concrete panels with two sets of tie-back anchors. The original tie-back anchors, installed when the wall was built in the 1960's are connected to a concrete trench type "deadman", located approximately 25 to 30 feet north of the seawall. The second set of tie-back anchors were installed in 2008 in order to reinforce the seawall, and to accommodate deepening of the mudline as part of the marina re-construction project. The second set of tie-back anchors were steeply inclined pressure grouted friction anchors with lateral spacing of 7 feet 5 inches and lengths of 43 feet.

3.2 SUBSURFACE SOIL CONDITIONS

The subsurface soil profile consists mostly of fine to medium sands with variable silt content. These sands are typically medium dense to dense in the upper 20 to 25 feet and become very dense at greater depths. However, at CPT C-1, medium dense sands were encountered to a depth of 30 feet. Two highly compressible clay and elastic silt layers were also detected in CPT C-1. The shallow layer, which was also sampled in hand auger boring HA-1 between depths of 5 and 6 feet and detected in C-3 between depths of 12.5 and 13.5 feet, consists of an organic clay with peat. In a laboratory consolidation test this material exhibited very high compressibility and significant secondary compression (long-term creep). A soft to firm elastic silt (MH) layer was found between 29 and 36 feet with very dense sands below 36 feet down to the maximum depth explored of 50 feet. In borings drilled by Caltrans within the eastern parts of the Newport Bay Bridge (closest to the site), the very dense sands extended to a typical elevation of -60 feet (about 70 feet below the existing site grades at the proposed restaurant site).

3.3 GROUNDWATER CONDITIONS

In the two hand auger explorations groundwater was encountered at an approximate depth of 6.5 feet, corresponding to an elevation of +3.5 feet (MLLW). This groundwater level was also consistent with piezometric levels measured in CPT C-2 at a depth of 25.5 feet. Due to the proximity of the site to open water and the sandy nature of the site soils, groundwater levels can be expected to fluctuate with tide levels. During high tide events, the groundwater level could rise to elevation +6 feet (i.e. within 3 feet of the proposed finish floor level of the proposed structure at parking level).

3.4 SEISMIC HAZARDS

The site is not located within an Alquist-Priolo earthquake fault zone and generally, good layer continuity was observed across the site. Therefore, the potential for ground rupture due to faulting at the site is low.

The site is located within a liquefaction hazard zone as mapped by the California Geological Survey (previously California Division of Mines and Geology). Therefore, an evaluation of the potential for liquefaction, seismic settlement and lateral spreading is warranted for this project.

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 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 2013 California Building Code, which is based on the ASCE 7.10 Standard, has much higher peak ground acceleration requirements than the 2010 CBC for evaluating the potential for liquefaction and lateral spreading. Based on the new requirements, the peak ground acceleration for this site, derived from the USGS Design Maps website is 0.71g.

We evaluated the potential for liquefaction and seismic settlement using methods presented by Idriss and Boulanger (2008), based on CPT data. We considered peak ground accelerations of 0.71g for evaluations. Our analyses indicate that most sandy soils at the site are dense enough to resist liquefaction even under very high ground motion. Marginal resistance to liquefaction was indicated in limited relatively thin layers of medium dense sands found mostly at shallow depths. The results of our analyses are summarized on the following page:

CPT No	Depth Interval of Layers Susceptible to Liquefaction (feet)	Seismic Settlement (inches)
1	6-7 11-13 12-24 26-29	1.88
2	4-5.5 8-10 11-13 19-20 21-22	1.02
3	13-16	0.44
4	5.5-6.5 10-12 14-15 18.5-19.5	0.89

The calculated magnitude of seismic settlement under such high level of ground motion is considered to be relatively small. The potential of seismic settlement on the design of pile foundations is discussed in Section 4.5 of this report.

The potential for liquefaction will result in a temporary loss of strength in limited layers which, in turn, will result in some permanent slope movement in the western parts of the site. None of these layers contained very loose to loose sands that would be susceptible to flows upon liquefaction. We evaluated the potential for lateral spreading based on data from CPT's C-2 and C-4, and methods proposed by Youd, et al. (Reference 9) and Zhang et al. (Reference 10). The analyses indicated lateral spreading potential less than 5 inches for a peak ground acceleration of 0.71g. Both of these methods are considered to be reasonable screening tools for predicting the potential for relative large lateral spreads but have been shown to grossly over-predict small displacements (Chu et al., Reference 11). Therefore, the potential for lateral spreading at this site due to liquefaction is considered to be negligible. We also evaluated permanent slope displacements by slope stability methods using residual shear strength parameters for the limited layers that would be susceptible to liquefaction. The slope stability analyses are summarized in Section 4.3.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 OVERVIEW

Based on the results of our geotechnical investigation, it is our opinion that the geotechnical conditions at the site are suitable for supporting the proposed development. However, the constraints discussed below need to be considered in the design and construction of the proposed development.

The proposed structure will need to be supported on pile foundations, in order to limit surcharge loads on the existing seawall. Furthermore, in locating columns and pile foundations, it would be imperative to avoid impacting the existing tie-back anchors that provide support to the existing seawall.

The potential for liquefaction in limited layers at the site is expected to have a minor impact on a pile supported structure. Under very strong earthquake conditions, up to 1½ inches of differential settlement may be experienced between slab-on-grade floors and the pile-supported superstructure. If this is not acceptable, structural floor slabs could be used at substantially higher cost. Liquefaction in limited layers could also result in some slope deformation, as discussed in Section 4.3. The deformation of the slope will be resisted by any piles located within 15 to 20 feet from the top of the slope, reducing the lateral load capacity available to resist seismic lateral loads from the structure. The adverse impact of slope deformation on the lateral load capacity of pile foundations can be minimized if the west edge of the structure is located at least 20 feet east of the top of slope.

The geotechnical investigation disclosed the presence of two highly compressible cohesive soil layers in the eastern parts of the site. The compressibility of these layers, found below depths of 5 feet and 29 feet, respectively, will mainly impact the support of the retaining wall and fill planned east of the building. If left unmitigated, up to approximately 3½ inches of settlement is anticipated under the weight of 10 feet of fill. Over 2½ inches of the estimated settlement will be due to compression of the shallower of the two compressible soil layers, which consists of organic clay and peat. In order to mitigate the potential for excessive differential settlement along the retaining wall and between the finished ground surface and the pile-supported building, we recommend a combination of two measures. The shallow organic clay/peat layer should be overexcavated and replaced with compacted fill. Additionally, the above grade fill should be placed well in advance of building and retaining wall construction to allow settlement to occur before structures are built. With this combination of measures, differential settlement can be reduced to less than ½-inch. Detailed earthwork recommendations are presented in Section 4.4.

4.2 SEISMIC DESIGN

The method of seismic design should be determined by the structural engineer. For seismic design by the 2013 California Building Code (ASCE 7-10 Standard), the site parameters are as follows:

Site Class: D

S_S	= 1.719	S_1	= 0.634
S_{MS}	= 1.719	S_{M1}	= 0.951
S_{DS}	= 1.146	S_{D1}	= 0.634

4.3 SLOPE STABILITY

The finished slope at the west edge of the site will have an average inclination of 2 horizontal to 1 vertical, a toe elevation at -10 feet (MLLW), and a top elevation of +9 feet. We evaluated the stability of the slope for static and seismic load conditions using the computer program Slide 6.0 and the Modified Bishop method for both circular and non-circular failure surfaces.

The calculated minimum factor of safety under static loading conditions is approximately 1.8. A factor of safety of 1.5 or greater is considered to be adequate for static load conditions.

The stability of the slope under seismic conditions was evaluated in general accordance with guidelines in Special Publication 117 (CGS 2008) by "Newmark" type cumulative displacement analyses. The procedure first involves calculation of the pseudostatic "yield" acceleration that would result in a calculated factor of safety of 1.0. Then the ratio of the peak ground acceleration to the yield acceleration is used in empirical relationships to estimate the cumulative slope displacement. For our evaluations, we used empirical relationships outlined in NCHRP Report 611 (Reference 12). In our analyses, we used static shear strength parameters for the dense to very dense sand layers and residual undrained shear strength parameters for medium dense sand layers that would be susceptible to liquefaction. The residual undrained shear strengths used for these analyses were obtained from empirical relationships proposed by Seed and Harder (Reference 8). Under free-field conditions, the calculated slope top displacements are on the order of six inches for a peak ground acceleration of 0.71g. Lateral displacements can be expected to decrease with distance from the slope. With the pile supported structure in place, the piles located within a distance of about 20 feet from the top of slope will provide shear pinning further reducing the magnitude of slope displacement. On the other hand, piles located within 20 feet from the top of slope will not have reserve capacity to provide lateral resistance to seismic loads from the structure.

4.4 EARTHWORK

The earthwork anticipated at the project site will consist of clearing and grubbing, excavations, subgrade preparation, and placement and compaction of fill.

4.4.1 Clearing and Grubbing

Prior to grading, the areas to be developed should be stripped of any vegetation and cleared of all demolition debris. Any roots, buried footings of demolished structures, abandoned utility lines, buried tanks, and other underground structures should be removed in their entirety. If concrete piles are encountered within building areas, as a minimum, they should be cut off at a depth of 5 feet below finish grades. Cesspools, if encountered, should be emptied of their contents and either removed entirely or backfilled to within 5 feet of the finished subgrade with one sack cement slurry. The upper 5 feet should be backfilled with compacted soil. All deleterious material generated during the clearing operations, including all organic material, should be removed from the site.

4.4.2 Excavations

Excavations at the site will include over-excavations to remove the highly compressible organic clays, footing excavations, and trenching for utility lines.

In building pad areas, soils disturbed by demolition activities should be overexcavated and replaced with properly compacted fill. These materials require densification to provide adequate support for slab-on-grade floors. In the proposed deep fill and retaining wall area, east of the proposed building, the highly compressible organic clay found between approximate depths of 5 and 6 feet should be overexcavated, and replaced with compacted sandy fill. The lateral extent of the overexcavation should extend from the slope to the north to 5 feet beyond the retaining wall to the south. The approximate limits of the overexcavations are shown in Figure 2.

All excavations and shoring systems should meet the minimum requirements given in the most current State of California Occupational Safety and Health Standards. In accordance with OSHA criteria, the soils at shallow depths (upper 7 feet) are classified as Type C soils. Such soils are susceptible to caving. Any excavations extending below the groundwater level will need to be dewatered before the excavation reaches the groundwater level. Due to the sandy nature of soils prevailing at shallow depths, if excavation below the groundwater level is attempted without dewatering in advance, unstable, “quick” conditions will be created at the bottom of excavations.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut from the toe of the excavation or 5 feet from the top of the slope, whichever is greater, unless the cut is properly shored. The shoring must be designed to resist earth pressures from gravity loads plus surcharge loads. All excavations and shoring systems should meet the minimum requirements given in the State of California Occupational Safety and Health Standards.

In general, the excavation should be readily accomplished by conventional soil excavation equipment such as backhoes, loaders, scrapers, or dozers. However, rubber-tired equipment is likely to experience mobilization difficulty in wet organic clays, typically found below a depth of 5 feet.

4.4.3 Subgrade Preparation

After removals are complete and prior to placing any fills or constructing pavements or structures, the subgrade soils should be scarified to a depth of 6 inches moisture-conditioned, and compacted to at least 90 percent of maximum dry density in accordance with ASTM D 1557.

The upper 12 inches of the pavement subgrade should be compacted to a relative compaction of 95 percent.

4.4.4 Material for Fill

The majority of the soils at the site are non-expansive sands. Such soils are suitable for re-use in fills. Clayey soils, found in a limited thin layer below 5 feet, could be used in deep fills provided they are thoroughly blended with the non-expansive sands.

Imported fill material should be predominately granular, non-expansive (EI$\leq$ 20) and contain no more than 40 percent fines (portion passing No. 200 sieve). The Geotechnical Engineer should be notified at least 72 hours in advance of the location of any soils proposed for import. 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 the Geotechnical Engineer may be rejected if not suitable for use as compacted fill.

Crushed, inert demolition debris, such as crushed asphalt pavement or concrete may be used in fills with the following processing requirements:

- If the inert debris is crushed to a well graded mixture with maximum particle size of 1½ inches, the crushed material may be used directly in the fill without further blending.
- Inert debris up to a maximum size of 6 inches may be also be used in deep fills provided it is thoroughly blended with moisture-conditioned on-site soil to form a well-graded mixture. In general, at least four volumes of soil will be required per volume of debris.

4.4.5 Placement and Compaction of Fills

All fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to dry densities equal to at least the following percentages of their respective maximum densities, determined in accordance with ASTM D 1557.

On-site sands (pavement subgrade):	95 percent
On-site sands (all other fills):	92 percent
Base course:	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 for soil fills.

Plate compactors (wackers):	4-6 inches
Small vibratory or static rollers:	6-8 inches
Scrapers and heavy loaders:	8-10 inches
Heavy vibratory (pad foot, 20-ton dynamic)	10-12 inches

For soils, the maximum lift thickness should never be greater than 12 inches.

Fills within 2 feet of retaining walls or basement walls should be compacted using light equipment (such as plate compactors) in order to minimize lateral pressures on the walls.

The moisture content of the sandy fills will need to be within 2 percent of optimum moisture to readily achieve the required degree of compaction. "Pumping" could be experienced, if compaction to high densities is attempted at moisture contents more than 2 percent above optimum. The in-place moisture content of sandy soils at the site was found to be variable. Therefore, moisture conditioning could involve significant drying as well as wetting, to reach optimum moisture conditions.

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

4.4.6 Shrinkage and Subsidence

Shrinkage is the loss of soil volume caused by compaction of fills to a higher density than the existing in place density. 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 15 percent and subsidence of 0.2 feet may be assumed for the existing fills and natural soils. It should be realized that the site soils exhibit variable densities, making shrinkage factors difficult to determine. These values are estimates only and exclude losses due to removal of vegetation or demolition debris. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be determined during grading.

4.4.7 Surcharging Deep Fill Area

As noted in Section 4.1, fill in areas east of the building should be placed well in advance of construction of foundations and the ground subsidence be monitored to confirm that most of the anticipated subsidence takes place before foundations are constructed. We recommend that the limits of this initial "surcharge" fill extend at least 5 feet beyond the building and retaining wall line. Subsequently, the fill should be trimmed back with a 1:1 slope to allow construction of the building and the retaining wall.

4.4.8 Trench Backfill

Utility trench 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. The Geotechnical Engineer should observe and test all 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 one sack 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.

4.4.9 Observation and Testing

A representative of the project Geotechnical Engineer should observe all overexcavations, 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. Test areas that do not meet minimum compaction requirements should be reworked and tested prior to placement of any additional fill.

4.5 PILE FOUNDATIONS

As noted previously, the proposed building will be supported on pile foundations, mainly in order to minimize impacts on the existing seawalls. Prior to finalizing the locations of pile foundations, particularly in the southern parts of the building, the locations of existing underground structures such as tie-back anchors and the deadman trench related to the existing seawall should be accurately surveyed and shown on the foundation plans.

4.5.1 Pile Types

We considered a range of pile foundation alternatives, considering the soil and groundwater conditions, as well as noise and vibration impacts.

Conventional drilled pile foundations are not suitable for this project, primarily because of the caving potential of the sandy soils and the shallow groundwater levels.

Driven pre-stressed concrete piles are commonly used for supporting buildings and would offer high end bearing and side friction capacity in the sandy soils, making them an efficient pile type for the soil conditions at the site. The driving resistance observed during construction is an indication of the compressive capacity of each driven pile; thus, providing a quality control check for each installed pile. However, driven piles have much higher noise and vibration impacts than the other pile alternatives considered.

Auger-cast pressure grouted (APG) piles will be well-suited for this project because they offer close to the load capacities of driven piles but with substantially lower noise and vibration impacts. However, such piles need to be installed by qualified specialty contractors that offer detailed "real-time" monitoring of grout volumes and pressures on a continuous basis, in order to verify the continuity of the grout column. Such monitoring should be performed using Automated Monitoring Equipment (AME), such as the Pile Installation Recorder (PIR) by Pile Dynamics Inc. APG piles are typically contracted on a "design-build" basis. The adequacy of the design needs to be verified by pile load tests, because the installation process itself does not provide verification of pile capacity (as it does for driven piles).

Based on preliminary discussions with the design team, we understand that APG piles will be the preferred alternative, mainly because the lower noise impacts. Therefore, our recommendations are limited to APG piles.

4.5.2 Axial Pile Capacities

As indicated previously, APG pile installations are typically contracted on a "design-build" basis, with performance criteria for axial and lateral load capacities. Based on the maximum column loads of about 400 kips, we anticipate axial pile capacities will be on the order of 200 kips or less.

For preliminary planning purposes, we evaluated axial capacities for 16-inch APG piles, a typical size used for static compressive service loads up to approximately 250 kips. We evaluated capacities for two soil profiles. The lower bound capacities are for the soil profile encountered at CPT C-1, which will impact the eastern parts of the building (see Figure 2 for approximate limits). The conditions at CPT C-2 represent the typical conditions in the rest of the site. We recommend a minimum depth of embedment of 40 feet for the eastern parts of the site and 35 feet for the rest of the site. The calculated allowable axial compressive capacities for either static or seismic loads are presented in Figure 4. It should be noted that, typically, a one-third increase is allowed under seismic loads. However, in this case, liquefaction in limited layers at shallow depths will result in some down drag, off-setting the increase due to dynamic loads. Therefore, we recommend using the same compressive axial capacities for static and seismic loads. The allowable axial capacities in uplift will be approximately equal to one-half the axial capacities in compression.

Reduction in axial capacities due to group action can be neglected as long as the center to center pile spacing for a pair of piles is more than 3 pile diameters (4 feet for 16-inch piles).

The allowable axial capacities presented in Figure 4, will be mobilized with a tip deflection less than ¼-inch. The compression of the pile should be added to this value to obtain the pile top deflection. The axial compressive loads pile top stiffness can be obtained by dividing the axial load with the pile top deflection.

Up to an additional ¼-inch to ½-inch of pile settlement can be experienced after strong earthquake shaking, due to down drag resulting from liquefaction in limited medium dense sand layers at the site.

4.5.3 Lateral Pile Capacities

We evaluated the response of 16-inch piles for two cases of lateral loads. Case 1 represents typical foundation loading applied at the top of the pile (base of pile cap) assumed to be about 3 feet below the finish floor level. For this case, we evaluated pile response for both free and fixed pile top conditions. Case 2 represents lateral loading on piles located within 20 feet from the top of slope, due to potential slope movement under seismic loads. In this case, soils within the upper 6 feet below the pile cap (above the critical slip surface) were assumed to act as the driving force rather than to provide shear reinforcement reducing the slope displacement (shear pinning effect). In this case, the load is applied at the mid point of the sliding soil mass (3 feet below the actual pile cap). For this case, we assumed partial fixity at the top of the pile. The impact of slope movement on lateral loading of piles could be minimized if the structure were to be moved to the east allowing a clear distance of 20 feet between the west edge of the structure and the top of slope to the west.

Lateral load capacities were evaluated using the computer program LPILE Plus, Version 5.0. The results of the analyses are presented below:

CASE	LOAD (kips)	TOP DEFLECTION[1] (inches)		MAXIMUM BENDING MOMENT (in-kips)		DEPTH[2] (feet)	
		FREE	FIXED	FREE	FIXED	FREE	FIXED
1	10	0.07	0.03	+215	-300, +80	3.6	7.5
	20	0.14	0.06	+450	-600, +170	3.8	7.7
	30	0.27	0.09	+755	-915, +245	4.0	7.9
	40	0.43	0.14	+1075	-1240, +355	4.2	8.0
2	10	0.31		-540, +395		10.8	
	15	0.56		-810, +650		11.0	
	20	0.84		-1080, +940		11.3	
	25	1.15		-1370, +1240		11.6	

NOTES: 1. The deflections calculated above are based on the assumed structural stiffness of (EI) of 1.44×10^{10} in^2-lb and must be verified based on the structural design of the piles. Furthermore, these deflections are valid up to the elastic bending limit of the piles and would increase significantly when the bending capacity of the pile is exceeded.

2. The depth is the distance to maximum positive bending moment below the top of the pile. The maximum negative bending moment for fixed and partially fixed conditions is at the top of the pile.

The lateral spring constants for the piles can be calculated by dividing the lateral load by the deflection.

Group action in lateral loading is more significant than in axial loading, and depends on the direction of loading relative to the orientation of the piles. Side-by-side means loading perpendicular to pile alignment while in line means loading along the alignment of adjacent piles.

LATERAL LOAD REDUCTION FACTOR VS. PILE SPACING

PILE SPACING (diameters)	REDUCTION FACTOR FOR LOADING	
	Side-by-Side	In-Line (trailing pile)
6	1.0	0.7
4	1.0	0.4
3	0.9	0.3
2	0.7	0.2

4.5.4 Pile Caps

Vertical bearing and friction on the bottom of pile caps should be neglected because practically all of the vertical load will be transferred to the piles. On the other hand, the pile caps will provide lateral resistance to loading under seismic loads. The passive resistance against the embedded portions of the pile caps can be calculated based on an equivalent fluid pressure of 350 psf/foot.

We recommend that the pile caps be tied together with grade beams in order to resist the potential for differential ground displacement under seismic conditions, particularly in the western parts of the site.

4.6 SHALLOW FOUNDATIONS

Shallow foundations will not be appropriate for supporting the restaurant building. However, shallow foundations and mats could be used to support lightly loaded structures, including retaining walls, equipment pads and vaults. Shallow foundations should be supported on compacted fill or undisturbed natural soils. The footings for the retaining wall on the east side of the building and any other settlement-sensitive structures should be supported on compacted fill placed as recommended in Section 4.4.

4.6.1 Allowable Bearing Pressures

Based on the shear strength and elastic settlement characteristics of the recompacted on-site soils, a maximum static allowable bearing pressure of up to 3,000 pounds per square foot may be used for design for the support of the retaining wall on the east side of the building. This bearing pressure is for dead load plus sustained live load, and may be limited to lower pressures, depending on foundation sizes, as discussed below. The allowable pressures may be increased by one-third for short-term, transient, wind and seismic loading. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values.

Foundations for minor structures supported on the existing soils (short retaining walls, transformer pads, trash enclosures etc.) may be designed for an allowable bearing pressure of 1,500 pounds per square foot.

4.6.2 Minimum Footing Size

The minimum allowable widths of footings will depend on the bearing pressure used for design, as follows:

STATIC BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (inches)
3,000	24
2,500	18
1,500	12

4.6.3 Minimum Footing Embedment

The recommended minimum depths to the bottom of footings below lowest adjacent finish grade are as follows:

Retaining walls:	18 inches
Minor equipment pads:	12 inches

Minor equipment pads include trash enclosures, and slabs supporting utility equipment.

4.6.4 Estimated Settlements

Under static (sustained) load, the estimated maximum settlement of footings designed in accordance with recommendations presented herein is expected to be less than ½-inch. As noted previously, under strong earthquake shaking up to 2 inches of additional settlement can be experienced at the site. The maximum seismic settlement potential is in areas east of the proposed building location.

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

4.6.5 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 between concrete and undisturbed soil, a coefficient of friction of 0.4 may be used for design. For passive resistance in flat ground, an allowable lateral bearing pressure equal to an equivalent fluid weight 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.

For retaining wall footings located at the top of slope along the west side of the site, the allowable passive resistance will be limited to 150 psf/ft.

4.6.6 Footing Excavation Observation

Prior to placement of concrete and steel, a representative of GPI should observe and approve all footing excavations.

4.7 BUILDING FLOOR SLABS

Slab-on-grade floors should be supported on re-compacted existing subgrade soil or non-expansive ($EI \leq 20$) fills, placed and compacted as discussed in the "Placement and Compaction of Fill" section.

The structural design of the floor slabs should be performed by the Structural Engineer based on static and seismic load demands. The on-site surficial soils are non-expansive.

A vapor/moisture retarder should be placed under slabs that are to be covered with moisture-sensitive floor coverings. Currently, common practice is to use 10-mil polyethylene (visqueen) as a vapor retarder, placed either directly on the subgrade or over a thin layer of sand. In recent years, other types of vapor retarders with much lower permeability and higher puncture resistance have become available and should be considered as an alternative. Polyolefin in 10-mil or 15-mil thickness is such a material and should be considered for this project.

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. For example, limiting the water-cement ratio in the concrete and allowing enough drying time are critical factors. Other factors include effective sealing of joints and edges (particularly at pipe penetrations). 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.

Common practice is to cover the vapor retarder with a layer of clean sand (less than 5 percent by weight passing the No. 200 sieve) having a minimum thickness of 2 inches. 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 barrier should be dry. 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.

For lateral resistance design, a coefficient of friction of 0.40 can be used for concrete in direct contact with sandy fill. For slabs constructed over a visqueen or polyolefin moisture retarder, a friction coefficient of 0.1 should be used. If structural floor slabs are used for the project, the friction under the slab should be ignored because a gap between the bottom of the structural slab and subgrade soils will develop as a result of seismic settlement.

4.8 RETAINING WALLS

The most significant retaining walls of the proposed project are the partial basement walls on the north and eastern parts of the parking structure and the exterior retaining wall, supporting a grade differential up to 10 feet on the east of the proposed building. We assume that the partial basement walls will be supported on pile-supported grade beams. The exterior retaining wall east of the building will be supported on a continuous footing bearing on compacted fill. A shallow retaining wall is also planned at the top of slope on the west side of the building. This shorter retaining wall will be supported on existing soils. Foundation recommendations are presented in Sections 4.5 and 4.6 of this report.

Active earth pressures can be used for designing walls that can yield at least ¼-inch laterally under the imposed loads. For level non-expansive granular backfill the magnitude of active pressures are equivalent to the pressures imposed by a fluid weighing 30 pounds per cubic foot (pcf). For areas where the walls will retain slopes inclined at 2:1 (horizontal:vertical), the active pressure should be taken as 50 pcf.

At-rest pressures should be used for restrained walls that remain rigid enough to be essentially non-yielding. At-rest pressures are equivalent to the pressures imposed by a fluid weighing 50 pounds per cubic foot (pcf) should be used for level 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. All retaining walls should be equipped with back drains to eliminate the potential for build-up of water pressures.

Significant increases in lateral earth pressures can be experienced under strong earthquake loading conditions. The incremental additional earth pressures will depend mainly on the level of ground shaking. The estimated seismic pressure increases (above the static active pressures) are as follows:

EARTHQUAKE CONDITIONS	PEAK GROUND ACCELERATION	PSEUDO STATIC COEFFICIENT	PRESSURE INCREASE (%)	TOTAL (STATIC + SEISMIC) EQUIV. FLUID PRESSURE (psf/ft)
MCE	0.71g	0.36	100	60
2/3 MCE	0.46g	0.23	70	51

A triangular pressure distribution (same as for the static case) can be used for all retaining walls.

4.9 DRAINAGE

Positive surface gradients should be provided adjacent to all 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 or adjacent to buildings. Additionally, landscaped areas should be properly drained to prevent moisture infiltration into the base course of pavements in adjacent areas.

The soil and groundwater conditions at the site are NOT suitable for subsurface storm water discharge for the following reasons:

a. The groundwater level is less than 10 feet below the ground surface; and

b. Introduction of water into the subsurface soils will adversely impact foundation and pavement support conditions.

4.10 EXTERIOR CONCRETE AND MASONRY FLATWORK

Exterior concrete and masonry flatwork should be supported on a zone of compacted fill with low expansion potential. Prior to placement of concrete, the subgrade should be prepared as recommended in "Subgrade Preparation". The subgrade soils should not be allowed to dry to a moisture content below optimum until concrete is placed. Because the surficial on-site soils exhibit a low potential for expansion, no special reinforcement is necessary to resist expansive forces. However, nominal reinforcement, as a minimum, consisting of 6x6 No. 10 welded wire mesh, is recommended. The use of the clayey soils in the slab subgrade should not be permitted.

4.11 PAVED AREAS

The soils at shallow depths consist predominantly of silty sands, with estimated R-values on the order of 40 Final pavement design should be based on R-value testing performed near the conclusion of rough grading.

Preliminary pavement design has been based on an R-value of 40 and conventional Traffic Indices (TI's) typically used for commercial developments. The California Division of Highways Design Method was used for design of the recommended preliminary pavement sections. The following pavement sections are recommended for planning purposes only.

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		ASPHALTIC CONCRETE	AGGREGATE BASE COURSE
Driveways	6	3	5
Parking Stalls	4	3	4

If a Portland Cement concrete section is desired for driveways, we recommend a preliminary pavement section consisting of 7 inches of concrete over subgrade compacted to 95 percent, as discussed in the "Subgrade Preparation" section of this report. The concrete should have a Modulus of Rupture of at least 570 psi (equivalent to an approximate compressive strength of 4,000 psi) at the time the pavement is subjected to truck traffic.

The pavement subgrade underlying the Class II 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 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 above recommendations are based on the assumption that the base course 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.

4.12 SOIL CORROSIVITY

Soil corrosivity testing performed by A.P. Engineering and Testing (reported in Appendix C) indicates that both the soils at the site are highly corrosive to metals. We recommend that a corrosion engineer be retained to provide recommendations for corrosion protection measures for any metallic structures or utility lines that would be in contact with soils.

Soil corrosivity testing was performed on a representative sample of soils from shallow depths, indicating sulfate content of 0.186 percent by weight. In accordance with ACI 318, concrete in contact with the site soils should be designed for "Moderate" sulfate exposure.

4.13 GEOTECHNICAL REVIEW

At the time this report was prepared, the design of the project was in a preliminary stage. Continued geotechnical input, as the design progresses, is needed for this project, because design decisions in several aspects of the project have significant geotechnical implications.

GPI should continue to participate in the design effort and should review foundation plans, grading plans, retaining structure plans and drainage plans.

5.0 LIMITATIONS

This report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for use by Irvine Company 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.

The soils 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 by GPI during grading, excavation, and foundation construction. If construction phase services are performed by others, they must accept full responsibility 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.

REGISTERED PROFESSIONAL ENGINEER
BYRON KONSTANTINIDIS
No. GE 482
Exp. 12/31/15
GEOTECHNICAL
STATE OF CALIFORNIA

Byron Konstantinidis, G.E.
Principal

BK:sph

APR - 8 2014

REFERENCES

1. Geotechnical Professionals Inc., "Geotechnical Investigation Report, Dredging and Dock Reconfiguration, Balboa Marina, Newport Beach, California," GPI Project No. 1917.I, dated February 6, 2004.

2. Geotechnical Professionals Inc., "Report of Geotechnical Observation and Evaluations Related to the Installation of Guide Piles and Underwater Sheet Pile Wall at Balboa Marina, Newport Beach, California," GPI Reference No. 1917.C, dated June 11, 2009.

3. Geotechnical Professionals Inc., "Summary of Geotechnical Observation of Tieback Anchors, Balboa Marina Seawall Repair Project, 201 East Coast Highway, Newport Beach, California," GPI Project No. 1917.C, dated June 16, 2009.

4. Geotechnical Professionals Inc., "Geotechnical Investigation Report, Balboa Marina West Expansion and Public Transient Dock, Newport Beach, California," GPI Project No. 2501.I, dated December 17, 2012.

5. State of California, "Seismic Hazard Zones, Newport Beach Quadrangle," Official Map, April 17, 1997.

6. California Geological Survey, "Special Publication 117, Guidelines for Evaluating and Mitigating Seismic Hazards in California," (2008).

7. Idriss, I.M. and Boulanger, R.W., "Soil Liquefaction During Earthquakes," EERI MNO-12, 2008.

8. Seed, R.B. and Harder, L.F., "SPT-Based Analysis of Cyclic Pore Pressure Generation and Residual Strength", Proceedings H. Bolton Seed Memorial Symposium, Volume 2. pp 351-376, 1990.

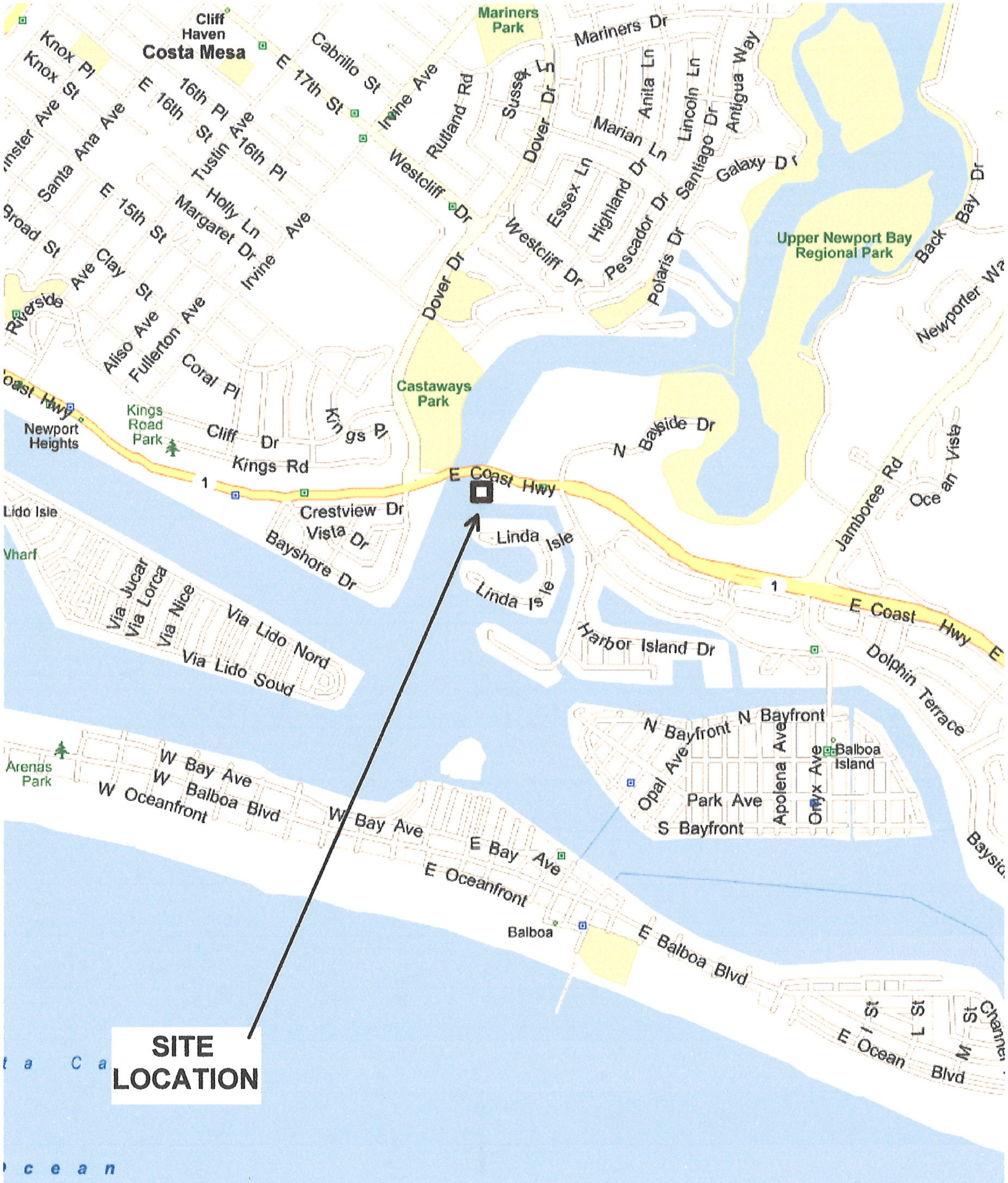
9. Youd, T.L., Hansen, C.M., and Bartlett, S.F. (2002), "Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement," J. Geotech. Geoenviron. Eng., 128(12), 1007-1017.

10. Zhang, G. Robertson, P.K. and Brachman, R.W. (2004), "Estimating Liquefaction Induced Lateral Displacements using the Standard Penetration Test or Cone Penetration Test," J. Geotech. Geoenviron. Eng., 130(8), 861-871.

11. Chu, D.B., Stewart, J.P., Youd, L., and Chu, B.L. (2006), "Liquefaction-Inducted Spreading in Near-Fault Regions during the 1999 Chi-Chi, Taiwan Earthquake," Journal of Geotechnical and Geo-environmental Engineering, pp 1549-1565.

12. Anderson, D.G., Martin, G.R., Lam, I.P., Wang, J.N., "Seismic Analysis and Design of Retaining Walls, Buried Structures, Slopes, and Embankments," NCHRP Report 611, 2008.

13. Rocsciene Inc., "Slide 6.0 – 2D Limit Equilibrium Slope Stability Analysis.



0 2000 4000 FEET

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

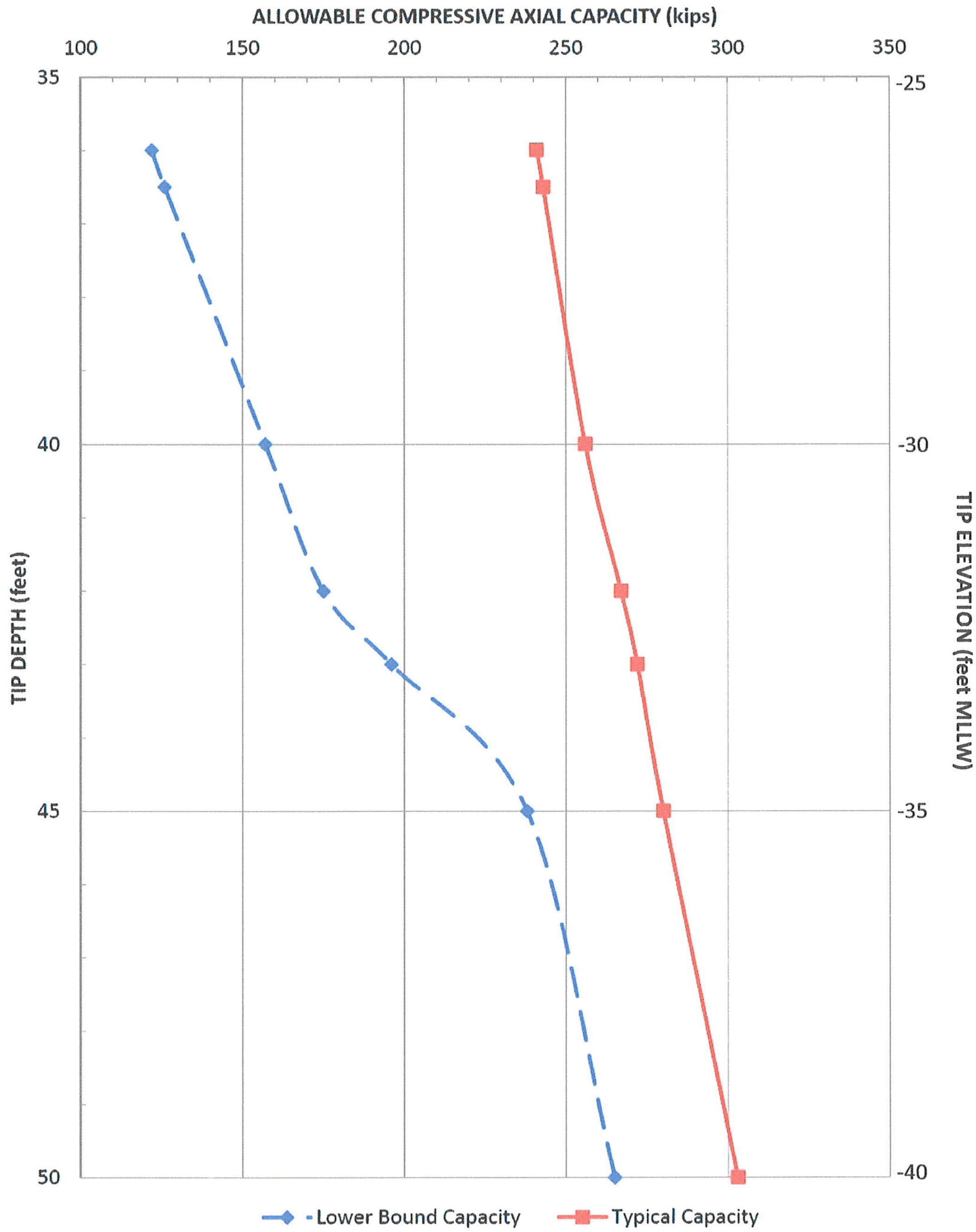
GEOTECHNICAL PROFESSIONALS, INC.

BALBOA MARINA RESTAURANT

GPI PROJECT NO. 2569.I	SCALE: 1" = 2000'

SITE LOCATION MAP

FIGURE 1



THE TIP DEPTH IS BELOW THE EXISTING GROUND SURFACE

LOWER BOUND CAPACITY = EASTERN ONE THIRD OF BUILDING

TYPICAL CAPACITY = WESTERN TWO THIRDS OF BUILDING

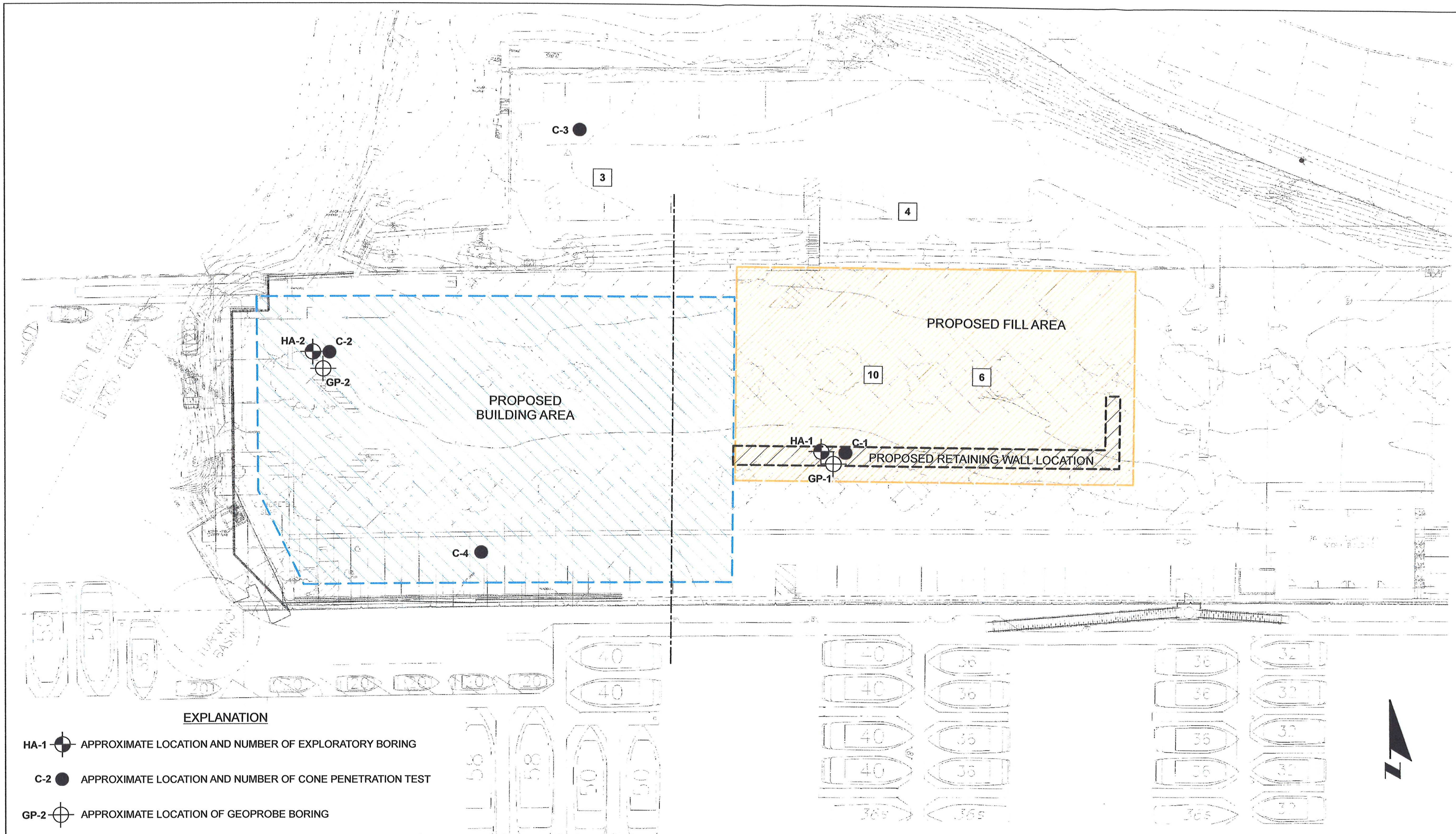
GEOTECHNICAL PROFESSIONALS, INC.

BALBOA MARINA RESTAURANT

GPI PROJECT NO.: 2569.I	NO SCALE

AXIAL PILE CAPACITY

FIGURE 4



EXPLANATION

HA-1 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

C-2 APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST

GP-2 APPROXIMATE LOCATION OF GEOPROBE BORING

APPROXIMATE WESTERN LIMIT OF "LOWER-BOUND" SOIL PROFILE FOR PILES

10 PROPOSED DEPTH OF FILL IN FEET

APPROXIMATE LIMITS OF OVEREXCAVATION

BASE PLAN REPRODUCED FROM TOPOGRAPHIC MAP PROVIDED BY URS: (UNDATED)

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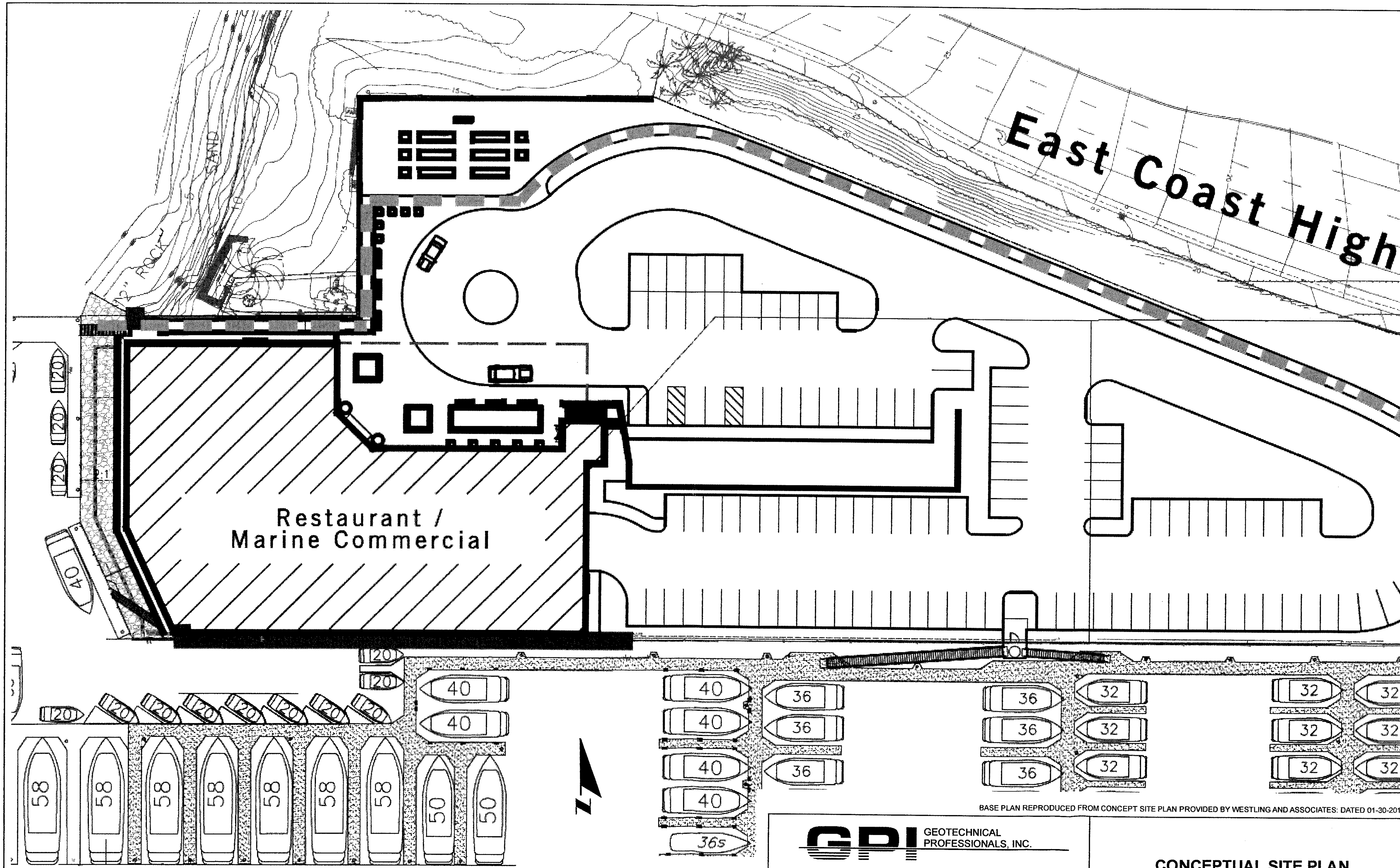
BALBOA MARINA RESTAURANT

GPI PROJECT NO.: 2569.I | SCALE: 1" = 40'

SITE PLAN
EXISTING CONDITIONS

FIGURE 2



GPI GEOTECHNICAL PROFESSIONALS, INC.

BALBOA MARINA RESTAURANT

GPI PROJECT NO.: 2569.I

SCALE: 1" = 40'

CONCEPTUAL SITE PLAN

FIGURE 3

APPENDIX A

APPENDIX A

CONE PENETRATION TESTS

The subsurface conditions were investigated by performing three Cone Penetration Tests (CPT's) at the site during the current investigation. We also utilized data from one CPT (C-4) performed during a prior investigation in October 2003 near the west end of the original marina reconstruction. The soundings from the current investigation were advanced to depths of approximately 50 feet below existing grades. The sounding from the prior investigation was advanced to a depth of 40 feet below existing grades. The locations of the CPT's from both investigations are shown on the Site Plan, Figure 2.

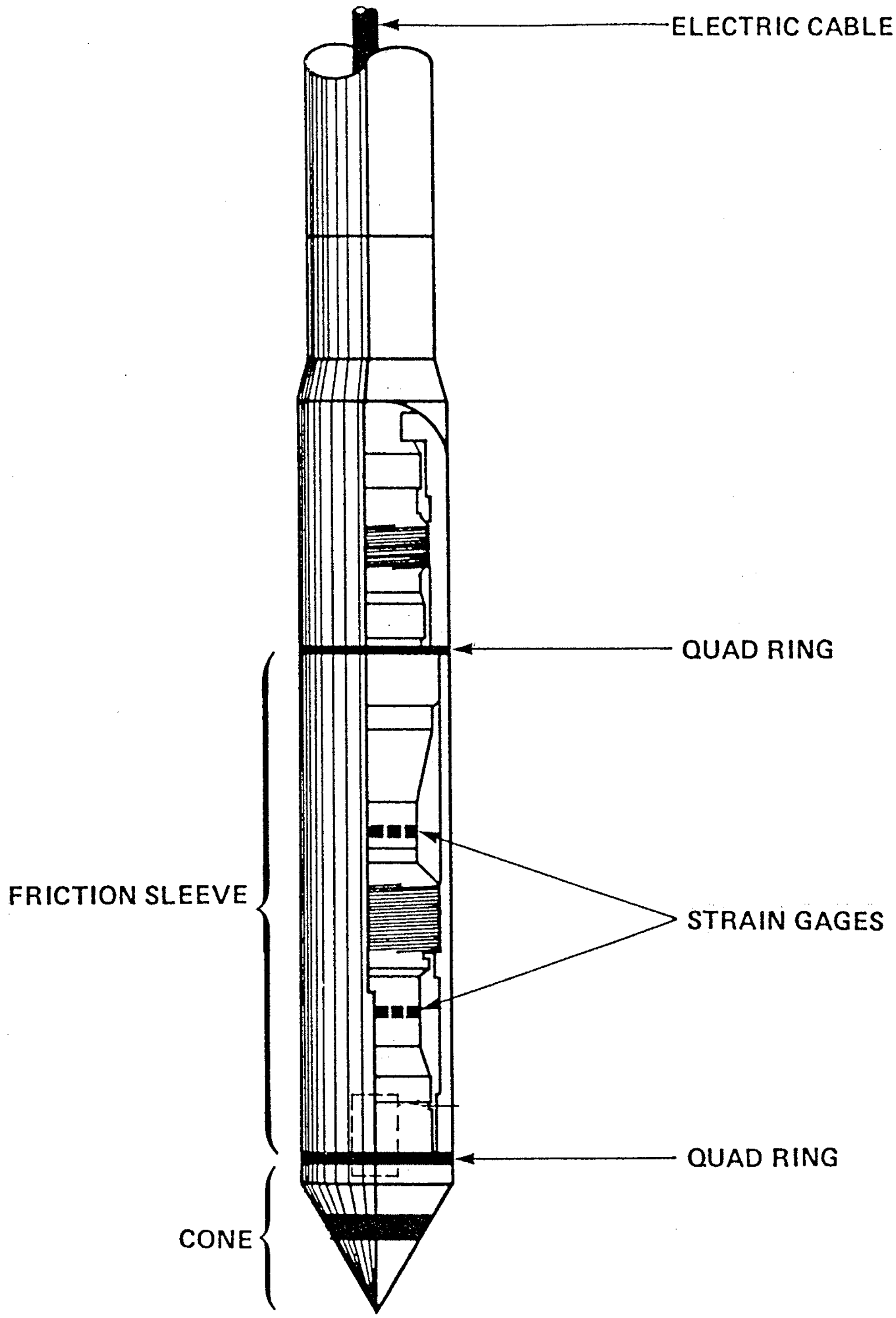
The Cone Penetration Test consists of pushing a cone-tipped probe into the soil deposit while simultaneously recording the cone tip resistance and side friction resistance of the soil to penetration (refer to Figure A-1). The CPT described in this report was conducted in general accordance with ASTM 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. These CPT's were performed using a specially designed truck 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 these investigations are 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.

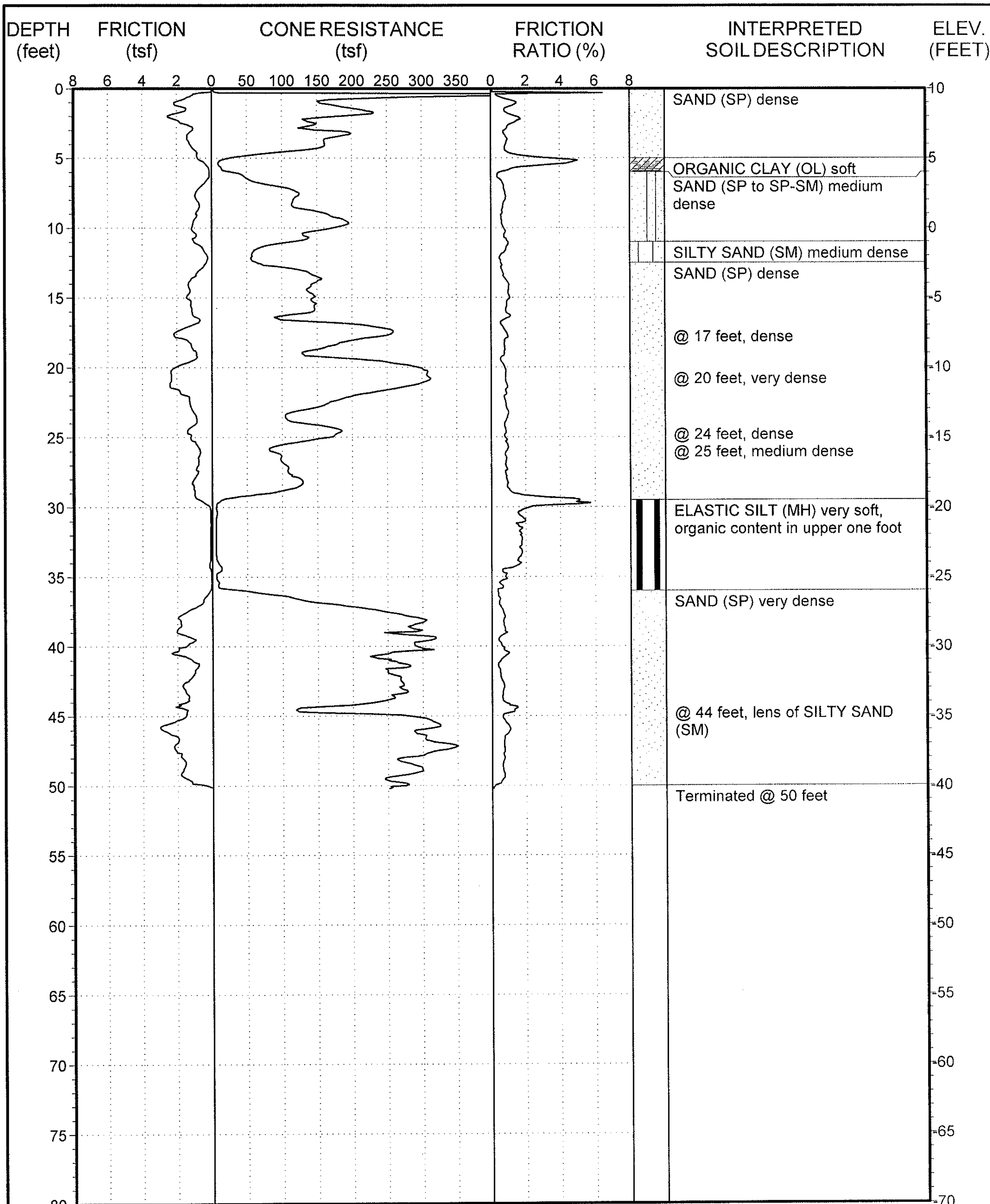
The CPT locations were laid out in the field by measuring from existing site features and using GPS applications. Ground surface elevations at the CPT locations were estimated from topographic survey map provided by Burton Landscape Architecture Studio and reproduced as the base map for Figure 2.



GEOTECHNICAL PROFESSIONALS INC.

CONE PENETROMETER

FIGURE A-1



Date performed:8-15-13

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.

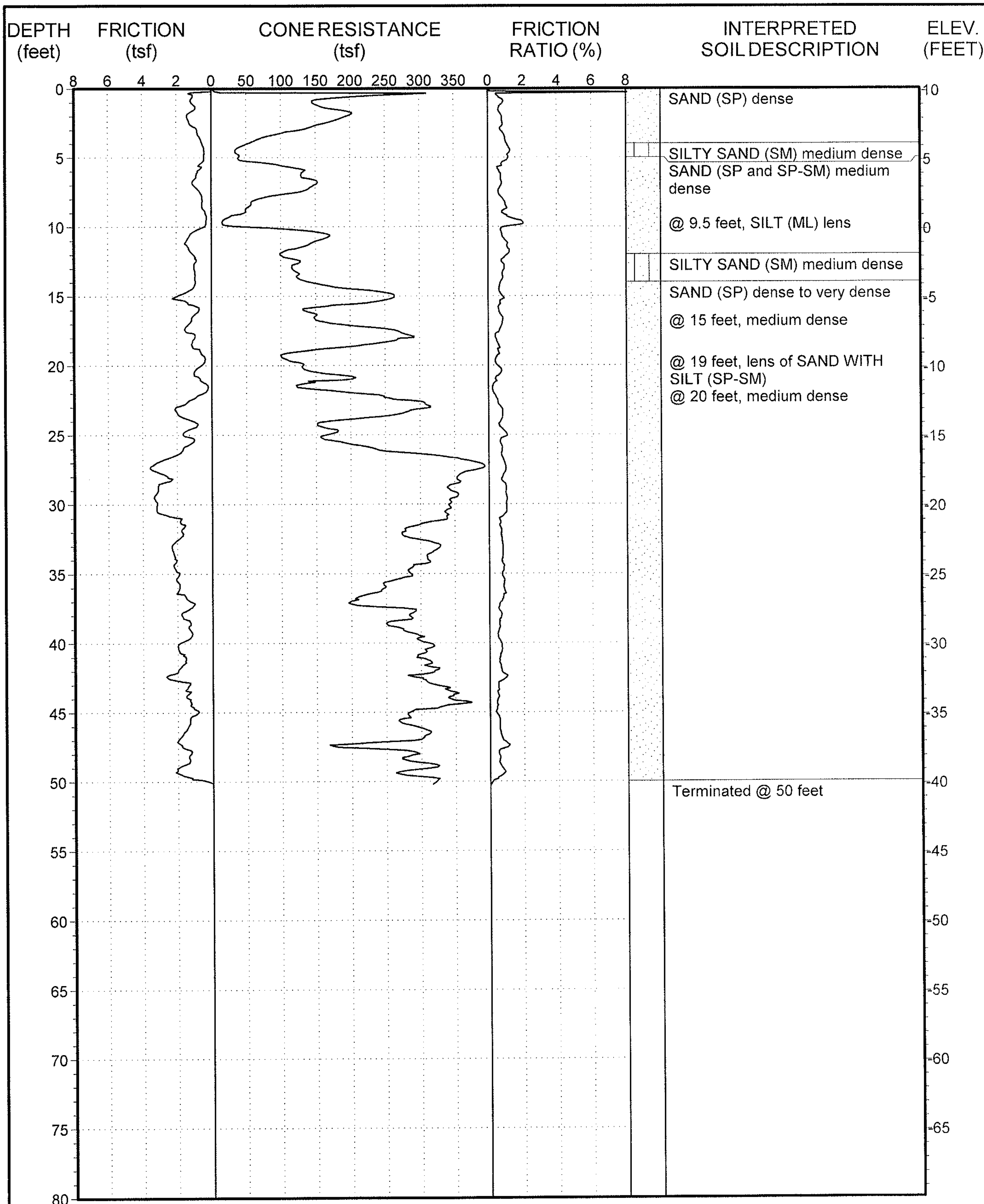
GPI

PROJECT NO.: 2569.I

BALBOA MARINA RESTAURANT

LOG OF CPT NO. C-1

FIGURE A-2



Date performed: 8-15-13

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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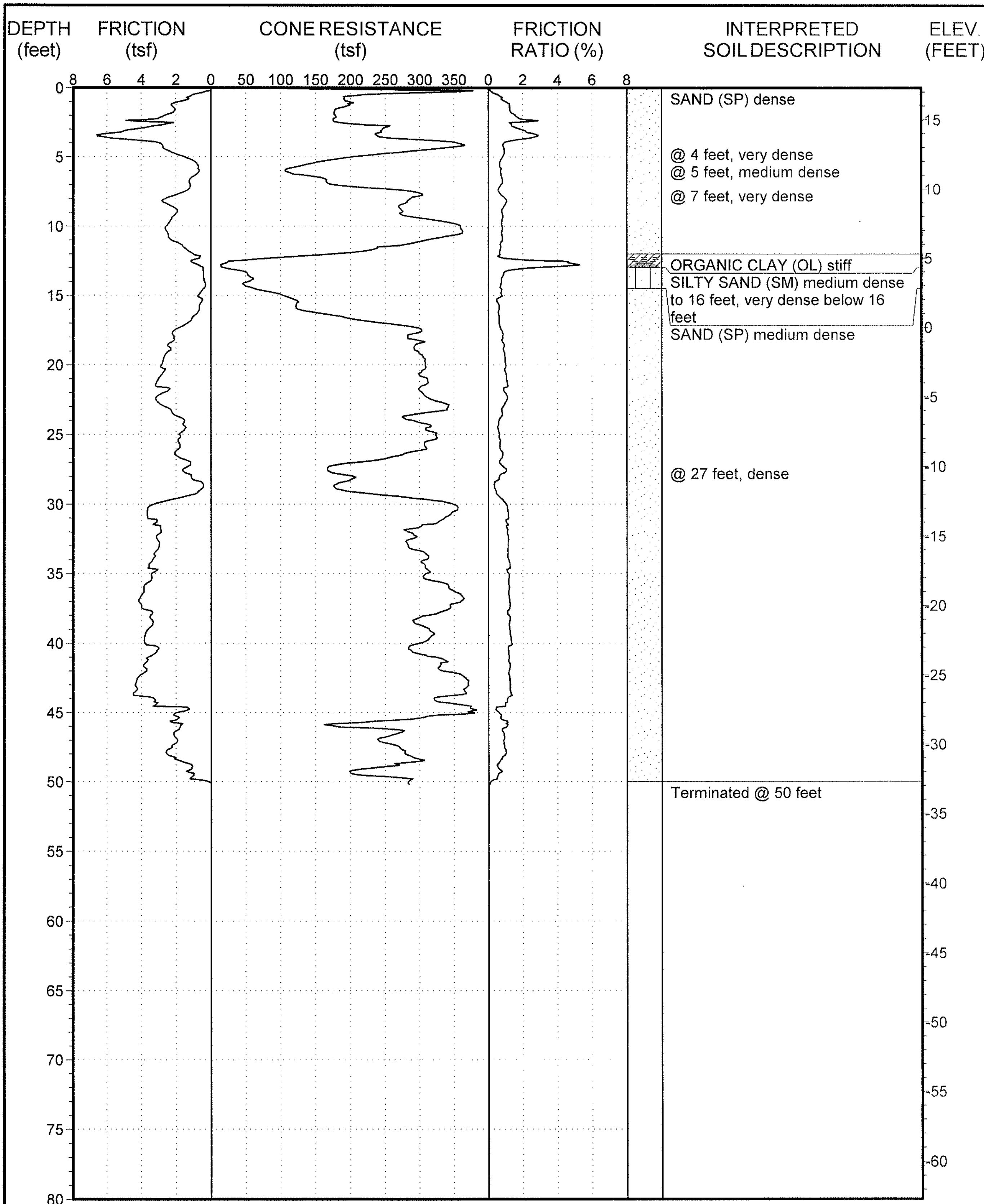
BALBOA MARINA RESTAURANT

LOG OF CPT NO. C-2

FIGURE A-3



Date performed: 8-15-13

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.

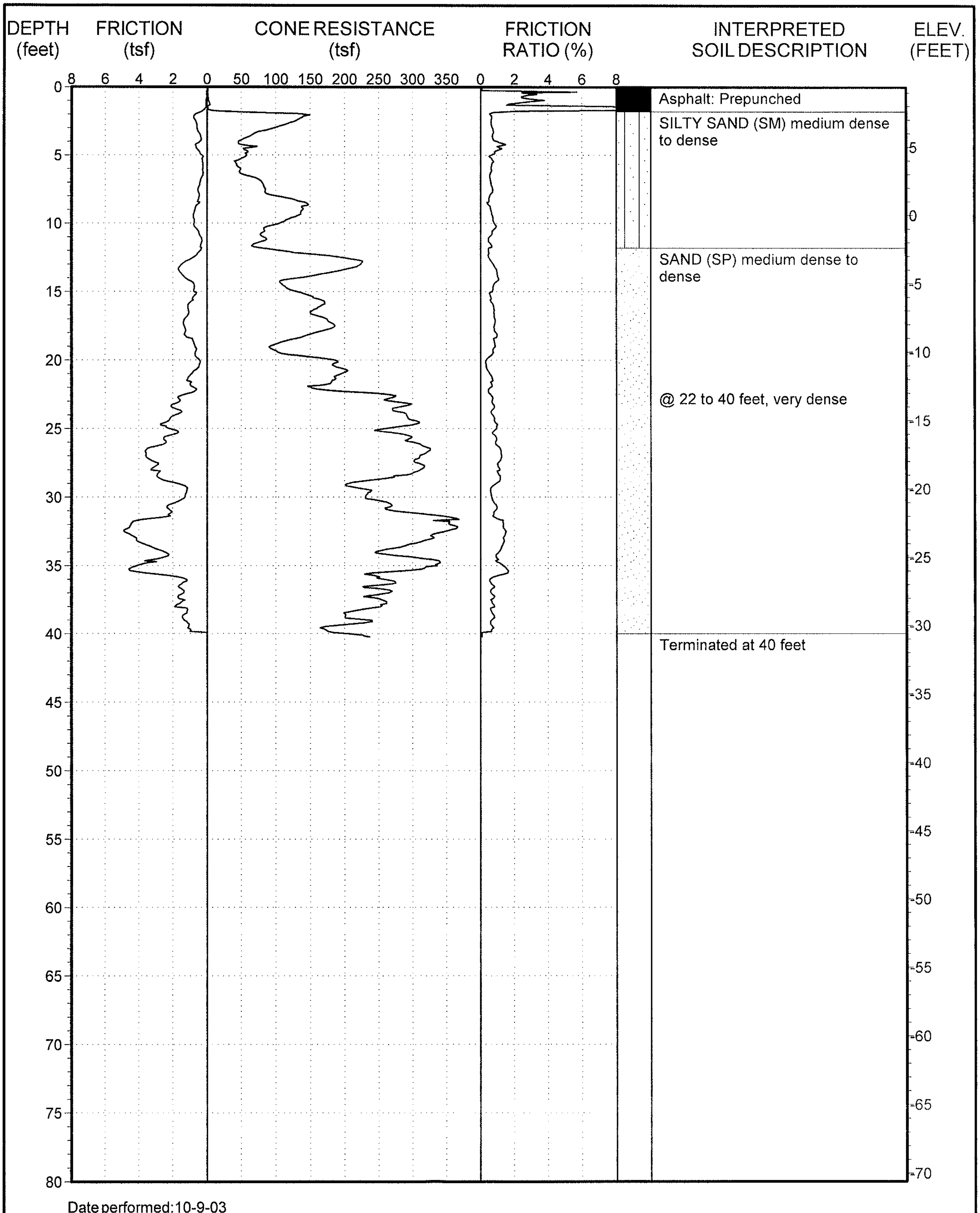
GPI

PROJECT NO.: 2569.I

BALBOA MARINA RESTAURANT

LOG OF CPT NO. C-3

FIGURE A-4



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.

GPI

PROJECT NO.: 1917.I
BALBOA MARINA

LOG OF CPT NO. C-4

FIGURE A-5

APPENDIX B

APPENDIX B

EXPLORATORY BORINGS

Two hand auger borings were performed to depths of 7.0 to 7.5 feet to obtain bulk and drive samples of soils at shallow depths for laboratory testing and to measure the depth to the groundwater level. The two hand auger borings, designated HA-1 and HA-2, were excavated next to CPT's C-1 and C-2, respectively, as shown in Figure 2.

The soils encountered in the borings were logged by a geotechnical technician in accordance with ASTM D2488. Logs of the hand auger borings are presented in Figures B-1 and B-2. The ground surface elevations were obtained from the topographic map provided by Burton Landscape Architecture Studio and reproduced as the base map in Figure 2.

Relatively undisturbed samples of soils at selected depth intervals were obtained in accordance with ASTM D3550 using a brass ring-lined sampler. The brass rings have an inside diameter of 2.42 inches. The sampler was driven into the soil by a 35-pound hammer dropping 20 inches. The number of blows needed to drive the sampler 12 inches was recorded as the penetration resistance. However, it should be noted that the number of blows in this case is much higher than the Standard Penetration Test (ASTM D1586) blowcount because of the lower energy level.

Soil samples for laboratory testing were also obtained at selected depth intervals in two Geoprobe borings located next to the two hand auger borings and CPT's. The sampler was lined with plastic liners approximately 1½ inches in diameter and driven to the top of the selected depth interval, while a conical tipped piston covered the tip of the sampler. Then the outer sampler casing was driven to obtain the sample. These borings were only logged at the sample intervals. A summary of the soils sampled is presented below.

GEOPROBE NO.	DEPTH (feet)	SOIL DESCRIPTION	MOISTURE CONTENT (%)	DRY UNIT WEIGHT (pcf)
GP-1	5.0-6.2	Silty fine sand (SM) and lean clay (CL)	---	
	11.0-12.0	Silty sand (SM)	24.7	100
	31.0-32.2	Elastic silt (MH)	52.3	72
GP-2	4.0-5.2	Sand with silt (SP-SM)		
	8.5-9.6	Sand with silt (SP-SM)	18.4	102
	12.0-13.3	Silty Sand (SM)	25.8	97
	18.5-19.6	Sand with Silt (SP-SM)	15.3	102

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	7" Asphalt	10
					SILTY SAND (SM) brown, moist, trace gravel	
					SAND (SP) brown, moist	
					@ 2.5 feet, trace clay	
					@ 3 feet, trace clay	
					SILTY SAND (SM) brown, moist, medium dense	
28.8	74	49	D	5		5
151.6	31				**ORGANIC CLAY WITH PEAT (OL)** grey, wet, soft	
44.9						
					SAND WITH SILT (SP-SM) grey, very moist	
26.2	96	82/10"	D		@ 6.5 feet, wet, medium dense	
					Total Depth 7.5 feet	
						0

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

DATE DRILLED:
8-15-13

EQUIPMENT USED:
4" Hand Auger

GROUNDWATER LEVEL (ft):
7

GPI

PROJECT NO.: 2569.I

BALBOA MARINA RESTAURANT

LOG OF BORING NO. HA-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)
					0		6" Asphalt	10
							SILTY SAND (SM) brown, moist, with sand	
							SAND (SP) brown, moist, with shells	
	11.1	89	53	D			**SAND WITH SILT (SP-SM)** brown, moist, medium dense	
					5		**SILTY SAND (SM)** brown, very moist	5
							SAND (SP) brown, very moist	
	24.6	94	105	D			@ 6 feet, medium dense to dense	
							Total Depth 7 feet	

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

DATE DRILLED:
8-15-13

EQUIPMENT USED:
4" Hand Auger

GROUNDWATER LEVEL (ft):
7

GPI

PROJECT NO.: 2569.I

BALBOA MARINA RESTAURANT

LOG OF BORING NO. HA-2

FIGURE B-2

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 and tabulation in Appendix B.

ATTERBERG LIMITS

Liquid and plastic limits were determined for a sample of cohesive material in accordance with ASTM D 4318. The results of the Atterberg Limits test are presented in Figure C-1.

GRAIN SIZE DISTRIBUTION

Four 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 below and on the following page.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
GP-1	11	Silty Sand (SM)	13
GP-2	9	Sand With Silt (SP-SM)	7
GP-2	12	Silty Sand (SM)	14
GP-2	19	Sand With Silt (SP-SM)	11
HA-1	7	Sand With Silt (SP-SM)	7
HA-2	4	Sand With Silt (SP-SM)	6
HA-2	6	Sand (SP)	3

DIRECT SHEAR

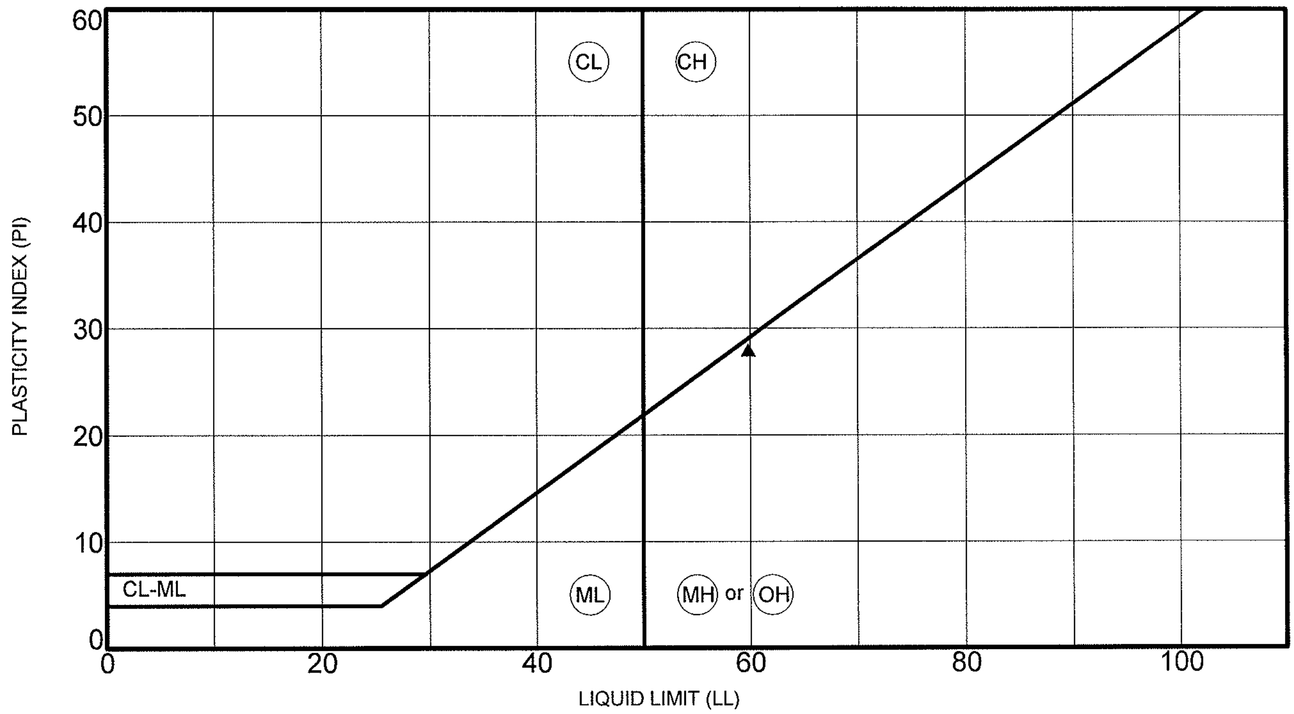
Direct shear tests were performed on an undisturbed sample in accordance with ASTM D 3080. The sample was placed in the shear machine, and pre-selected normal loads were applied. The sample was submerged, allowed to consolidate, and then was sheared to failure. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear test are presented in Figure C-2.

CONSOLIDATION

One-dimensional consolidation tests were performed on an undisturbed sample in accordance with ASTM D 2435. After trimming the ends, the sample was placed in the consolidometer and loaded to up to 0.4 ksf. Thereafter, the sample was incrementally loaded to a maximum load of 3.2 ksf. The sample was inundated at 0.4 ksf. Sample deformation was measured to 0.0001 inch. Rebound behavior was investigated by unloading the sample back to 0.2 ksf. Results of the consolidation test, in the form of percent consolidation versus log pressure are presented in Figure C-3.

SOIL CORROSIVITY TESTING

Soil corrosivity testing was performed by A.P. Engineering and Testing on a soil sample provided by GPI. Test results are presented at the end of this Appendix.



	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
▲	GP-1	32.0	60	32	28		ELASTIC SILT (MH)

PROJECT: BALBOA MARINA RESTAURANT

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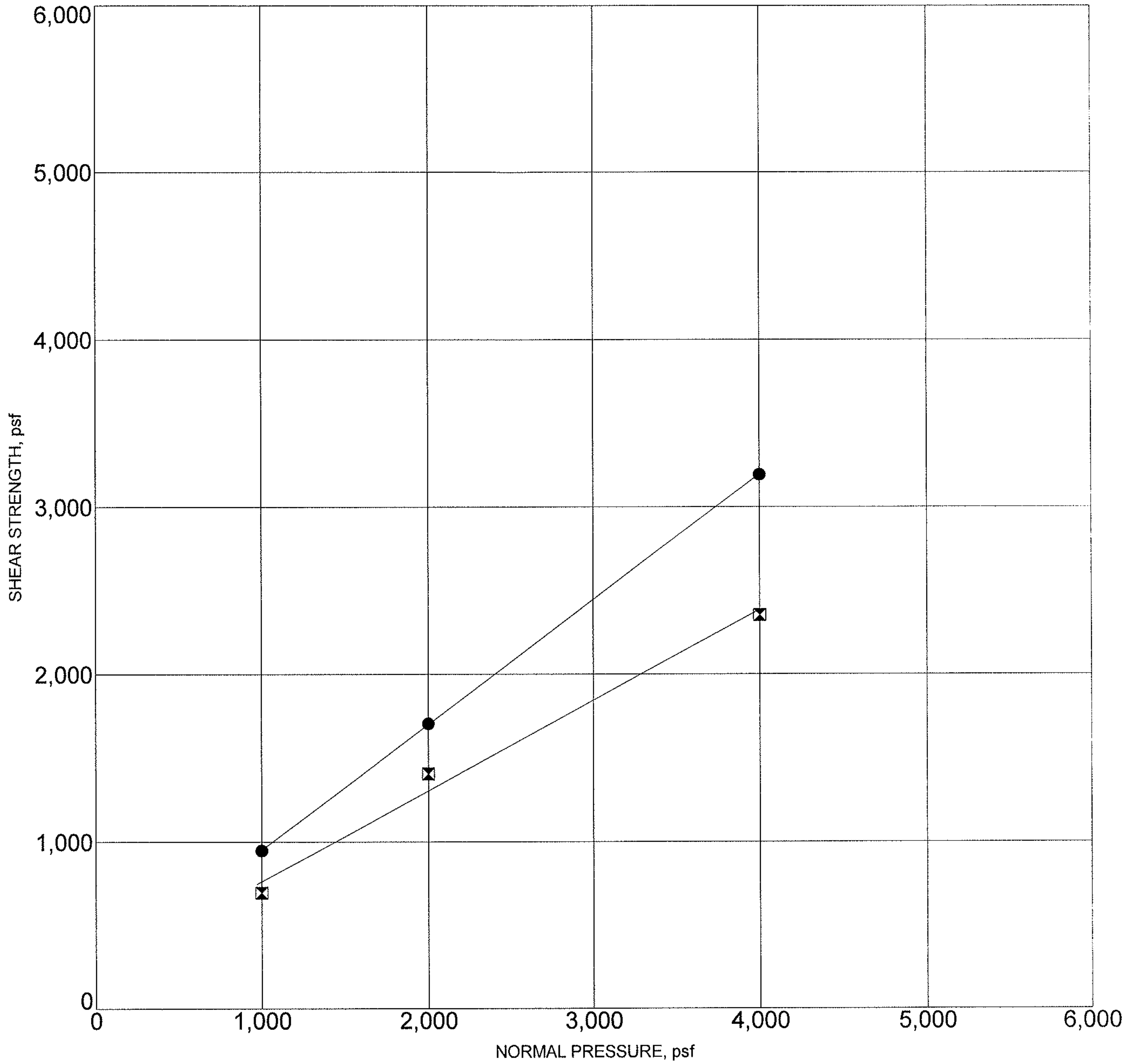
ATTERBERG LIMITS TEST RESULTS

FIGURE C-1



● **PEAK STRENGTH**
Friction Angle= 37 degrees
Cohesion= 204 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 28 degrees
Cohesion= 222 psf

Sample Location		Classification	DD,pcf	MC,%
HA-2	6.0	SAND WITH SILT (SP-SM)	94	24.6

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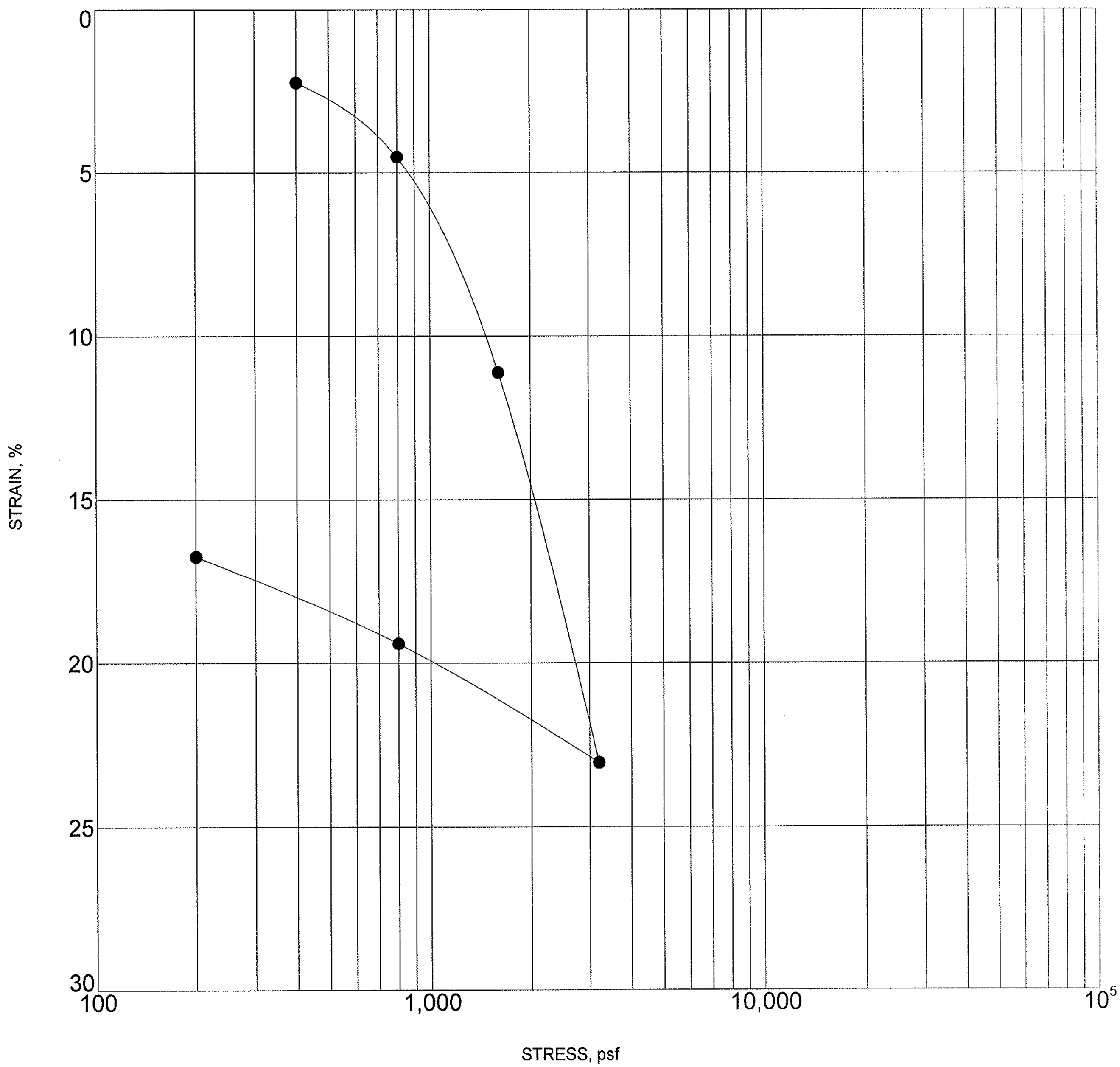
DIRECT SHEAR TEST RESULTS

FIGURE C-2



Sample inundated at 400 psf

	Sample Location		Classification	DD,pcf	MC,%
●	HA-1	5.0	ORGANIC CLAY WITH PEAT (OL)	74	28.8

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CONSOLIDATION TEST RESULTS

FIGURE C-3

AP Engineering & Testing, Inc.

CORROSION TEST RESULTS

Client Name: Geotechnical Professionals, Inc.
Project Name: Balboa Restaurant
Project No.: 2569.1

AP Job No.: 13-1037
Date 10/15/13

Boring No.	Sample No.	Depth (feet)	Soil Type	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
HA-1	-	6-7.5	SP-SM	468	7.2	1863	360

NOTES:
Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

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