APPENDIX B

Geotechnical Investigation

GEOTECHNICAL
PROFESSIONALS INC.

GEOTECHNICAL INVESTIGATION
MIXED-USE DEVELOPMENT
NWC OF AVALON BOULEVARD AND
CARSON STREET
CARSON, CALIFORNIA

Prepared for:
Faring Property Group Inc.
8899 Beverly Boulevard, Suite 716
West Hollywood, California 90048

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

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for a mixed-use development, to be located northwest of the intersection of North Avalon Boulevard and East Carson Street in Carson, California. The project site location is shown on the Site Location Map, Figure 1.

The proposed project will consist of eastern and western parcels combining apartments, retail/commercial buildings, and parking structures. The architectural plans provided by tsk indicate that the eastern structure will have three floors of apartments (wood-frame) over a three level podium (with one basement level) of parking and commercial/retail use. The western structure will have four floors of apartments (wood-frame) surrounding a five-level parking structure (all above grade). The currently proposed site configuration is presented in the Site Plan, Figure 2. Based on information recently provided by Mr. Simon Ha, we understand that the eastern parts of the site may also be developed with apartments wrapping around a parking structure, which may have a subterranean level. Updated plans were not available at the time this report was prepared.

At the time this report was prepared structural design was in a preliminary stage and detailed foundation loads were not available. Preliminary general structural loads provided by Mr. Tony Ghodsi of Englekirk Partners indicate average dead plus sustained live loads of 190 psf per level for the parking structure and 80 psf per floor for the apartment buildings. Based on this information, maximum loads for the western parking structure columns are anticipated to range from 500 to 1000 kips for exterior and interior columns respectively. Maximum wall loads for the four-story apartment buildings (wood-frame) are expected to be less than 8 kips per lineal foot. For the eastern podium structure, maximum column and wall loads are expected to be less than 800 kips and 12 kips per lineal foot, respectively

We have assumed the proposed grades will be within 2 feet of the existing site grades, except for the proposed basement. New surface drives are also planned.

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

2.0 OBJECTIVE AND SCOPE OF WORK

The primary objective of this investigation was to provide geotechnical input for the design of foundations for the proposed structures.

The scope of our geotechnical investigation included field investigation, laboratory testing, geotechnical engineering analyses and preparation of this report.

Our field investigation consisted of five cone penetration tests (CPT's) and three hollow-stem auger borings. The locations of the subsurface explorations are shown on the Site Plan, Figure 2. The CPT's were performed to depths of 50 to 70 feet below the existing ground surface. The borings were performed to depths of 45 to 51 feet below the existing ground surface. Details of the field explorations and logs of the CPT's and borings are presented in Appendices A and B, respectively.

Laboratory tests were performed on selected representative soil samples as an aid in soil classification and to evaluate the engineering properties of the soils. Our laboratory testing included determinations of moisture content and dry density, Atterberg Limits, grain size distribution, shear strength (direct shear), compressibility, compaction (maximum density/optimum moisture), expansion potential, and soil corrosivity. Laboratory test procedures and results are presented in Appendix C.

Soil corrosivity testing was performed by HDR under subcontract to GPI. Their test results are presented at the end of Appendix C.

Engineering evaluations were performed to provide earthwork criteria, foundation and retaining wall design parameters, and preliminary pavement sections as well as an assessment of seismic hazards. The results of our evaluations are presented in the remainder of the report.

3.0 SITE CONDITIONS

3.1 EXISTING SITE CONDITIONS

The full site is approximately 5 acres bounded on the north by existing light industrial developments, on the east by Avalon Boulevard, on the south by Carson Street, and on the west by existing residential developments. The southeastern corner of the site is currently occupied by a retail center and a gas station. A small office building is located west of the retail center. The remaining area of the site is currently undeveloped and vacant.

Based on a topographic ALTA survey map for the site, the ground surface across the subject site is relatively flat with changes in grade from approximate elevations of +22 to +24 feet (MLLW).

The existing ground surface in the developed portion of the site is paved. Boring B-3 disclosed a pavement section consisting of approximately 6-inches of asphalt concrete overlying 8-inches of aggregate base course. The remaining undeveloped areas have light weed growth.

3.2 SUBSURFACE SOILS

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

Fill soils were encountered to depths of 3 feet below existing grades. The fill soils consisted predominantly of moist to very moist clays and silts, and in a limited area silty sands. The fill soils and shallow underlying natural soils appear to be weaker than the deeper native material. Documentation regarding the placement and compaction of the fill was not available.

In general, the natural soils consist of layered sandy clays, clays, silts, silty sands, and sands to a depth of approximately 35 feet below the existing grade. The fine-grained soils (clays and silts) within this depth were very stiff to hard and very moist to wet. The sands and silty sands within this depth were medium dense to very dense and very moist to wet. These soils generally exhibited moderate strength and compressibility characteristics.

Between depths of 35 to 50 feet, the natural soils were predominantly wet and stiff silts and clays, becoming very stiff to hard below depths of 50 feet.

3.3 GROUNDWATER CONDITIONS

The groundwater levels at the site were evaluated by stabilized piezometric pore pressure measurements obtained in cone penetration tests (CPT's) and water level measurements in boreholes upon completion of borings.

Piezometric pore pressure measurements in sandy layers within the upper 50 feet below the ground surface indicated groundwater levels 25 to 29 feet below the ground surface.

Piezometric measurements in sand layers at depths greater than 50 feet indicated groundwater levels 35 to 38 feet below the ground surface. Groundwater levels measured in boreholes were 31 to 32 feet below the ground surface. These measurements indicate that, at the present time, the groundwater levels in shallow sand layers, which are more pertinent for the proposed development, are shallower than those at greater depths. This difference is more likely due to pumping of groundwater from deeper aquifers but could also be due to perched groundwater conditions.

A map of historically highest groundwater contours published by the State of California (CDMG, 1998) indicates that the depth to groundwater has been as shallow as 20 feet below grade in the vicinity of the site. For our geotechnical evaluations, we considered the groundwater level to be at an average of 20 feet below the existing ground surface.

3.4 CAVING

Caving was not observed during the removal of the small diameter CPT's and hollow-stem auger borings. Caving is anticipated in the sandy soils below groundwater.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 OVERVIEW

Based on the results of our geotechnical investigation, the site conditions are suitable for developing the site as proposed. The proposed structures can be supported on shallow foundations following limited remedial grading to mitigate the geotechnical constraints discussed below. The most significant geotechnical issues that will affect the design and construction of the proposed structures are as follows:

- The undocumented fill and natural soils within approximately the upper five feet at the site are not considered to be suitable for uniform support of new building foundations. 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.

- The upper clayey soils have moderate potential for expansion (E.I of 50+). Our earthwork recommendations in Section 4.3 and floor slab design recommendations in Section 4.5 present measures for mitigating the potential for expansion.

- Retaining wall backfill should consist of granular, non-expansive fills. Based on our findings, such material is not anticipated to be available in significant quantities within the upper 10 feet below existing grades, even with selective grading.

- Moisture contents of the near surface soils (within 6 feet of existing grades) are up to 10 percent over optimum. Therefore, drying of these materials prior to placement as fill or backfill should be expected. Also, rubber-tire equipment may cause "pumping" and disturbance of the subgrade soils. The contractor should evaluate the in-place moisture conditions when planning the work to allow for moisture conditioning and reducing subgrade disturbance.

- Chemical testing of the near surface clayey soils indicates that they are severely corrosive to metals and have soluble sulfate contents up to 5,105 mg/kg (+0.5% by weight), which corresponds to a severe level of sulfate exposure for soil.

Based on our investigation, we conclude that the development, during construction and after completion, will be safe from adverse landslides, settlement and/or slippage. In addition, development of the site will not adversely affect off-site property, from a geotechnical engineering standpoint. These conclusions are based on the expectation that the site earthwork will be performed in accordance with our recommendations (Los Angeles County, Section 111 Statement).

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 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 California Building Code, 2013 edition. For the 2013 CBC, a Site Class D may be used. The seismic code values can be obtained directly from the tables in the building code using the above values and appropriate United States Geological Survey web site (earthquake.usgs.gov). The seismic design method should be determined by the Project Structural Engineer.

4.2.2 Strong Ground Motion Potential

Based on published information (Risk, 2013), the most significant fault in the proximity of the site is the Newport-Inglewood Fault, which is located about 3.2 kilometers (2.0 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 0.63g for a magnitude 7.0 earthquake. This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from ASCE 7-10 (ASCE, 2010) and a site coefficient (F_{PGA}) based on site class. The structural design will need to incorporate measures to mitigate the effects of strong ground motion.

4.2.3 Potential for Ground Rupture

There are no known faults crossing or projecting through the site. The site is not located in an Alquist-Priolo Special Studies zone. Therefore, ground rupture due to faulting is considered unlikely at this site.

4.2.4 Liquefaction

Soil 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 cohesionless soils. Thus, three conditions are required for liquefaction to occur: (1) a cohesionless soil of loose to medium density; (2) a saturated condition; and (3) rapid large strain, cyclic loading, normally provided by earthquake motions.

The site is located just outside but near an area mapped as having a potential for soil liquefaction, in accordance with the Seismic Hazards Mapping Act as shown in the Torrance Quadrangle (CDMG, 1999). As defined by the Act, characteristics of the site require investigation for the potential hazard and, if a hazard exists, that its effects be mitigated. The published depth to groundwater at the site is 20 feet historically, while the recent geotechnical investigation at the site disclosed a groundwater depth of up to 25 feet below the existing grade. A groundwater depth of 20 feet was used in our evaluation.

Revisions to the 2013 California Building Code, ASCE 7-10 and Special Publication 117A (CGS, 2008) require that the ground motion used for this evaluation be based on the Peak Ground Acceleration (PGA_M) adjusted for site class effects. Utilizing the United States Geological Survey (USGS) web site (earthquake.uss.gov) and in accordance with the 2013 CBC, we considered a MCE_G peak ground acceleration of 0.63g for a magnitude 7.0 earthquake (Newport-Inglewood) for our analyses, which corresponds to the PGA_M obtained using the method described above.

The potential for liquefaction was evaluated using the methods presented by the NCEER and updated by Robertson (Robertson, 2009) and modifications provided in Special Publication 117A.

The majority of the soils encountered in our CPT's and borings are very stiff to hard clays and silts with lesser deposits of medium dense to dense sands and silty sands. Criterion for liquefaction susceptibility of the fine-grained soils was based on methods presented in Bray and Sancio (2006).

Based on our findings, the potential for liquefaction induced seismic settlement to adversely affect the proposed project is considered to be low. Total and differential seismic settlement is estimated to be less than ½-inch and ¼-inch, respectively.

4.2.5 Seismic Ground Subsidence

Seismic ground subsidence, not related to liquefaction, occurs when loose, sandy soils (above the groundwater) are densified during strong earthquake shaking. Earthquake-induced seismic subsidence during a strong earthquake is estimated to be less than ¼-inch.

4.3 EARTHWORK

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

4.3.1 Clearing and Grubbing

Prior to grading, the areas to be developed should be cleared of all debris. Any buried foundations of demolished structures, abandoned utility lines, buried tanks, and other underground structures should be removed in their entirety. 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 sand-cement slurry. The upper 5 feet should be backfilled with compacted soil. All deleterious material generated during the clearing operations, including all debris, trash or organic material, should be removed from the site.

4.3.2 Excavations

Excavations at the site will include removals of undocumented fill soils and upper loose natural soils, footing and basement excavations, and trenching for proposed utility lines.

Prior to placing fills or construction of the proposed buildings or other foundation support improvements, undocumented fills and weaker natural soils occurring within the proposed building areas should be removed and replaced as properly compacted fill. For structures being founded with at-grade first floor levels, removals should extend to depths of at least 5 feet below existing grades or 2 feet below the base of planned foundations, whichever is deeper. The actual depths of removals should be determined in the field during grading by GPI.

Overexcavations are not anticipated for structures with basements.

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 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.

In proposed pavement areas, the existing near-surface soils should be removed to a depth of 2 feet below existing grades or one-foot below finished subgrade, whichever is deeper, and replaced as properly compacted fill.

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 which 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 cuts up to 12 feet deep, the cuts should be properly shored or sloped back to at least ¾:1 or flatter. For cuts deeper than 12 feet, the slopes should be properly shored or sloped back at least 1½:1 or flatter. The slope inclination should be measured from the toe to the top of the excavation. 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.

Surcharge loads should not be permitted within a horizontal distance equal to the height of cut from the top of the excavation or 5 feet from the top of the slopes, whichever is greater, unless the cut is properly shored. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site facilities should be properly shored to maintain support of adjacent elements. All 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 8 inches, moisture-conditioned as necessary, and compacted to at least 90 percent of the maximum dry density in accordance with ASTM D 1557.

4.3.4 Material for Fill

The on-site soils are, in general, suitable for use as compacted fill. However, the on-site clays and silts are not suitable for use as backfill behind retaining walls. The clays encountered should not be used as fill directly beneath concrete slabs-on-grade. Material that is suitable for use as retaining wall backfill does not appear to be available on-site in abundance.

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 at least 20 if used in proposed paved areas. We should be provided with a sample (at least 50 pounds) and notified of the location of any soils proposed for import at least 72 hours in prior to importing. Each proposed import source should be sampled, tested and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by GPI may be rejected if not suitable.

Both imported and existing on-site soils, to be used as fill, should be free of deleterious debris and any 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. Granular (sandy) 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 content of the fill materials should be between optimum and 2 percent over optimum moisture conditions at the time of compaction. The moisture content of the existing near surface soils are generally above optimum and may require some drying prior to compaction. The contractors should allow for drying of these materials and stabilization of the subgrade in their bids.

During backfill of excavations, the fill should be properly benched into the construction slopes as it is placed in lifts. Any fills placed within 2 feet of retaining walls should be compacted using light equipment, such as wackers, in order to limit lateral earth pressures on the walls.

4.3.6 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 5 to 10 percent and subsidence of 0.1 feet may be assumed for the natural soils. 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. GPI should observe and test all trench and wall backfills.

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 FOUNDATIONS

4.4.1 Foundation Type

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

4.4.2 Allowable Bearing Pressures

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

4.4.3 Minimum Footing Width and Embedment

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

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

4.4.4 Estimated Settlements

For the anticipated building loads, total settlement of the more heavily loaded column footings (1000 kips) is expected to be on the order of ¾- to 1-inch. Maximum differential settlements between similarly loaded adjacent footings or along a 40-foot span of a continuous footing are expected to be about ¼- to ½-inch.

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

4.4.5 Lateral Load Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of footings and underlying soils and by passive soil pressures acting against the embedded sides of the footings. For frictional resistance, a coefficient of friction of 0.35 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 315 pounds per cubic foot may be used, provided the footings are poured tight against compacted fill soils. These values may be used in combination without reduction.

4.4.6 Foundation Concrete

Laboratory soil corrosivity testing by HDR Schiff on two samples of site soils from the upper 11 feet indicated soluble sulfate content up to 5,105 mg/kg (0.51 percent by weight). Based on these results, foundation concrete should conform to the requirements outlined by ACI 318, Section 4.3 and the 2013 CBC for severe sulfate exposure.

4.4.7 Footing Excavation Observation

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

4.5 BUILDING FLOOR SLABS

Slab-on-grade floors should be supported on at least 12 inches of non-expansive sandy soils, compacted soils as discussed in the "Placement and Compaction of Fill" section. The on-site soils encountered in the upper 5 feet of the ground surface can be used below the 12 inches of sand.

A vapor/moisture barrier should be placed under any slabs that are to be covered with moisture-sensitive floor coverings (parquet, vinyl tile, etc.). Currently, common practice is to use 10-mil polyethylene as a vapor barrier, placed either directly on the subgrade or over a thin layer of sand. Recently, other types of vapor barriers with much lower permanence 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 could be considered for this project. This material should be covered by a layer of clean sand (less than 5 percent by weight passing the No. 200 sieve) having a minimum thickness of 2 inches. The function of the sand layer is to protect the vapor barrier 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 only slightly moist. If the sand gets wet (for example, as a result of rainfall) it must be allowed to dry prior to placing concrete.

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) as well as excess moisture in the concrete. The manufacturer of floor coverings should be consulted for establishing acceptable criteria for the condition of floor surface prior to placing moisture-sensitive floor coverings.

For elastic design of a slab-on-grade supporting sustained concentrated loads, a modulus of subgrade reaction (k) of 50 pounds per cubic inch (pounds per square inch per inch of deflection) may be used, provided the recommended earthwork is performed. The structural design of the floor slabs should consider both long-term loads related to operations and short-term construction loads.

For lateral resistance design, a coefficient of friction value of 0.35 between the properly compacted subgrade soils and concrete may be used. For a slab on a visqueen moisture barrier, a coefficient of 0.1 should be used.

4.6 LATERAL EARTH PRESSURES

Based on information available to us at the time this report was prepared, basements were planned for the site. The following recommendations are provided for walls no more than 12 feet in height. We recommend that walls be backfilled with imported non-expansive granular soils as adequate sources do not appear to be available on-site.

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 imported granular soils (no more than 40 percent passing No. 200 U.S. standard sieve), the magnitude of active pressures are equivalent to the pressures imposed by a fluid weighing 35 pounds per cubic foot (pcf). This pressure may also be used for the design of temporary excavation support.

At-rest pressures should be used for restrained walls that remain rigid enough to be essentially non-yielding. At-rest pressures imposed by a fluid weighing 50 pounds per cubic foot should be used for drained granular backfill.

Walls subject to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge pressure for unrestrained and restrained walls, respectively. The upper 10 feet of basement walls adjacent to driveways should be designed to resist a uniform lateral pressure of 150 pounds per square foot, acting as a result of the surcharge induced by normal street traffic. If traffic is kept at least 10 feet from the wall, the traffic surcharge may be neglected.

If walls need to be designed to include seismic forces, a seismic increment of 32H in pounds per square foot (where H is equal to the height of the wall) may be added to the static lateral earth pressures to estimate seismic loading.

These values may also be used in the design of shoring against the native clayey soils.

The wall backfill should be well-drained to relieve possible hydrostatic pressure or designed to withstand these pressures. A drain consisting of perforated pipe and gravel wrapped in filter fabric should be used. One cubic foot of rock should be used for each lineal foot of pipe. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top. We prefer pipe and gravel drains to weep holes to avoid potential for constant flow of surface water in front of the wall. The subsurface drains should be connected to piping separate from the surface drains to allow discharge of the relatively low volume subsurface water without risking introduction of larger volume stormwater into the subdrainage system.

The Structural Engineer should specify the use of select, granular wall backfill on the plans. The granular fill should extend a distance horizontally behind the wall equal to at least one-half the height of the wall. Wall footings should be designed as discussed in the "Foundations" section.

4.7 CORROSIVITY

Resistivity testing of a representative sample of the on-site soils indicates that they are severely corrosive to metals. GPI does not practice corrosion protection engineering. If buried metal pipe is to be used, a corrosion engineer such as HDR should be consulted.

4.8 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. We do not recommend the placement of landscape planters directly 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 the clayey soils encountered in the slab-subgrade should not be permitted.

4.10 PAVED AREAS

Preliminary pavement design has been based on an assumed R-value of 5 due to the clayey near-surface soils. 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. Final pavement design should be based on R-value testing performed near the conclusion of rough grading.

The following pavement sections are recommended for planning purposes only.

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		ASPHALT/PORTLAND CONCRETE	AGGREGATE BASE COURSE
Asphalt Concrete			
Automobile Parking	4.0	3.0	7
Automobile Drives	5.0	3.0	10
Truck Drives	6.0	3.5	13
Portland Cement Concrete			
Automobile Parking	4.0	7.5	----
Automobile Drives	5.0	8.0	----
Truck Drives	6.0	8.0	----

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.

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 STORMWATER INFILTRATION

Current regulations require that storm water be infiltrated into the site soils of new developments, when possible. The soil types present at the site control the ability of water to infiltrate into the subgrade. Based upon our subsurface investigation, the subsurface natural soils underlying the site consist predominately of clays and silts, which typically do not have sufficient permeability to accept infiltration.

During our investigation, groundwater was measured at levels as shallow as 25 feet below existing grade. Historical high groundwater levels, as discussed earlier, are as shallow as 20 feet below the ground surface. As required by the County of Los Angeles (GS200.1), the lowest point of infiltration must be at least 10 feet above the groundwater table. This requires infiltration to occur within the upper 10 to 15 feet of the subsurface soils.

The clayey soils encountered within these depths can be expected to have infiltration rates significantly below the 0.3-inches per hour minimum required for the subsurface discharge of storm water. In addition, the clayey soils are moderately expansive. Introduction of water into expansive soils will cause ground heave, which could damage slabs and hardscapes.

Due to the presence of clayey soils and shallow groundwater, infiltration of stormwater is not considered feasible for the site.

4.12 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 Faring Property Group, Inc. 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.

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

Byron Konstantinidis, G.E.
Principal

DJB/BK:sph

REFERENCES

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

Bray, J.D. and Sancio, (2006), "*Assessment of the Liquefaction Susceptibility of Fine-Grained Soils*," Journal of Geotechnical and Geoenvironmental Engineering, ASCE, September 2006, pp. 1165-1177.

California Geological Survey, 2008, "Guidelines for Evaluating and Mitigating Seismic Hazards in California," Special Publication 117A

California Department of Conservation, Division of Mines and Geology, 1999, "State of California Seismic Hazard Zones, Torrance Quadrangle, Official Map," dated March 25, 1999.

California Department of Conservation, Division of Mines and Geology, 1998, "Seismic Hazard Zone Report for the Torrance 7.5-minute Quadrangle, Los Angeles County, California," Seismic Hazard Zone Report 035.

County of Los Angeles Department of Public Works, Geotechnical and Materials Engineering Division, "*Guidelines for Design, Investigation, and Reporting Low Impact Development Stormwater Infiltration,*" Administrative Manual GS200.1, dated June 30, 2014.

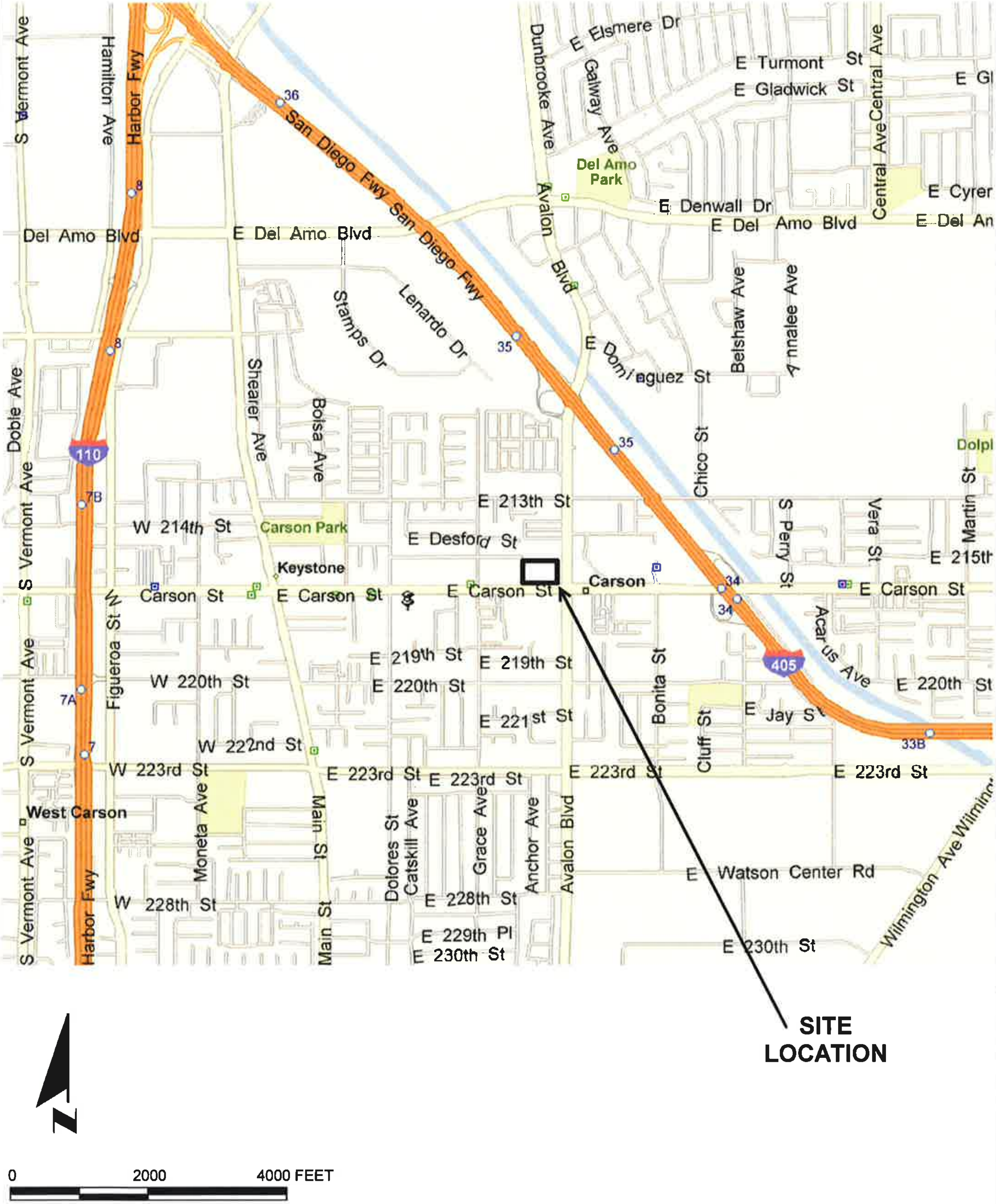
Idriss, I.M., Boulanger, R.W., "*Soil Liquefaction During Earthquakes,*" Earthquake Engineering Research Institute (EERI) Monograph, 2008.

Risk Engineering (2013), "EZFRISKP, Software for Earthquake Ground Motion Estimation, Version 7.62

Robertson, P.K., "*Performance based earthquake design using the CPT",* Keynote Lecture, International Conference on Performance-based Design in Earthquake Geotechnical Engineering – from case history to practice, IS-Tokyo, June 2009.

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

Youd, T.L. and Idriss, I.M. (1997), "Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils," Technical Report NCEER-97-0022.



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

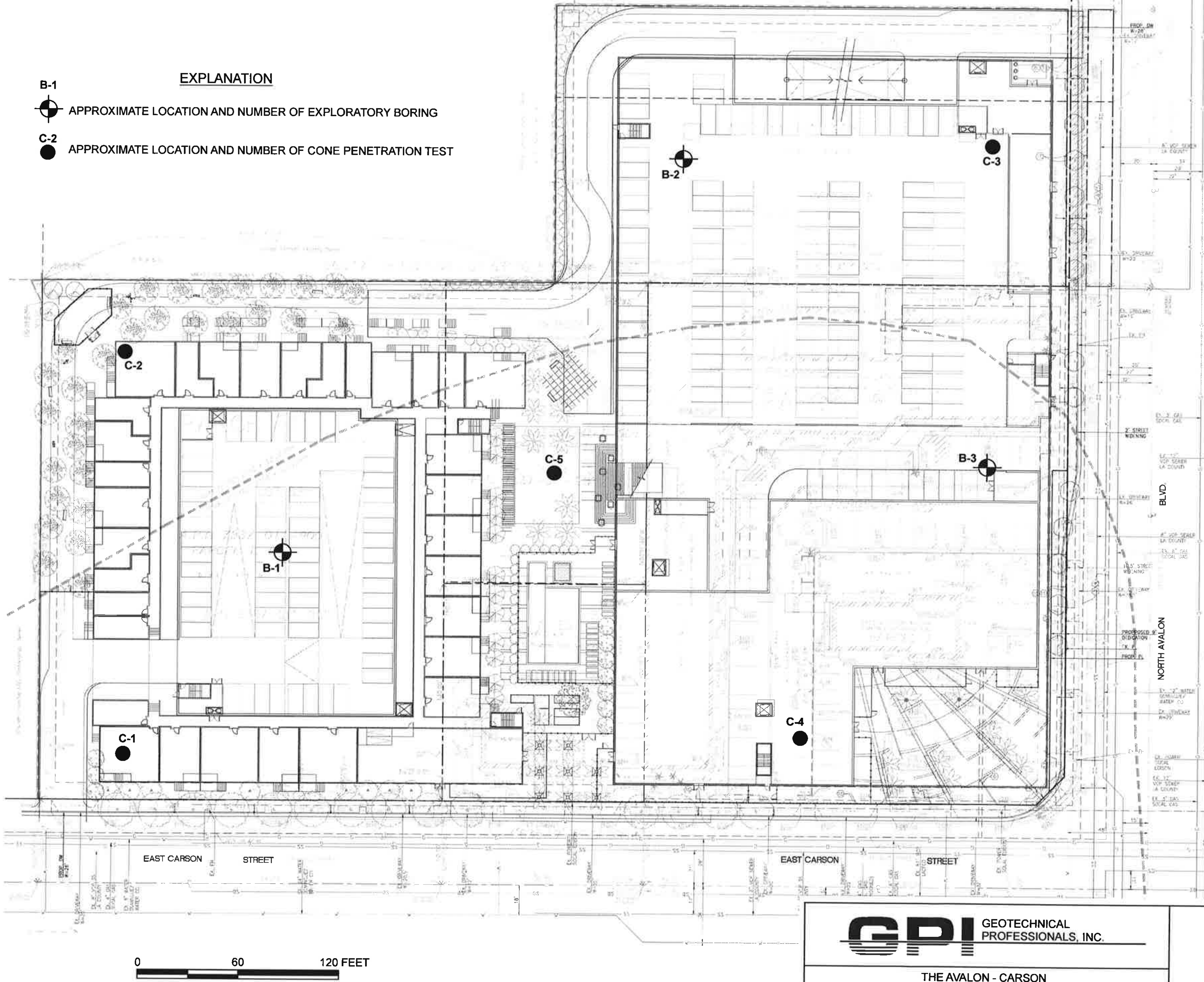
GEOTECHNICAL PROFESSIONALS, INC.

THE AVALON - CARSON

GPI PROJECT NO. 2633.I	SCALE: 1" = 2000'

SITE LOCATION

FIGURE 1



BASE MAP REPRODUCED FROM CIVIL EXHIBIT
PREPARED BY KPFF: DATED 07/29/2014

GPI GEOTECHNICAL PROFESSIONALS, INC.

THE AVALON - CARSON

GPI PROJECT NO.: 2633.I | SCALE: 1" = 60'

SITE PLAN

FIGURE 2

APPENDIX A

APPENDIX A

CONE PENETRATION TESTS

The subsurface conditions were investigated by performing 5 Cone Penetration Tests (CPT's) at the site. The soundings were advanced to depths ranging from approximately 50 to 70 feet below existing grades. The locations of the CPT's 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'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.

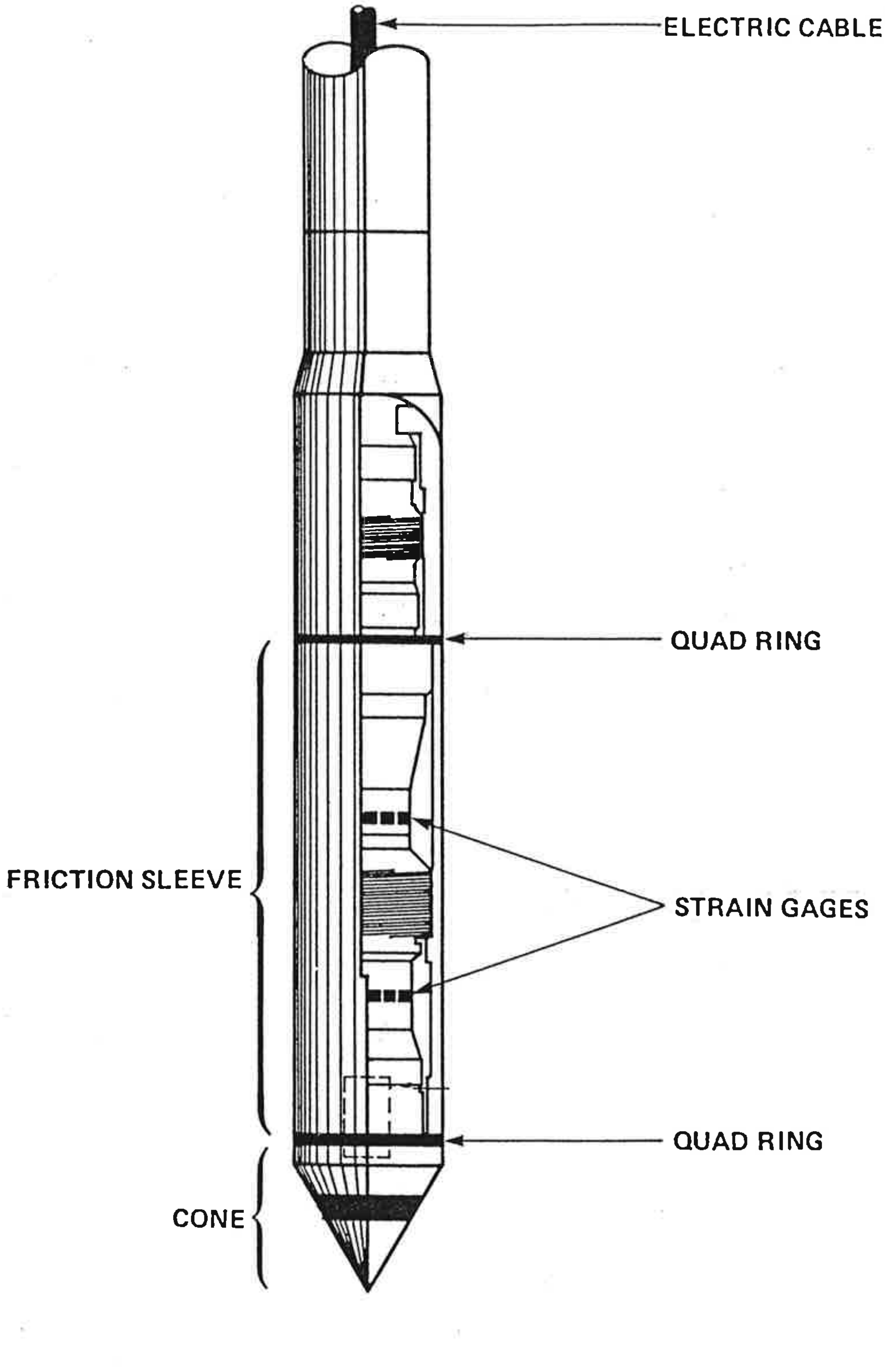
Computer plots of the reduced CPT data acquired for this investigation are presented in Figures A-2 to A-6 of this appendix. The field testing and computer processing was performed by Kehoe Testing and Engineering under subcontract to Geotechnical Professionals Inc. (GPI). The interpreted soil descriptions were prepared by GPI.

At selected sand layers, the penetration was stopped and piezometric pore pressures were measured until stabilized readings were obtained, as an indication of the piezometric groundwater levels. The results of these measurements are summarized below:

CPT No.	Measurement Depth (ft)	Stabilized Pressure (psi)	Piezometric Groundwater Depth (ft)	Piezometric Groundwater Elevation (ft)
C-1	33.9	2.0	29.3	-5.3
C-3	58.5	9.4	36.9	-14.7
C-4	49.8	10.7	25.1	-1.1
C-4	50.8	6.8	35.1	-11.1

CPT No.	Measurement Depth (ft)	Stabilized Pressure (psi)	Piezometric Groundwater Depth (ft)	Piezometric Groundwater Elevation (ft)
C-5	33.3	1.8	29.2	-5.0
C-5	69.4	13.6	38.0	-13.8

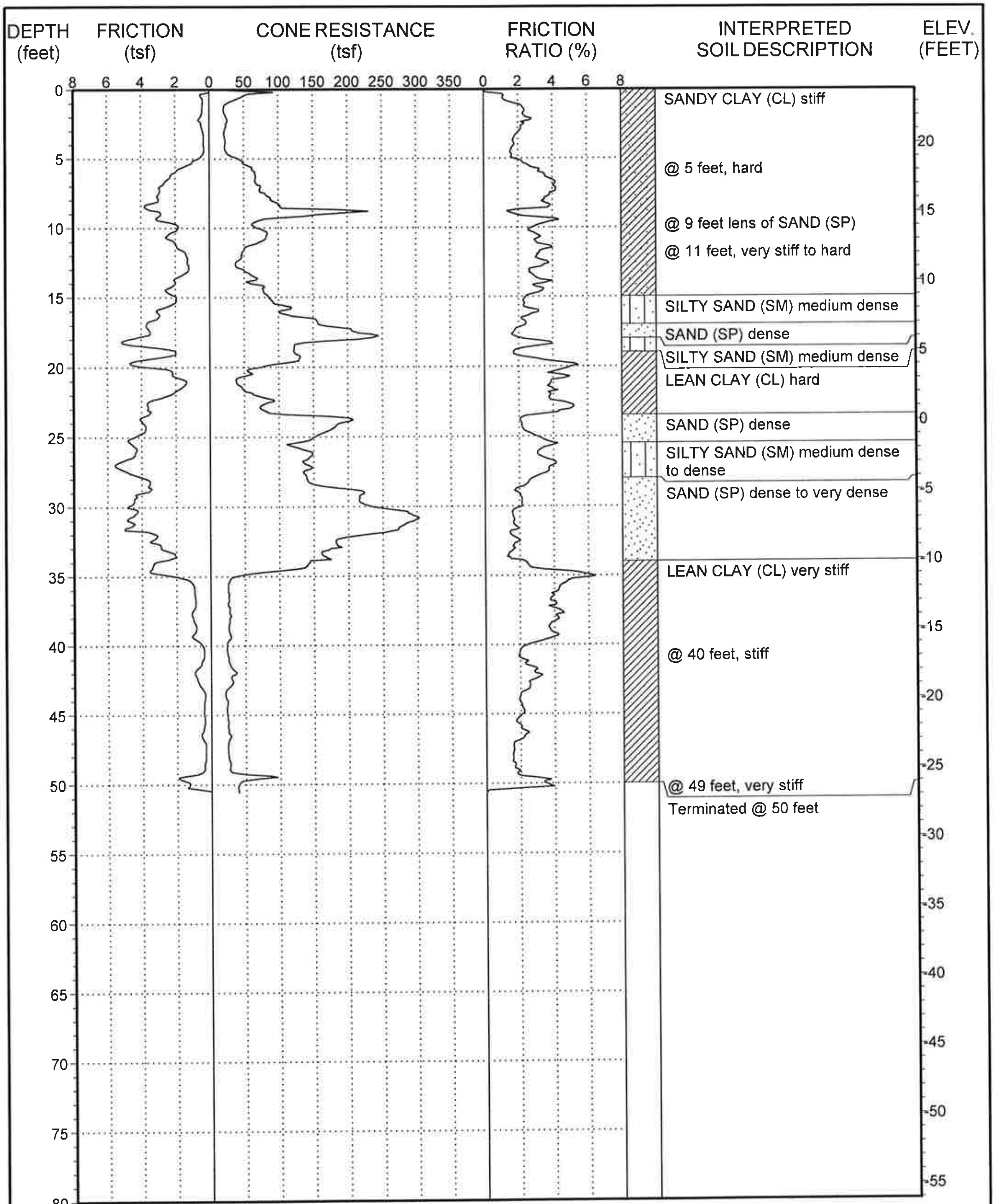
The CPT locations were laid out in the field by measuring from existing site features. Upon completion, the uncaved portions of the CPT holes were backfilled with bentonite chips. CPT's performed in asphalt or concrete areas were patched with cold-patch asphalt or rapid-set concrete, respectively. Ground surface elevations at the CPT locations were estimated from a topographic/ALTA survey.



GEOTECHNICAL PROFESSIONALS INC.

CONE PENETROMETER

FIGURE A-1



Date performed:9-29-14

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

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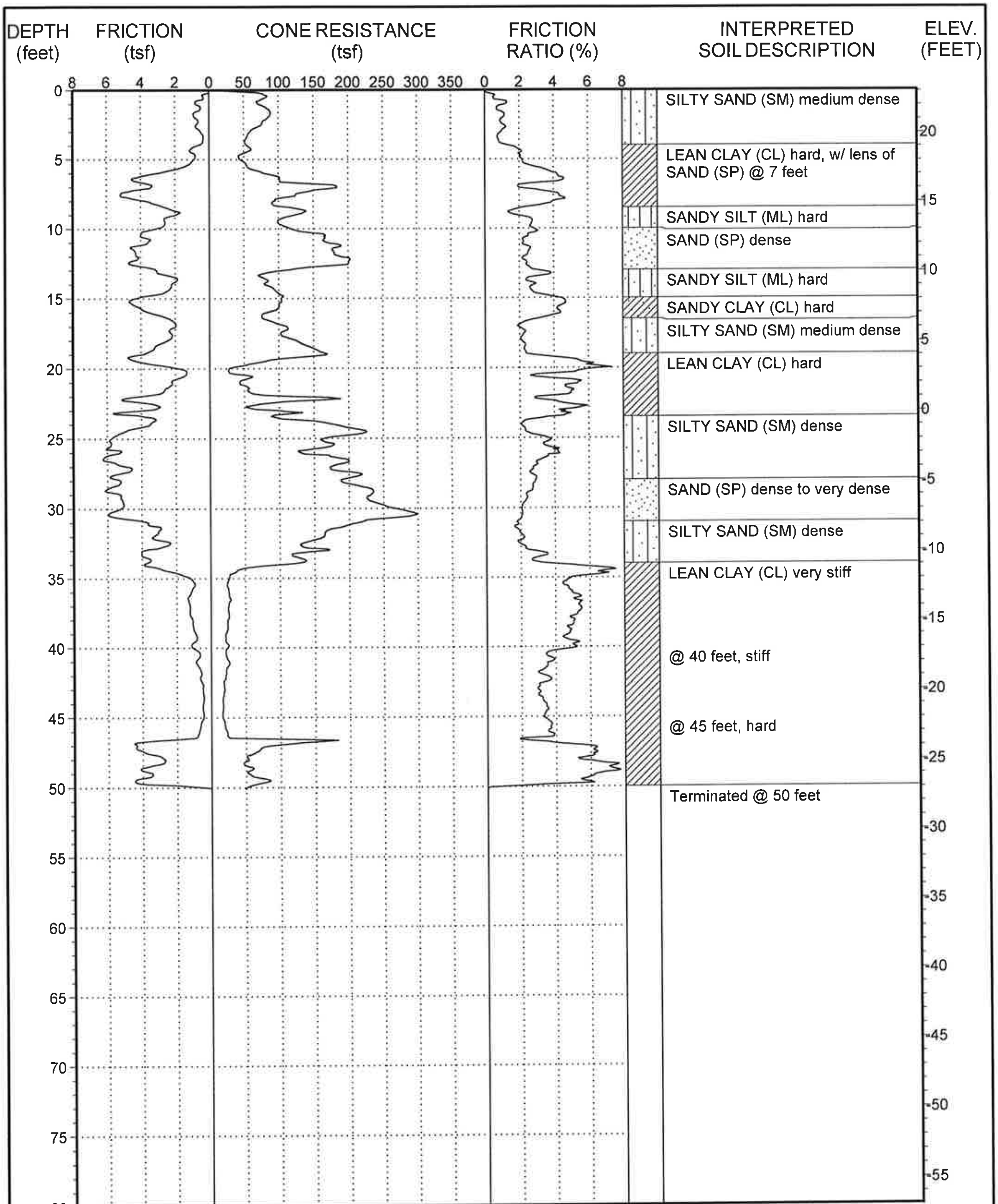
LOG OF CPT NO. C-1

FIGURE A-2



Date performed: 9-29-14

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.

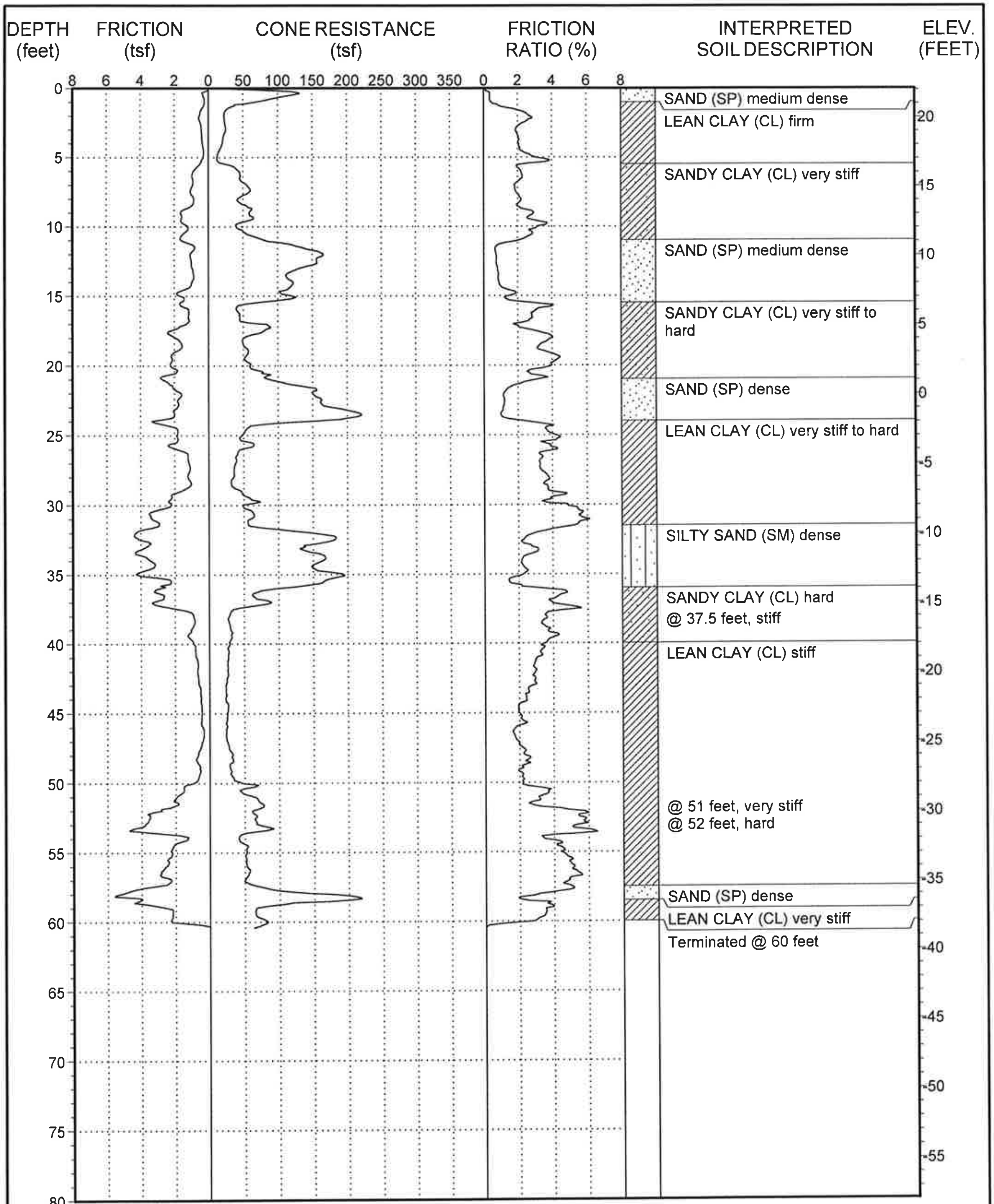
GPI

PROJECT NO.: 2633.I

THE AVALON - CARSON

LOG OF CPT NO. C-2

FIGURE A-3



Date performed:9-29-14

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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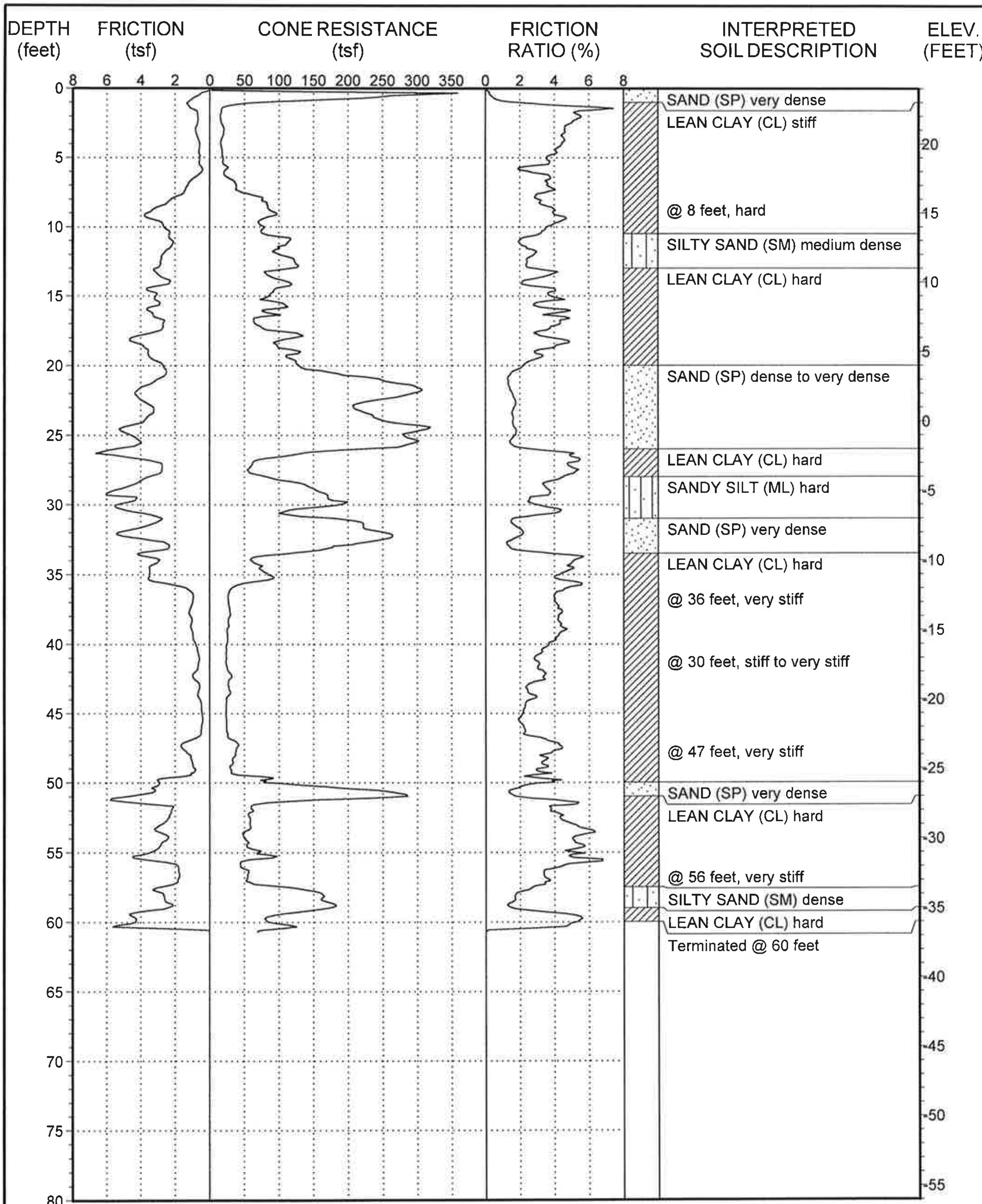
PROJECT NO.: 2633.I
THE AVALON - CARSON

LOG OF CPT NO. C-3

FIGURE A-4



Date performed:9-29-14

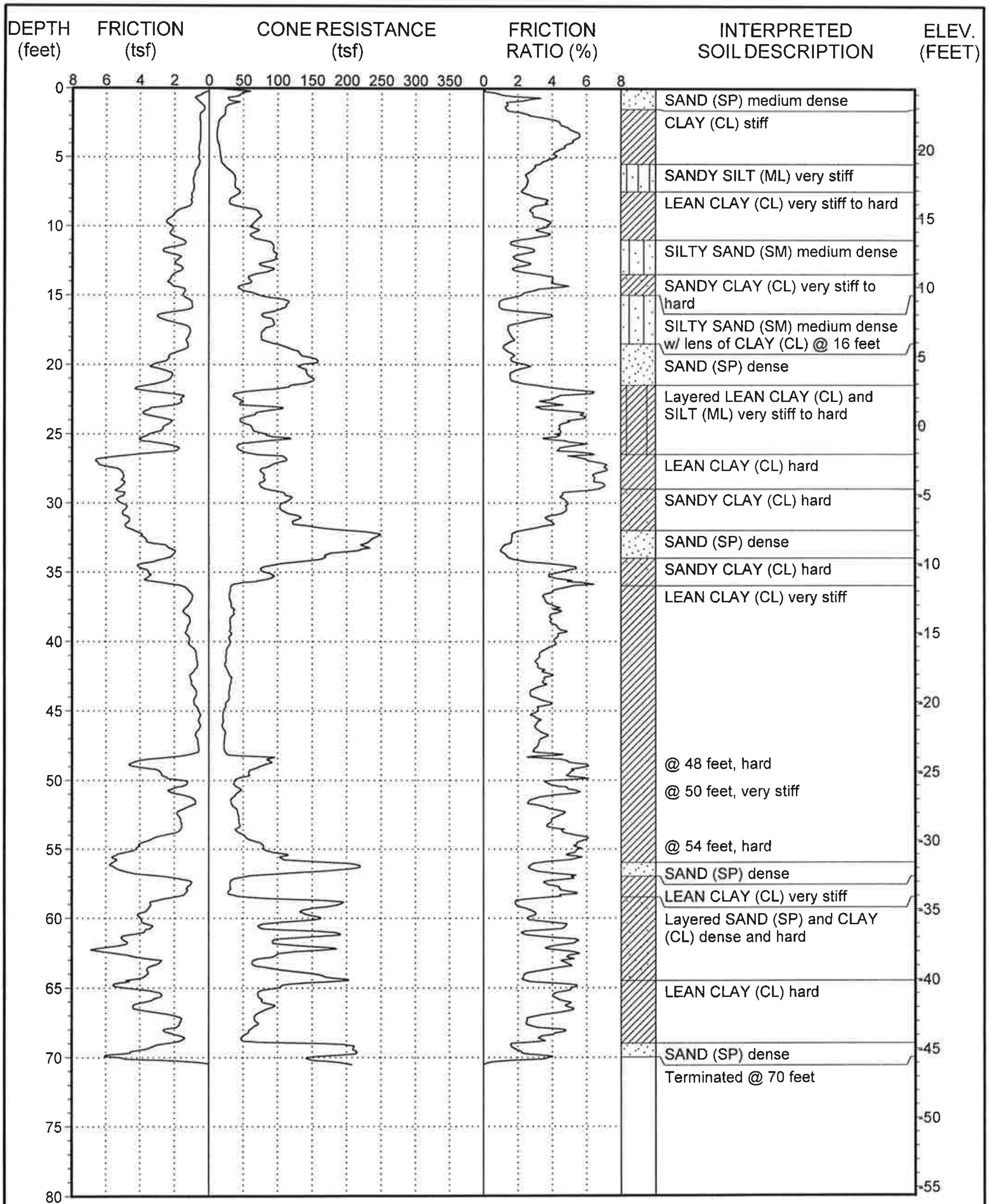
This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance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

GPI

PROJECT NO.: 2633.I
THE AVALON - CARSON

LOG OF CPT NO. C-4

FIGURE A-5



Date performed: 9-29-14

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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PROJECT NO.: 2633.I
THE AVALON - CARSON

LOG OF CPT NO. C-5

FIGURE A-6

APPENDIX B

APPENDIX B

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling three exploratory borings. The borings were advanced to depths ranging from 45 to 51 feet below the existing ground surface. The locations of the explorations are shown on the Site Plan, Figure 2.

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

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-3 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/ALTA survey.

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
			B	0	Fill: **LEAN CLAY (CL)** brown, slightly moist, with gravel, asphalt, and concrete	
11.9	114	52	D		Natural: **LEAN CLAY (CL)** brown, slightly moist to moist, hard	20
				5		
20.2	104	48	D		@ 6 feet, very moist	
			B			
						15
16.6	110	68	D		@ 9 feet, moist	
				10		
11.9	102	45	D		@ 12 feet, slightly moist to moist, very stiff to hard	
						10
				15		
13.0	101	46	D		@ 16 feet, hard	
						5
23.1	101	50	D	20	@ 20 feet, very moist	
						0
16.5	111	59	D	25	**SILTY SAND (SM)** brown, very moist to wet, dense	
						-5
				30		
22.8	101	49	D		@ 31 feet, wet, trace shells	
						-10
				35		
26.3	96	43	D		**LEAN CLAY (CL)** brown, wet, very stiff	
						-15

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

DATE DRILLED:
10-2-14

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL (ft):
31

GPI

PROJECT NO.: 2633.I
THE AVALON - CARSON

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)
	29.1	93	28	D	40	@ 40 feet, brown, wet, very stiff	
							-20
	43.1	78	35	D	45	@ 45 feet, grey	
							-25
	20.5	110	42	D	50	**SANDY CLAY (CL)** grey, very moist, very stiff	
						Total Depth 51 feet	

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

DATE DRILLED:
10-2-14

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL (ft):
31

GPI

PROJECT NO.: 2633.I

THE AVALON - CARSON

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)
			B	0	Fill: **SANDY CLAY (CL)** brown, moist, with gravel and asphalt	
					Natural: **LEAN CLAY (CL)**brown, moist, stiff	
						20
17.0	108	20	D			
				5		
16.2	112	55	D		@ 6 feet, hard	
						15
				10		
13.6	102	38	D		**SILT (ML)** brown, moist, very stiff	
						10
21.6	103	34	D	15	**SANDY CLAY (CL)** brown, very moist, very stiff	
23.6	100	37	D		**LEAN CLAY (CL)** brown, very moist, very stiff	5
				20		
18.5	105	41	D		**CLAYEY SAND (SC)** brown, very moist to wet, medium dense	0
					SANDY SILT (ML) brown, very moist to wet, very stiff	
				25		
18.9	109	40	D		**SANDY CLAY (CL)** brown, very moist, very stiff	
					SANDY SILT (ML) brown, very moist to wet, very stiff	-5
				30		
20.0	98	49	D		**SILTY SAND (SM)** brown, wet, dense, trace shells	-10
				35		
						-15
27.4	96	35	D		**SANDY CLAY (CL)** brown, wet, very stiff	

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

DATE DRILLED:
10-2-14

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL (ft):
32

GPI

PROJECT NO.: 2633.I

THE AVALON - CARSON

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)
					40		-20
	30.3	90	24	D	45	**LEAN CLAY (CL)** grey, wet, very stiff	
						Total Depth 45 feet	

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

DATE DRILLED:
10-2-14

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL (ft):
32

GPI

PROJECT NO.: 2633.I

THE AVALON - CARSON

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	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	5.5" Asphalt, 8" Base	
			B		Fill: **LEAN CLAY (CL)**brown, moist, stiff	
					Natural: **LEAN CLAY (CL)**brown, very moist, stiff	20
22.2	103	14	D			
				5		
13.9	116	34	D		**SANDY CLAY (CL)** brown, moist, very stiff	
			B			15
				10		
14.9	114	40	D		**SILT (ML)** brown, moist to very moist, very stiff	
						10
13.8	117	58	D	15	**SANDY SILT (ML)** brown, moist, hard	
						5
12.7	101	51	D			
				20		
19.8	105	49	D		**LEAN CLAY (CL)** brown, very moist, hard	0
				25		
19.2	108	39	D		**SANDY CLAY (CL)** brown, very moist, very stiff	
						-5
				30		
28.2	92	58	D		**SILTY SAND (SM)** brown, wet, dense	
21.0					**SANDY SILT (ML)** brown, wet, hard	-10
				35		
						-15
30.5	90	30	D		**LEAN CLAY (CL)**brown, wet, very stiff	

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

DATE DRILLED:
10-2-14
EQUIPMENT USED:
8" Hollow Stem Auger
GROUNDWATER LEVEL (ft):
31

GPI

PROJECT NO.: 2633.I
THE AVALON - CARSON

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	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.	
				40	**LEAN CLAY (CL)** brown, wet, very stiff	
						-20
29.6	91	23	D	45		
					Total Depth 45 feet	

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

DATE DRILLED:
10-2-14

EQUIPMENT USED:
8" Hollow Stem Auger

GROUNDWATER LEVEL (ft):
31

GPI

PROJECT NO.: 2633.I

THE AVALON - CARSON

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

ATTERBERG LIMITS

Liquid and plastic limits were determined for select samples in accordance with ASTM D 4318. The results of the Atterberg Limits tests are presented in Figure C-1.

GRAIN SIZE DISTRIBUTION

Three 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 percentage passing the No. 200 sieve is presented below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING NO. 200 SIEVE
B-1	25	Silty Sand (SM)	38
B-1	31	Silty Sand (SM)	16
B-2	32	Silty Sand (SM)	23

DIRECT SHEAR

Direct shear tests were performed on undisturbed samples in accordance with ASTM D 3080. The samples were placed in the shear machine, and a normal load comparable to the in-situ overburden stress was applied. The samples were inundated, allowed to consolidate, and then were sheared to failure. The tests were repeated on additional test specimens under increased normal loads. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear tests are presented in Figures C-2 to C-4.

CONSOLIDATION

One-dimensional consolidation testing was performed on four undisturbed samples in accordance with ASTM D 2435. After trimming the ends, the samples were placed in the consolidometer and loaded to either 0.4ksf. Thereafter, the samples were incrementally loaded to a maximum load of 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 samples back to 0.4 ksf. Results of the consolidation tests, in the form of percent consolidation versus log pressure, are presented in Figures C-5 to C-8.

COMPACTION TEST

A maximum dry density/optimum moisture test was performed in accordance with ASTM D 1557 on a representative bulk sample of the site soils. The test results are as follows:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	OPIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)
B-1	0-3	Clay (CL)	12.0	121

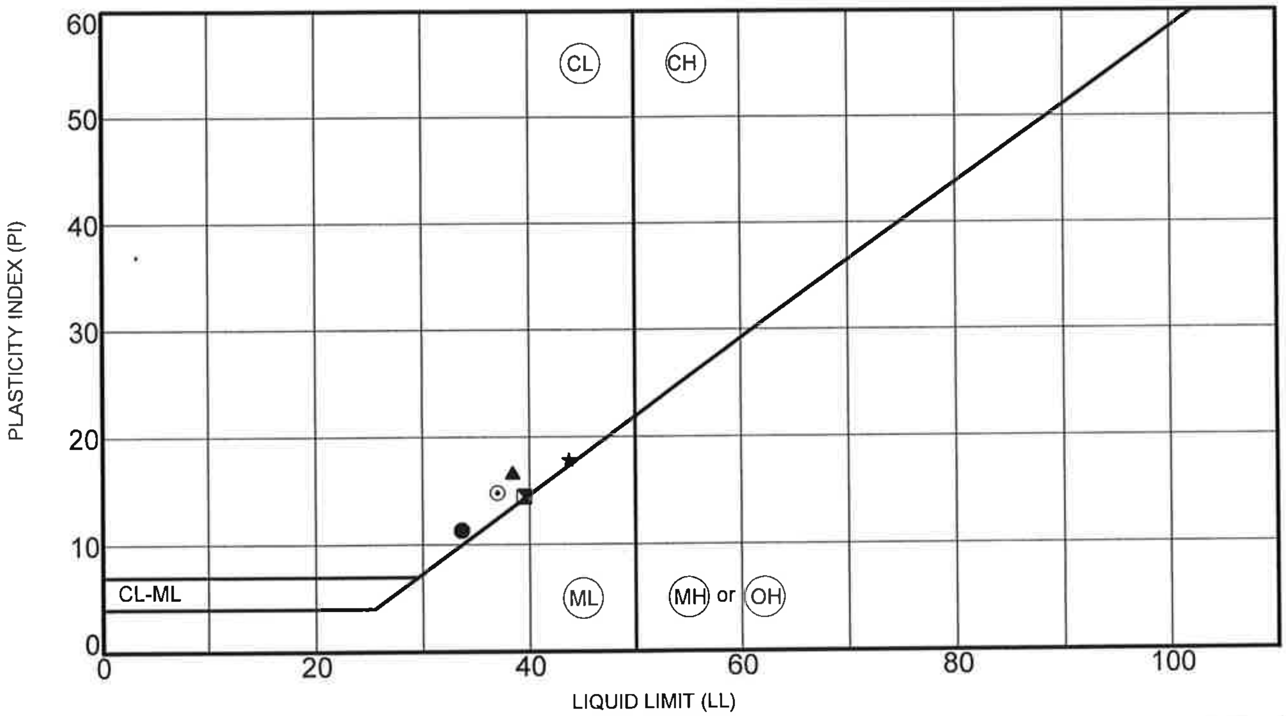
EXPANSION INDEX

Expansion tests were performed in accordance with ASTM D 4829 on bulk samples to assess the expansion potential of the on-site fill soils. The results of the test are summarized below.

BORING N0.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX
B-1	0-3	Clay (CL)	59
B-3	6-11	Sandy Clay (CL)	51

CORROSIVITY

Soil corrosivity testing was performed by HDR on two soil samples provided by GPI. The test results are summarized in Table 1 of this Appendix.



	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
●	B-1	15.0	34	22	11		LEAN CLAY (CL)
⧗	B-1	36.0	40	25	14		LEAN CLAY (CL)
▲	B-1	40.0	38	22	17		LEAN CLAY (CL)
★	B-2	15.0	44	26	18		SANDY CLAY (CL)
⊙	B-2	39.0	37	22	15		SANDY CLAY (CL)

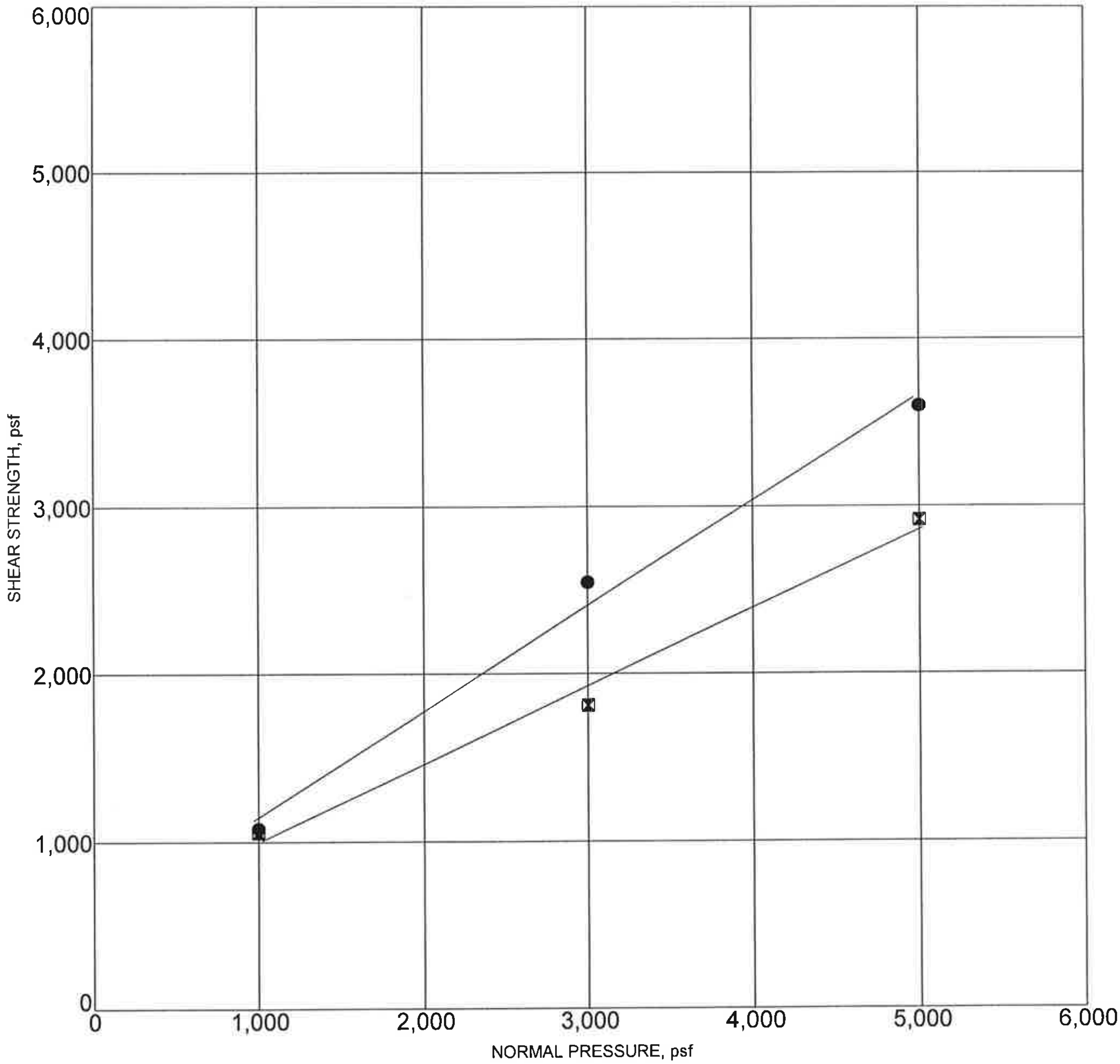
PROJECT: THE AVALON - CARSON

PROJECT NO. 2633.I

GPI

ATTERBERG LIMITS TEST RESULTS

FIGURE C-1



● **PEAK STRENGTH**
Friction Angle= 32 degrees
Cohesion= 518 psf

⊠ **ULTIMATE STRENGTH**
Friction Angle= 25 degrees
Cohesion= 533 psf

Sample Location		Classification	DD,pcf	MC,%
B-1	9.0	LEAN CLAY (CL)	110	16.6

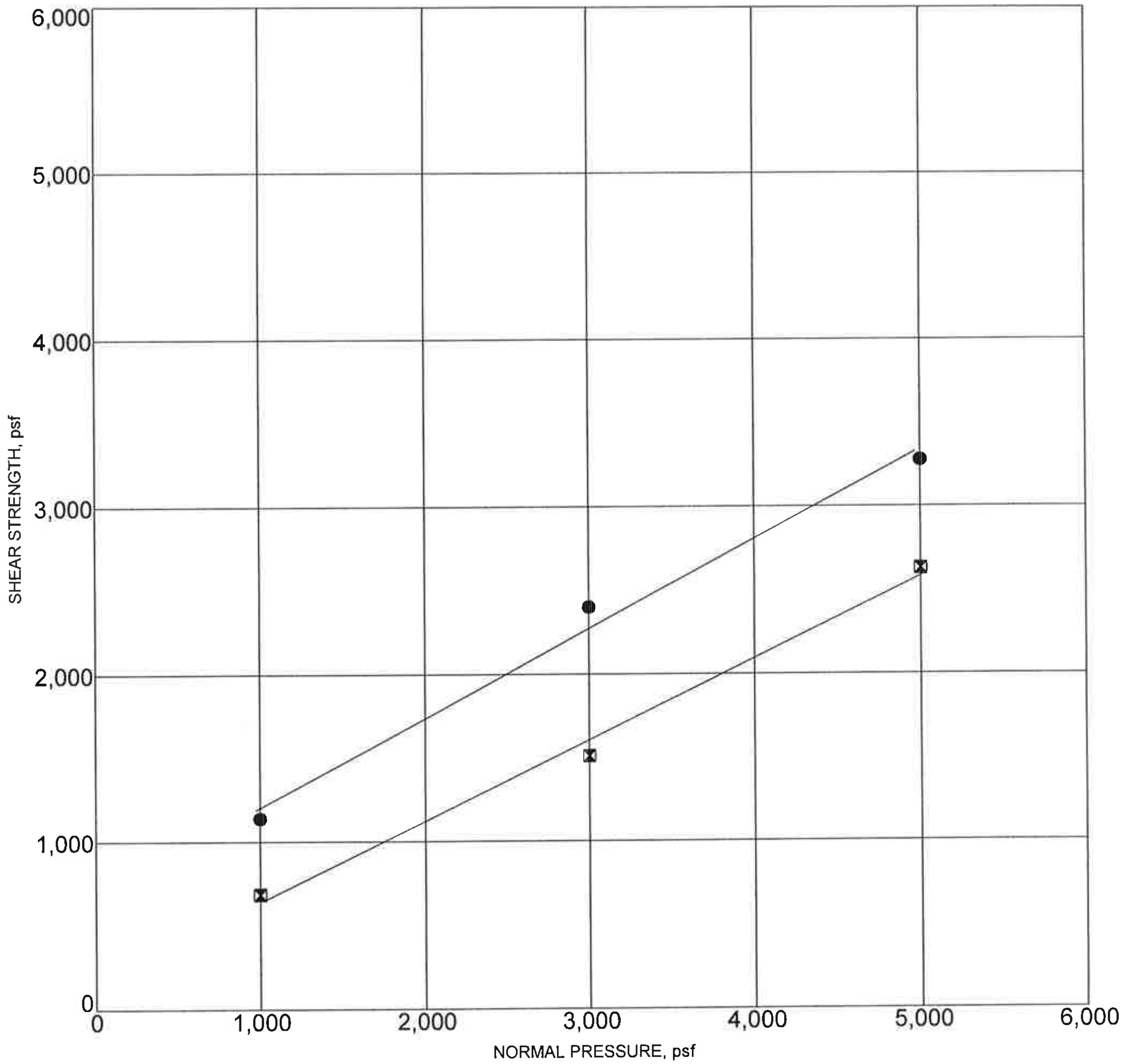
PROJECT: THE AVALON - CARSON

PROJECT NO.: 2633.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE C-2



● **PEAK STRENGTH**
Friction Angle= 28 degrees
Cohesion= 670 psf

⊠ **ULTIMATE STRENGTH**
Friction Angle= 26 degrees
Cohesion= 150 psf

Sample Location		Classification	DD,pcf	MC,%
B-2	18.0	SANDY CLAY (CL)	100	23.6

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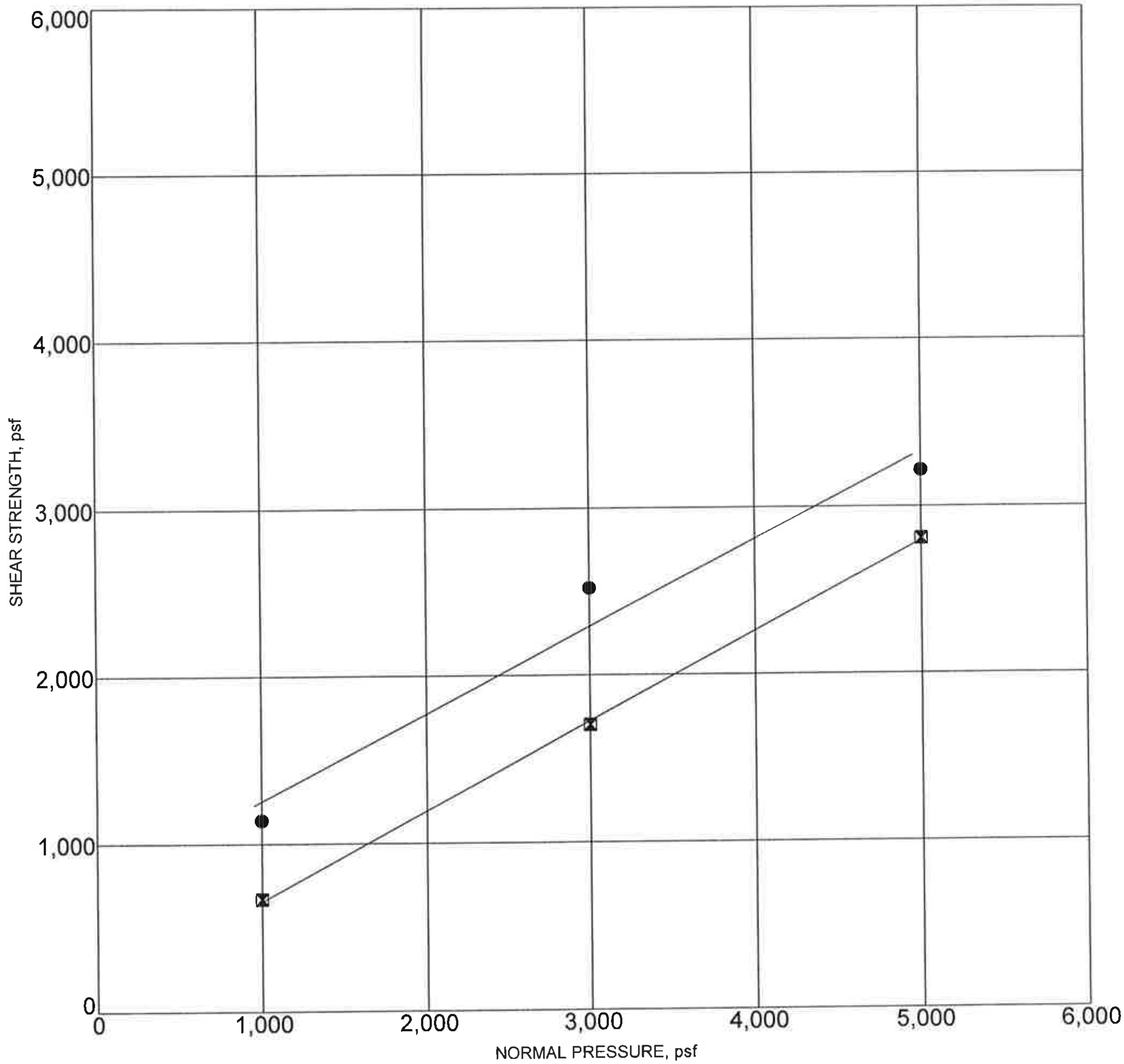
GPI

DIRECT SHEAR TEST RESULTS

FIGURE C-3



● **PEAK STRENGTH**
Friction Angle= 27 degrees
Cohesion= 735 psf

⊠ **ULTIMATE STRENGTH**
Friction Angle= 28 degrees
Cohesion= 126 psf

Sample Location		Classification	DD,pcf	MC,%
B-3	6.0	SANDY CLAY (CL)	116	13.9

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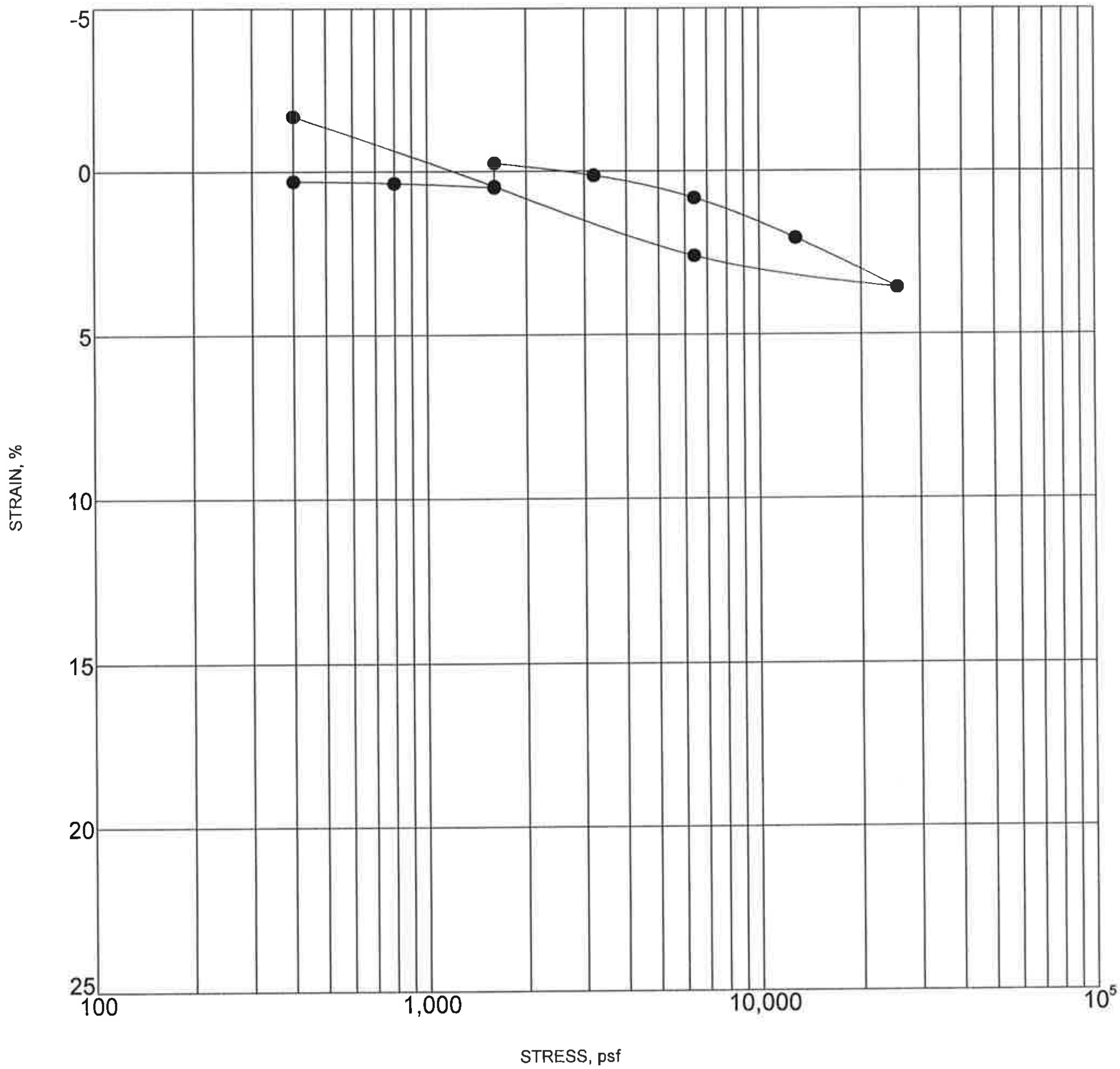
DIRECT SHEAR TEST RESULTS

FIGURE C-4



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-1	6.0	LEAN CLAY (CL)	104	20.2

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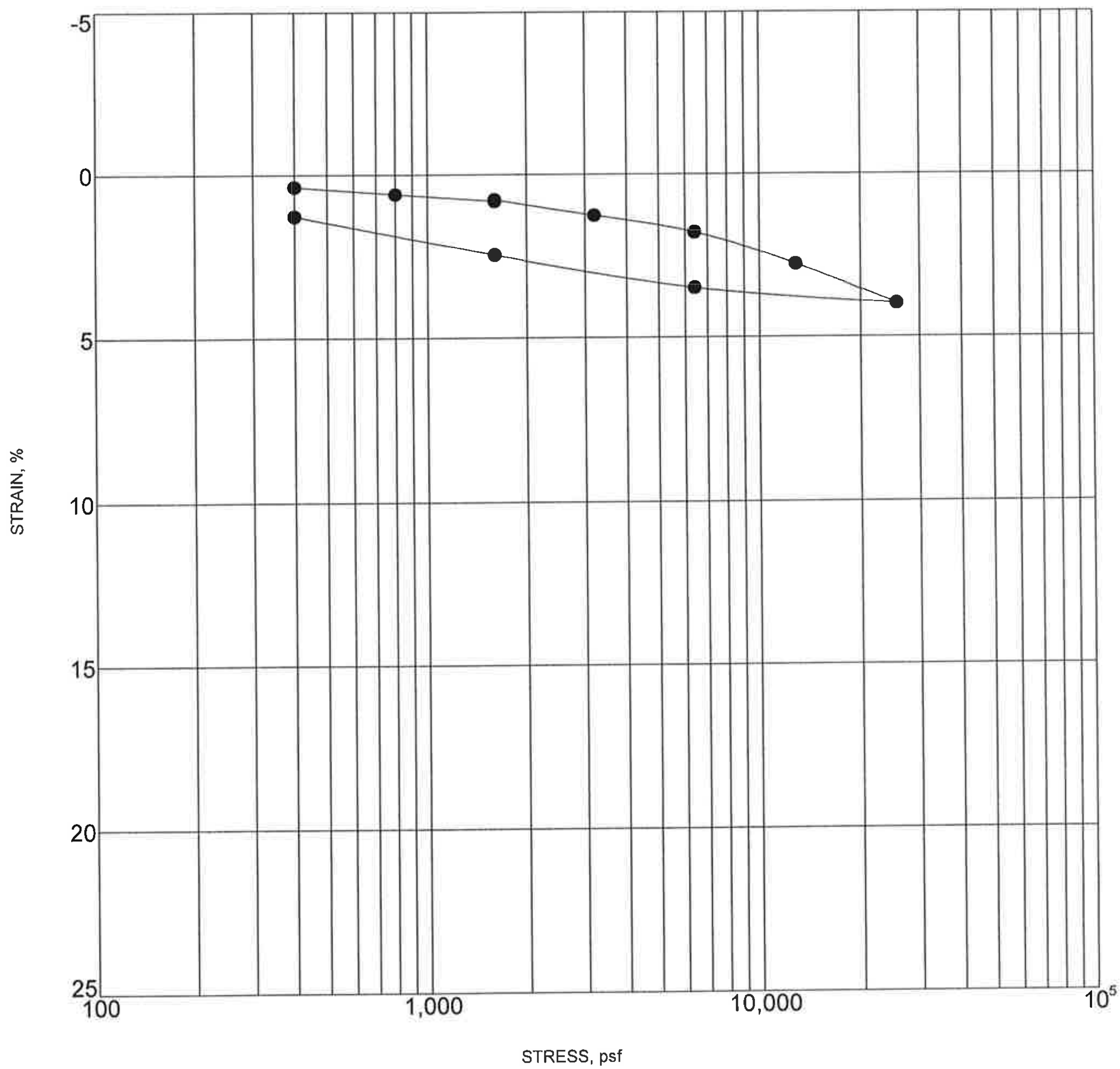
CONSOLIDATION TEST RESULTS

FIGURE C-5



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-1	12.0	LEAN CLAY (CL)	102	11.9

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PROJECT NO.: 2633.I

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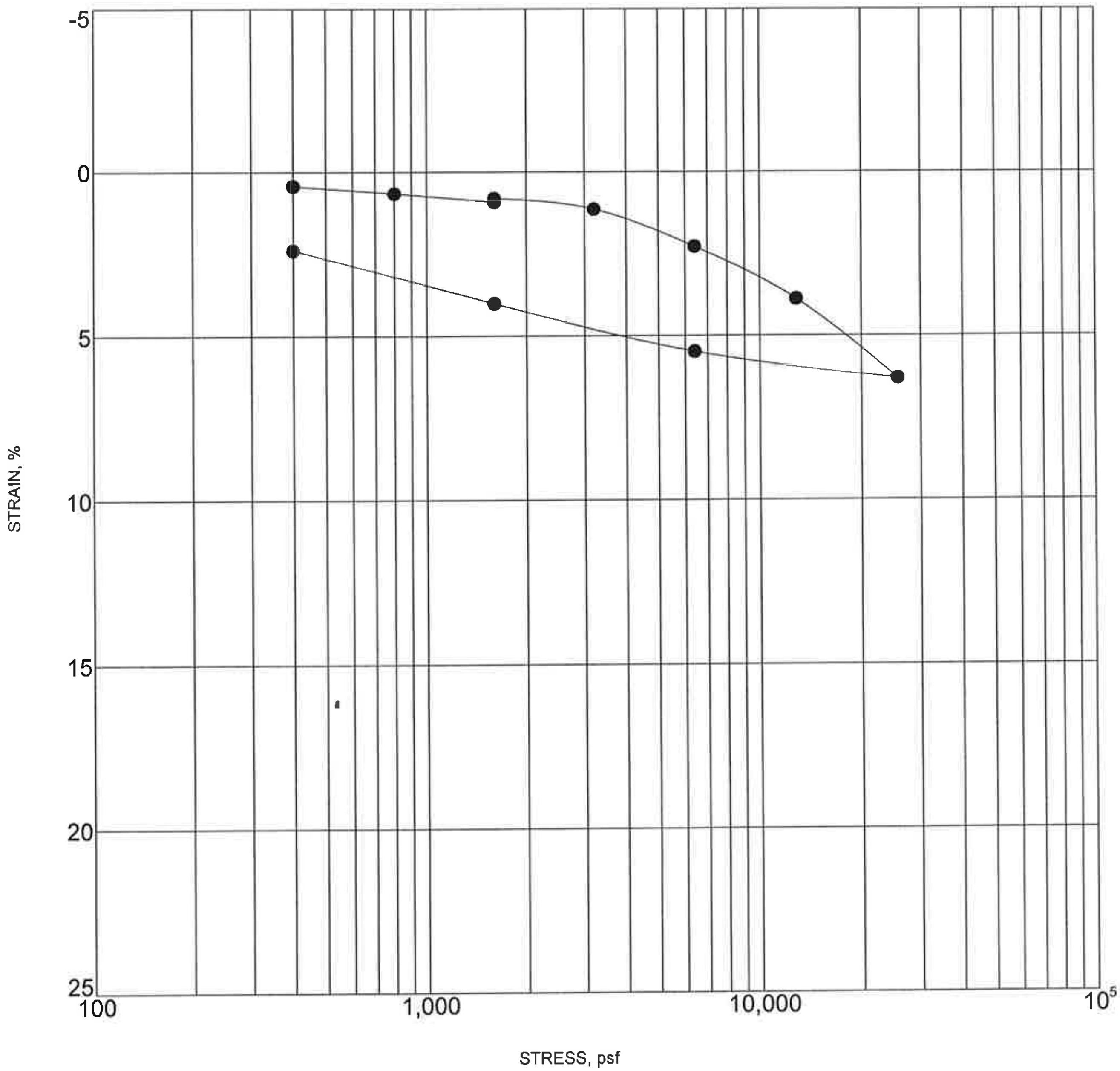
CONSOLIDATION TEST RESULTS

FIGURE C-6



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-2	15.0	SANDY CLAY (CL)	103	21.6

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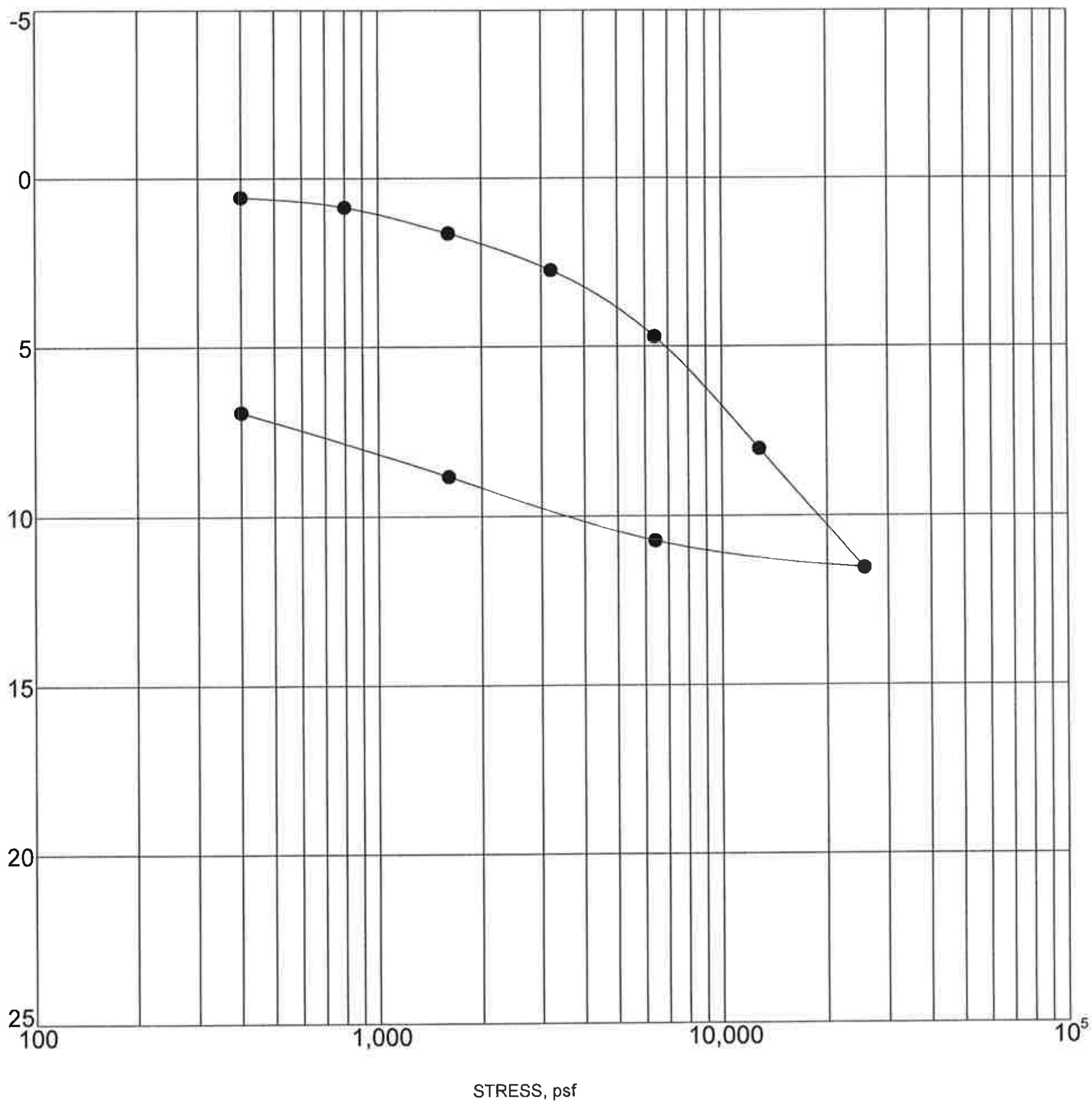
CONSOLIDATION TEST RESULTS

FIGURE C-7



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-3	3.0	LEAN CLAY (CL)	103	22.2

PROJECT: THE AVALON - CARSON

PROJECT NO.: 2633.I

CONSOLIDATION TEST RESULTS

FIGURE C-8

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
The Avalon
Your #2633.I, HDR Lab #14-0765LAB
7-Oct-14

Sample ID			B-1 @ 0-3'	B-3 @ 6-11'
Resistivity		**Units**		
as-received		ohm-cm	12,000	1,880
saturated		ohm-cm	332	384
pH			7.1	7.2
Electrical Conductivity		mS/cm	2.41	1.67
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	792	375
magnesium	Mg^{2+}	mg/kg	349	176
sodium	Na^{1+}	mg/kg	1,633	1,215
potassium	K^{1+}	mg/kg	21	17
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	125	171
fluoride	F^{1-}	mg/kg	11	16
chloride	Cl^{1-}	mg/kg	636	279
sulfate	SO_4^{2-}	mg/kg	5,105	2,444
phosphate	PO_4^{3-}	mg/kg	ND	ND
Other Tests				
ammonium	NH_4^{1+}	mg/kg	ND	ND
nitrate	NO_3^{1-}	mg/kg	63	24
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

431 West Baseline Road · Claremont, CA 91711
Phone: 909.962.5485 · Fax: 909.626.3316