GEOTECHNICAL
PROFESSIONALS INC

GEOTECHNICAL INVESTIGATION
PROPOSED CAMPUS MODIFICATIONS
McKINLEY ELEMENTARY SCHOOL
7812 McKINLEY AVENUE
LOS ANGELES, CALIFORNIA

Prepared for:
Los Angeles Unified School District
Design and A/E Technical Support
333 S. Beaudry Avenue, 22nd Floor, Room 217
Los Angeles, California 90017

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

GPI Project No. 2677,181

May 24, 2017

May 24, 2017

Los Angeles Unified School District
Design & A/E Technical Support
333 S. Beaudry Avenue, 22nd Floor, Room 217
Los Angeles, California 90017

Attention: Mr. Peyman Soroosh Moghadam, S.E.
Supervision Structural Engineer

Subject: Geotechnical Investigation
Proposed Campus Modifications
McKinley Elementary School
7812 McKinley Avenue
Los Angeles, California
GPI Project No. 2677.181

Dear Mr. Moghadam:

Transmitted herewith are four copies of our geotechnical investigation report for the proposed campus modifications at McKinley Elementary School.

We appreciate the opportunity of offering our services to your organization and look forward to seeing the project through its successful completion. Please do not hesitate to call us if you have any questions on the contents of our report or need further geotechnical assistance.

Very truly yours,
Geotechnical Professionals Inc.

Paul R. Schade, G.E.
Principal

PS:sph

Distribution: (4) Addressee (3 bound and 1 unbound plus flash drive)
(1) Ms. Cristina Cho, Los Angeles Unified School District (email)

2677-17I-01L (5/17)

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1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a preliminary geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for the proposed campus modifications at McKinley Avenue Elementary School in Los Angeles, California. The site location is shown on the Site Location Map, Figure 1.

A detailed geologic-seismic evaluation was performed for the project, including site-specific response spectra, as required by the 2016 California Building Code (CBC).

The project is at an early stage at this time, and specific details on the extent of the modifications and location of the new improvements are limited. A comprehensive investigation, utilizing the data obtained in this preliminary investigation, will be required prior to the final design to satisfy the regulatory agency requirements when further details of the project are available. Additional explorations and testing may also be required as part of the comprehensive investigation.

1.2 PROJECT DESCRIPTION

The project covered by this report includes modifications and modernization of the existing elementary school campus. We understand that the modifications may include new buildings as well as modernization of the existing buildings. The project also includes an evaluation of the non-wood-framed structures as outlined in AB300. The project is at an early stage at this time, and specific details on the extent of the modifications and locations of the new improvements are limited. The locations of the existing structures are shown on the Site Plan, Figure 2.

Detailed information regarding structural loads or site topography was not available at the time this report was prepared. We have assumed that the structural loads for the new buildings will be less than 150 kips for columns, and 2 to 3 kips per lineal foot for walls. We understand that the proposed buildings will predominantly be supported at or near the existing grade, but that one level subterranean construction (i.e. below-grade parking) may be considered. Modernization of the existing buildings may include additional loads being imposed to the existing foundations or new columns or foundations being added. Proposed grades are not anticipated to change significantly from the existing grades.

Our recommendations are based upon the above-assumed structural and finish grade information. We should be notified if the actual loads and/or grades differ or change during the project design to allow our office to either confirm or modify our recommendations. Also, when the project grading plan becomes available, we should be provided with a copy for review and comment.

1.3 PURPOSE OF INVESTIGATION

The primary purpose of this investigation and report is to provide an evaluation of the existing geotechnical and geologic conditions at the site as they relate to the design and construction of the proposed development.

2.0 SCOPE OF WORK

Our scope of work included a field investigation, laboratory testing, geologic and seismic evaluation, foundation analyses, and preparation of this report.

Our field investigation consisted of five Cone Penetration Tests (CPT's) and four exploratory borings. The CPT's were performed to depths of 40 to 60 feet below existing grades. The borings were performed to depths of 20 to 60 feet below the existing grade. A description of field procedures and logs of the CPT's and explorations are presented in Appendices A and B, respectively.

Our laboratory testing program included evaluations of in-place moisture content, Atterberg Limits, fines content, direct shear, consolidation, expansion index, maximum dry density and optimum moisture content, and 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.

An evaluation of geologic and seismic hazards is presented in Appendix D.

Engineering evaluations were performed to provide earthwork criteria and foundation design parameters. The results of our evaluations are presented in the remainder of the report.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The school site is bounded by East 78th Street to the north, McKinley Avenue to the west, East 79th Street to the south, and Wadsworth Avenue to the east. The site is approximately 4.85 acres in plan area, with various school buildings in the western half of the campus, and parking areas, blacktop playground, and smaller buildings in the eastern half of the campus. The topography across the site is relatively flat, with ground surface elevations ranging from approximately 139 feet (in the southeast) to 141 feet (in the northwest). The pavement sections at our exploration locations in pavement areas consisted of 4 inches of asphalt concrete without an underlying aggregate base course in Boring 8-1 and 3.5 inches of asphalt concrete over 3.5 inches of aggregate base in Boring 8-4.

We reviewed historical aerial photographs of the site dating back to 1952. In 1952, the school site appears to have been confined to the western half of the existing campus limits, with the eastern half appearing to be occupied by single-family residences. In 1963, the single family residences are no longer present and the bounds of the school site have been expanded to their approximate current limits, with new single story buildings appearing along the northern property line. Various buildings were added or removed from the site between 1963 and 2012, when the final currently existing building was visible in the aerial photographs. Since 2012, the site appears to have remained unchanged, apart from a few.

3.2 SUBSURFACE SOIL CONDITIONS

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

Undocumented fill soils to depths of 4 feet were encountered in the borings. The fill soils at the boring locations consisted of moist silty sands. The fill soils are likely undocumented and relatively old, given the age of the school. The upper fill soils exhibited a very low potential for expansion.

The underlying natural materials consisted predominantly of loose to medium dense silty sands and sands, with lesser deposits of firm to very stiff clays, silty clays, and sandy silts. Within the upper 12 feet, the natural soils consisted predominantly of loose to medium dense silty sands and sands. Below depths of 12 feet, the natural soils consisted of alternating layers of firm to very stiff fine-grained soils (clays, silty clays, and sandy silts) and medium dense coarse-grained soils (silty sands and sands). The natural soils become dense and very stiff to hard below depths of 32 to 34 feet. The natural soils are generally moist to wet, with higher moisture contents encountered within the fine-grained soils. Moisture contents in localized areas of the near surface soil were as high as 32 percent, roughly 22 percent above optimum moisture of 10.5 percent. The average moisture content of the soils within the upper 7 feet is approximately 13 percent. At shallow depths, the natural soils exhibited moderate strength and low compressibility characteristics.

The site is not located in a Methane Buffer Zone, as designated by the City of Los Angeles.

3.3 GROUNDWATER AND CAVING

Groundwater was not encountered within our explorations performed to depths of up to 60 feet below existing grade. Historical data provided by the California Geologic Survey (CGS) indicates a shallowest depth to groundwater of 15 feet in the vicinity of the site.

Caving was not encountered in our relatively small diameter borings.

3.4 GEOLOGIC - SEISMIC HAZARDS

A detailed evaluation of the geologic conditions at the site, including seismic hazards, is presented in Appendix D. Ground motion and seismic settlement is addressed in a following section of this report.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 OVERVIEW

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

- The site is located in a seismic hazard zone for soil liquefaction. Based on our analyses, we computed a potential total seismic-induced liquefaction settlement of 1 to 1¼ inches. Differential seismic settlement is estimated to be between ½- and ¾-inch across a span of 40 feet. These estimates are based on a published historical high groundwater level of 15 feet below the existing grade. Groundwater was not encountered within the 60-foot depth of our current explorations.

- The existing undocumented fill soils encountered to depths of 4 feet in our borings are not considered to be suitable for direct support of shallow foundations and floor slabs in their current state. To provide uniform support for the proposed improvements, we recommend that these materials be removed and replaced as properly compacted fill.

- The upper natural soils are loose to medium dense. To provide uniform support for the proposed shallow foundations and floor slabs, we recommend that the upper portion of the natural materials be removed and replaced as properly compacted fill.

- Support of the planned structures on isolated/continuous shallow footings or a mat foundation are feasible. The foundation system selected will depend on the tolerable total and differential settlements. Based on the structural loads assumed, the estimated combined static and seismic settlements slightly exceed the generally accepted limits (1 ½ inches of total settlement and ¾-inch differential settlement) for shallow spread footings. Additional settlement analyses should be performed when more detailed structural loads are available for the project.

- Conventional cast-in-place concrete piles may be used to support light standards and similar pole structures.

- Moisture contents of the near surface soils (within 7 feet of the existing grades) are moist to wet, averaging about 3 percent above the optimum moisture content, with localized samples up to 22 percent above optimum. Therefore, mixing and moisture conditioning will be required prior to being placed as properly compacted fill. In addition, over-optimum subgrade soils exposed during grading may require stabilization in order to support compaction equipment. Stabilization may be accomplished using crushed aggregate base and geogrid or in-place cement treatment.

- Corrosivity testing performed by HDR on samples provided from our borings indicates a negligible level of soluble sulfate content with respect to concrete. The soils are also considered to be moderately corrosive to ferrous metals.

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 DESIGN

4.2.1 General

Details of our geologic and seismic evaluation for the site are presented in Appendix D.

We assume the seismic design of the proposed development will be in accordance with the California Building Code (CBC), 2016 edition. For the 2016 CBC, a Soil 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 (geohazards.usgs.gov). We also present these values on Table 1, Site Specific Response Spectra Worksheet. The Project Structural Engineer should determine the seismic design method.

4.2.2 Site-Specific Ground Motion Analyses

Site-specific response spectra were generated in accordance with the 2016 California Building Code (CBC) (Section 1613A) and Section 21.2 of ASCE 7-10 (ASCE, 2010). Creation of a site-specific response spectrum requires analyzing site-specific deterministic and probabilistic seismic response spectra in order to create the Risk-Targeted Design and Maximum Considered Earthquake (MCE) response spectra.

Probabilistic and deterministic site response spectra were calculated using the computer program EZ-FRISK (Version 7.65, 2015). The program estimates uniform hazard spectra using faults as earthquake sources. The program database includes geographic and seismic information on known active faults in California from 2008. For both our deterministic and probabilistic analyses, we used NGA attenuation relationships for the maximum rotated component of ground motion as proposed by Boore and Atkinson (2008), Campbell and Bozorgnia (2008), and Chiou and Youngs (2007).

For our evaluations, we used a shear wave velocity, V_{s30}, of 295 meters per second, or about 968 feet per second, for attenuation relationships. This value corresponds to a CBC Site Class D (stiff soil) and was estimated from blow counts obtained during the investigation at the project site.

A site-specific probabilistic response spectrum was generated for the MCE per the requirements of ASCE 7-10. The MCE corresponds to an earthquake ground motion having a 2 percent probability of exceedance within a 50-year period, or an average return period of 2,475 years. The final probabilistic response spectrum was based on the maximum rotated component mean of the spectral response values at 5% damping for the three above noted attenuation relationships. The site-specific probabilistic response spectra, including the average probabilistic spectrum, is shown on Figure 3.

Site-specific deterministic MCE response spectra were generated per the requirements of ASCE Section 7-10. Response spectra were generated from known active faults within 100 kilometers of the subject site in order to determine the controlling spectral accelerations. Spectral acceleration ordinates were calculated as the 84th percentile of the maximum rotated component of the spectral acceleration at 5% damping (mean Sa + one standard deviation). The controlling deterministic response spectrum is based on the Puente Hills (LA) Fault. Site-specific deterministic response spectra from nearby faults, along with the required lower deterministic limit per Section 21.2.2 (Figure 21.2-1), are shown on Figure 4. The controlling upper bound site-specific deterministic response spectrum is shown on Figure 5.

The above-described analytical steps are presented in the attached Table 1, Site Specific Seismic Response Spectra Worksheet.

The site-specific MCE response spectrum was generated per the requirements of comparing the spectral response accelerations from the probabilistic MCE and the deterministic MCE, with the resulting MCE response spectrum being the lesser of the spectra accelerations at each period. The coordinates for the MCE response spectrum are presented in Table 1 (Column 9).

The site-specific design response spectrum was generated per the requirements of taking 2/3 of the MCE response spectrum, but confirming that the values are not less than 80 percent of the spectral acceleration determined per Section 11.4.5 of ASCE 7-10. The ordinates for the site-specific design spectrum are presented in Table 1 (Column 12).

We compared the spectral response accelerations from the probabilistic MCE of Section 21.2.1 (Figure 3) and the deterministic MCE of Section 21.2.2 (Figure 5), with the resulting MCE response spectra being the lesser of the spectral accelerations. The site-specific MCE and design response spectra are shown on Figure 6. The corresponding coordinates for the MCE and design response spectra are tabulated in Table 1.

4.2.3 Liquefaction, Lateral Spreading, and Seismic Settlement

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

The site is located in a Seismic Hazard Zone for liquefaction, as mapped by the State of California (Inglewood Quadrangle). Groundwater was not encountered in our explorations to depths of 60 feet below existing grade. Historical high groundwater levels provided by the California Geologic Survey indicate a shallowest groundwater table of approximately 15 feet below existing grades (CGS, 1999). As such, we assumed a groundwater depth of 15 feet in our liquefaction evaluation.

Revisions to the 2016 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 (PGAM) adjusted for site class effects or a site-specific response spectra. As detailed in the previous section, we developed site-specific response spectra per the requirements of Section 21.2 of ASCE 7-10 and the 2016 CBC. Per the requirements of Section 21.5.3, the site-specific peak ground acceleration shall not be taken as less than 80 percent of PGAM, which is defined as the product of the PGA for the mapped MCEG (Site Class B) and a site coefficient, FPGA. Based on this analysis, we considered a site-specific design peak ground acceleration of 0.57g for a magnitude 6.5 earthquake (Puente Hills - LA) for our analyses, which corresponds to the lesser of the probabilistic and deterministic spectral accelerations at a period of Oseconds obtained using the methods 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. Criterion for liquefaction susceptibility of the fine-grained soils was based on methods presented in Bray and Sancio (2006).

The materials encountered below the historical high groundwater level generally consisted of alternating layers of firm to very stiff fine-grained soils (silts and clays) and medium dense coarse-grained soils (sands and silty sands). Below depths of 32 feet, some of the sand and silty sand layers were dense. Overall, the soils encountered exhibited moderate strength.

Per the requirements of SP 117, liquefaction analyses are typically limited to depths of 50 feet below the structural foundation. Based on our analyses, and assuming a potential groundwater depth of 15 feet, we computed potential total seismic-induced liquefaction settlements of 1 to 1¼-inches. Differential seismic settlements (across a 40-foot span) are estimated to be between ½- and ¾-inch.

Seismic ground subsidence (not related to liquefaction induced settlements) occurs when strong earthquake shaking results in densification of loose to medium dense sandy soils above groundwater. Due to the shallow depths to groundwater used in our liquefaction analysis (15 feet) and the recommended depth of removal and recompaction, the potential for dry seismic to adversely affect the site is considered to be low. As such, we do not anticipate measurable seismic settlement of the soil above the groundwater.

4.3 EARTHWORK

The earthwork at the project site is anticipated to consist of clearing, subgrade preparation, and the placement and compaction of fill.

4.3.1 Clearing

Prior to grading, performing excavations, or constructing the proposed improvements, the areas to be developed should be cleared of debris and pavements. Buried obstructions, such as footings, abandoned utilities, and tree roots should be removed from areas to be developed. Deleterious material generated during the clearing operation, including organic topsoil or material within the existing undocumented fill, should be removed from the site. If approved by the District, inert demolition debris, such as concrete, asphalt, and brick may be crushed for reuse in engineered fills outside the planned building areas in accordance with the criteria presented in the "Materials for Fill" section of this report. It is our experience that such material will be required to be exported from the site.

If cesspools or septic systems are encountered during grading, they should be removed in their entirety. The resulting excavation should be backfilled as recommended in the "Subgrade Preparation" and "Placement and Compaction of Fill" sections of this report. As an alternative, cesspools can be backfilled with lean sand-cement slurry. At the conclusion of the clearing operations, a representative of the personnel from GPI should observe and accept the site prior to further grading.

4.3.2 Excavations

Excavations at this site will include removals of undocumented fill soils, soils disturbed during demolition and portions of the weak native soils, foundation excavations, and trenching for new utility lines.

Prior to placing fills or construction of the structures or pavement, the existing undocumented fill and loose soils disturbed during demolition, and portions of the upper soils should be removed and replaced as properly compacted fill. To provide uniform support for planned structures supported on shallow foundations, the footings and floor slabs should be underlain by properly compacted fill. For planning purposes, we anticipate average removal depths across the building pads of 7 feet below existing grades or 4 feet below the base of foundations, whichever is deeper, for planned at-grade buildings (e.g. classroom, administration buildings). Existing grades refer to the grades at our exploration locations.

For subterranean structures, removals should extend deep enough for the placement of at least 2 feet of properly compacted fill beneath the base of foundations.

For minor structures (e.g. site walls, trash enclosures), removals should extend 4 feet below grade or 2 feet below the base of foundations, whichever is deeper. Deeper removals may be required where deep undocumented fill soils are encountered. Removals are not required for pile supported minor structures such as light standards.

For new pavements and hardscape, removals should extend at least 1-foot below the existing or proposed subgrade, whichever is deeper.

For building retrofit where new foundations are required within the footprint of a building to remain, or where existing footings will be enlarged to carry additional loads, the extent of remedial grading will depend on the subsurface conditions encountered. For planning purposes, these foundations should be underlain by at least 1-foot of new properly compacted fill soils. Deeper removals may be required depending on the actual conditions exposed in the foundation excavations.

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

The Project Surveyor should accurately stake the corners of the areas to be overexcavated in the field. Where space is available, the base of the excavations should extend laterally at least 5 feet beyond the building line or edge of foundations, or a minimum lateral distance equal to the depth of overexcavation/compaction below finish grade (i.e., a 1:1 projection below the bottom outside edge of footings), whichever is greater. Building lines include the footprint of the building and other foundation supported improvements, such as canopies and attached site walls. For new footings inside an existing building related to retrofit, the limits of the removal can be limited to the lateral limits of the new foundation.

In general, the upper fill soils are considered moderately susceptible to caving in shallow excavations. Temporary construction excavations may be made vertically without shoring to a depth of 3 feet below the adjacent grade. For deeper cuts up to 10 feet, the slopes should be properly shored or sloped back to at least 1:1 or flatter. For cuts deeper than 10 feet but not exceeding 20 feet, slopes should be properly shored or sloped back to at least 1¼:1 (horizontal:vertical) or flatter. Some raveling of the sandy deposits should be anticipated at the slope inclinations recommended. If raveling cannot be tolerated, flatter slope inclinations should be considered. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing.

Excavations adjacent to existing foundations should not extend below an imaginary plane descending at an inclination of 1:1 from a point 1-foot above the base of an existing foundation unless slot cutting or shoring are used.

"ABC" slot cuts may be utilized in place of temporary shoring where removals adjacent to existing improvements or property lines are performed (e.g. retrofits to existing foundations. The slots should not exceed 8 feet in height and 8 feet in width and should be backfilled <u>immediately</u> to finished grade prior to excavation of the adjacent slots. If the slots are performed adjacent to an existing building that has perimeter pad footings in addition to a continuous footing, the slots should be aligned so that not more than one-half of the pad footing is exposed at a time. We should review the plans for excavation adjacent to existing buildings when they are developed.

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

In general, the excavation can be accomplished by conventional soil excavation equipment such as backhoes, loaders, scrapers, or dozers.

4.3.3 Subgrade Preparation

Prior to placing fills, the subgrade soils at the bottom of overexcavations 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 determined in accordance with ASTM D1557. This recommendation also pertains to the subgrade areas of asphalt pavement and hardscape.

During our investigation, moist to wet soils with moisture contents of up to 32 percent (roughly 22 percent above optimum) were encountered within the upper 7 feet. The earthwork subcontractors should review the moisture content information presented on the boring logs, as wet soils may be encountered that will require mixing, drying, or stabilization prior to compaction. Also, heavy rubber-tired equipment is likely to cause pumping or yielding of wet subgrade. We do not recommend that the earthwork be performed in wet-weather seasons.

If wet soils are encountered or if the exposed soils become wet from seasonal rains, subgrade stabilization may be required to support compaction equipment. For planning purposes, the stabilization would require the placement of 12 inches of crushed aggregate base (CAB) over a geogrid, such as Tensar BX1100. A thicker section of CAB could be used if the geogrid is omitted. As an alternative, the wet soils can be cement treated. For planning purposes, we anticipate stabilization can be achieved by mixing 4 percent cement within the upper 15 inches of exposed soil by unit weight (assume 120 pcf). The cement treatment should be performed by a subcontractor experienced with the process, using equipment that can thoroughly mix the soil-cement prior to compaction.

4.3.4 Material for Fill

Soils available from on-site excavations, less debris or organic matter, will be suitable for re-use in compacted fills. Soils placed behind retaining walls and within 1-foot of the finished subgrade for building floor slabs and hardscape should be predominately granular (containing no more than 40 percent fines - portion passing No. 200 sieve) and non-expansive (E.I. of 20 or less). Such materials are anticipated to be available in sufficient quantities within the upper 7 feet below existing grades.

Imported fill material should be predominately granular and non-expansive as defined above. Import or on-site materials used in compacted fills should not contain particles larger than 3 inches in diameter. GPI should be provided with a sample (at least 50 pounds) and notified of the location of soils proposed for import at least 72 hours in advance of importing. Each proposed import source should be sampled, tested and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by GPI may be rejected if not suitable.

If open-graded gravel is used as backfill, such as for stormwater infiltration or retention systems, the material should be separated from the adjacent soils with a suitable non-woven filter fabric, such as Mirafi 140N.

4.3.5 Placement and Compaction of Fills

Fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to densities equal to at least 90 percent of the maximum dry density, determined in accordance with ASTM D1557. 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±) or track equipment	6-8 inches
Scrapers, heavy loaders, or heavy 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.

Fills should be placed at moisture contents of 0 to 2 percent over the optimum moisture content for granular soils and silts, and 1 to 3 percent over optimum for clays. The moisture content of the soils encountered in the upper 7 feet in the explorations was, on average, roughly 3 percent above optimum. As such, adequate mixing and some moisture conditioning (drying) may be necessary prior to replacing the soils as properly compacted fill. The on-site soils should not be allowed to dry out prior to covering or additional moisture conditioning and processing will be required. The moisture content of the subgrade soils should be confirmed by GPI prior to covering.

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

4.3.6 Shrinkage and Subsidence

Shrinkage is the loss of soil volume caused by compaction of fills to a higher density than before grading. Subsidence is the settlement of in-place subgrade soils caused by loads generated by large earthmoving equipment. For earthwork volume estimating purposes, an average shrinkage value of 20 to 25 percent may be assumed for the surficial soils. Higher values may be realized if deep undocumented fills are encountered. Subsidence is anticipated to be about 0.1 feet. These values are estimates only and exclude losses due to removal of vegetation or debris. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be verified during grading.

4.3.7 Trench/Wall Backfill

Utility trench and wall backfill, consisting of the on-site materials or imported sand, should be mechanically compacted in lifts. Clays and silts should not be used for retaining wall or wall-below-grade backfill. Lift thickness should not exceed those values given in the "Placement and Compaction of Fill" section of this report. Moisture conditioning of the on-site soils will be required prior to re-use as backfill. Jetting or flooding of backfill materials should not be permitted. A representative of GPI should observe and test trench and wall backfills as they are placed.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, sand-cement slurry may be substituted for compacted backfill. The slurry should contain one sack of cement per cubic yard. Within building areas, the slurry should contain two sacks of cement per cubic yard. When set, such a mix typically has the consistency of compacted soil. Under foundations, concrete equal in strength to the foundation concrete should be used if fill is required.

4.3.8 Observation and Testing

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

4.4 FOUNDATIONS

4.4.1 General

The proposed structures and modifications to existing buildings may be supported on conventional isolated and/or continuous shallow footings or a mat foundation, provided the subsurface soils are prepared in accordance with the recommendations given in this report. The decision to support structure on a mat foundation instead of spread footings will depend on the allowable total and differential static and seismic settlements. Shallow foundations should be supported on properly compacted fill. We are also providing recommendations for design of deep foundations for support of both light standards and similar type structures.

4.4.2 Bearing Capacity

Spread Footings

Based on the shear strength and elastic settlement characteristics of the recompacted on-site soils, a static allowable net bearing pressure of up to 3,500 pounds per square foot (psf) may be used for both continuous footings and isolated column footings. The actual bearing pressure used may be less, such that economics and structural loads will determine the minimum width for footings as discussed below. These bearing pressures are for dead load-plus-live loads, and may be increased 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.

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)
3,500	24	24
3,000	18	24
2,500	18	18
1,500	15	15

* Depth to bottom of footing below lowest adjacent finish grade.

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

Mat Foundation

The allowable bearing pressure for a mat foundation is generally not the governing geotechnical issue as compared to the anticipated settlement. At this time, we have not been provided with estimated static mat foundation pressures for the proposed structures. If a mat foundation is to be considered, we should be provided with a detailed plot of the anticipated bearing pressures to review.

For the elastic design of the mat foundation, a modulus of subgrade reaction (k-value) of 175 pounds per cubic inch (pounds per square inch per inch of deflection) may be used. This value is for a 1-foot by 1-foot square loaded area and should be adjusted for the area of the mat foundation using appropriate elastic theory. Using generally accepted methods and our site specific consolidation test results, we recommend using a value of 40 pci for the adjusted k-value in designing the mat foundation. As previously discussed, we should be provided with the anticipated mat pressures when they are developed so that we can review and confirm the recommendations provided, as well as provide an estimate for the anticipated maximum static settlements for the mat foundations.

The allowable soil bearing pressure will be significantly greater than the average bearing pressures required for the mat foundation as discussed above. At localized areas of the mat, such as columns and point of load applications along exterior walls, a static allowable net bearing pressure of 2,000 pounds per square foot may be used. These allowable bearing pressures are for dead-plus-live loads, and may be increased one-third for short-term, transient, wind and seismic loading.

4.4.3 Settlement

Under the static load conditions assumed (column loads of up to 150 kips and wall loads of up to 3 kips per lineal foot), maximum total static settlement of the proposed structures is expected to be on the order of ½- to ¾-inch. Maximum differential static settlement between similarly loaded adjacent footings is estimated to be on the order of ¼-inch across a lateral distance of 40 feet.

As discussed earlier, we computed total seismic settlements of 1 to 1¼ inches for the purpose of evaluating total foundation settlement. As such, total combined static plus seismic settlement for the purposed of determining foundation feasibility is expected to be between 1½ and 2 inches. Combined differential static plus seismic settlement is expected to be between ¾- and 1-inch across a lateral distance of 40 feet. The combined settlements (total or differential) slightly exceed the generally accepted limits for spread and/or continuous footing foundations (1 ½ inches of total settlement and ¾-inch of differential settlement). A mat foundation should be used for support of structures if the estimated settlements are not tolerable for spread footings. When detailed structural loads are available, we should be provided with the information to further evaluate the settlements.

The above settlement 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.

4.4.4 Lateral Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of footings and underlying soils and by passive soil pressures acting against the embedded sides of the footings. For frictional resistance, a coefficient of friction of 0.35 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 275 pounds per cubic foot may be used, provided the footings are poured tight against the compacted fill. A one-third increase in the above allowable lateral bearing pressure (but not the frictional resistance) may be taken for short-term wind and seismic loads. The passive pressure provided also assumes a level ground surface extending to a horizontal distance from the wall or footing face at least twice the depth of embedment. These values may be used in combination without reduction.

4.4.5 Light Standards and Poles

Light standards and similar structures may be supported on drilled pier foundations. The design of such piers is typically governed by lateral loading conditions. Soil resistance to lateral loads can be provided by the piles. The design of the piles will be governed by lateral force considerations. For design by the simplified pole formula presented in Section 1807A.3.2.1 of the 2016 California Building Code, a unit passive resistance of 275 pounds per square foot per foot (to a maximum of 2,750 pounds per square foot) may be used for the piles with level ground in lieu of the presumptive lateral bearing values presented in Table 1806A.2. As stated in the code, a passive resistance of 550 pounds per square foot per foot (to a maximum of 5,500 pounds per square foot) may be used for isolated piles as determined by the Project Structural Engineer. This value incorporates the allowable increase stated in the Section 1806A.3.4 of the code for single poles that can tolerate a ½-inch of deflection under short-term loads. We recommend that the upper 1-foot of the subgrade soils be ignored in determining the required depth of embedment to allow for surface disturbance adjacent to the pile.

A pile designed for adequate embedment to resist the anticipated lateral loads should have adequate axial capacity to support the anticipated vertical loads. The net allowable vertical compressive capacity can be conservatively calculated based on a unit side friction of 325 pounds per square foot, neglecting end bearing contribution. We recommend that the upper 1-foot of the subgrade soils be ignored in determining the required depth of embedment to allow for future surface disturbance adjacent to the pile.

4.4.6 Foundation Concrete

Laboratory testing by HOR (Appendix B) indicates that the on-site soils have a soluble sulfate content of 14 mg/kg (0.0014 percent by weight). In accordance with the 2016 CBC, foundation concrete should conform to the requirements outlined to the requirements outlined in ACI 318, Section 4.3 for a negligible level of soluble sulfate exposure for soil (ACI Category 'SO'). Chloride was not detected.

4.4.7 Foundation Observation

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

4.5 CONCRETE SLABS

A moisture vapor retarder should be placed under slabs that are to be covered with moisture-sensitive floor coverings (parquet, vinyl, tile, etc.). Currently, common practice is to use a 10 or 15 mil polyolefin product such as Stego Wrap for this purpose. Whether to place the concrete slab directly on the vapor barrier or place a clean sand layer between the slab and vapor barrier is a decision for the Project Architect and General Contractor, as it is not a geotechnical issue. If covered by sand, the sand layer should be about 2 inches thick and contain less than 5 percent by weight passing the No. 200 sieve. Based on our explorations and laboratory testing, the soils at the site are not suitable for this purpose. The function of the sand layer is to protect the vapor retarder during construction and to aid in the uniform curing of the concrete. This layer should be nominally compacted using light equipment. The sand placed over the vapor retarder should only be slightly moist. If the sand gets wet (for example as a result of rainfall or excessive moistening) it must be allowed to dry prior to placing concrete. Care should be taken to avoid infiltration of water into the sand layer after placement of the concrete slab, such as at slab cut-outs and other exposures. A sand layer is not required beneath the vapor retarder, but we take no exception if one is provided.

It should be noted that the material used as a vapor retarder is only one of several factors affecting the prevention of moisture accumulation under floor coverings. Other factors include maintaining a low water-cement ratio for the concrete used for the floor slab and 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.

For the elastic design of slabs supporting sustained concentrated loads, a modulus of subgrade reaction (k) of 175 pounds per cubic inch (pounds per square inch per inch of deflection) may be used. This value is for a 1-foot by 1-foot square loaded area and should be adjusted by the structural designer for the area of the proposed building slab using appropriate elastic theory.

Concrete hardscape should be supported on non-expansive, compacted soils as discussed in the "Placement and Compaction of Fill" section. Suitable soils, such as the onsite sandy silts and silty sands, are anticipated to be readily available within the upper 7 feet below existing grades. Clays are not suitable for direct support of slabs and hardscape. The subgrade soils should not be allowed to dry out prior to concrete placement or additional processing and moisture conditioning will be required.

4.6 PAVED AREAS

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

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		ASPHALT/PORTLAND CONCRETE	AGGREGATE BASE COURSE
Asphalt Concrete			
Playground (no vehicles)	--	2.0	3.0
Automobile Parking	4.0	3.0	5.0
Automobile Drives	5.0	3.0	7.0
Truck/Bus Drives	6.0	3.5	9.0
Portland Cement Concrete			
Automobile Parking	4.0	6.0	4.0
Automobile Drives	5.0	6.5	4.0
Truck/Bus Drives	6.0	7.0	4.0

The portland cement concrete used for paving should have a modulus of rupture of at least 550 psi (equivalent to an approximate compressive strength of 3,700 psi at the time the pavement is subjected to traffic). If the site is base paved prior to the start of building construction, the above pavement sections should be re-evaluated based on the anticipated construction traffic loads.

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 200-2 of the Standard Specifications for Public Works Construction (Green Book) for crushed aggregate base (CAB) materials.

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.

4.7 RETAINING STRUCTURES AND SHORING

Based on information available to us at the time this report was prepared, significant retaining walls are not planned but relatively tall walls may be required for subterranean parking levels, if constructed. The following recommendations are provided for cantilevered site walls or subterranean building walls up to 15 feet in height. We recommend that walls be properly drained and backfilled with sandy soils (less than 40 percent passing the No. 200 sieve). The onsite clays and silts are not suitable for use as retaining wall backfill where conventional backfill is used.

Although data provided by CGS indicates an approximate historical high groundwater level of 15 feet below existing grades, we did not encounter groundwater within the 60-foot depth explored. Based on current groundwater management practices, the potential for groundwater to negatively impact the proposed development is considered to be negligible.

4.7.1 Basement and Retaining Walls

Active earth pressures can be used for designing walls that can yield at least ½-inch laterally per 10 feet of wall height under the imposed loads. For level, drained backfill, derived from non-expansive granular soils (EI 20), a lateral pressure of an equivalent fluid weighing of 38 pounds per cubic foot may be used. At-rest pressures should be used for restrained walls that remain rigid enough to be essentially non-yielding. For select, non-expansive, level, drained backfill, a lateral pressure of an equivalent fluid weighing 54 pounds per cubic foot can be used. If the wall backfill is not drained, the combined earth and water pressures could be much higher.

A seismic lateral pressure should be used for the design of retaining walls as required. We recommend a seismic lateral pressure of 20 pounds per cubic foot be added to the active earth pressure recommended above. If at-rest pressure is used to design the retaining wall, the total lateral pressure used (at-rest plus seismic) is not required to exceed the total active plus seismic pressure (58 pounds per cubic foot).

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 recommended pressures assume that the supported earth will be fully drained, preventing the build-up of hydro-static pressures. For traditional backfilled retaining walls, a drain consisting of perforated pipe and gravel, wrapped in a suitable filter fabric should be used. As a minimum, one cubic foot of rock should be used for each lineal foot of drain. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top. For basement walls cast directly against temporary shoring, we recommend a drainage board be placed between the wall and shoring that extends from about 3 feet below finished grade down to the base of the wall. The drainage board should be connected to a suitable collection device and discharged to a sump.

4.7.2 Temporary Shoring

Where there is not sufficient space for sloped embankments, such as along the property limits or adjacent to existing structures, shoring will be required. One method of shoring would consist of steel soldier piles placed in drilled holes and backfilled with concrete. Driven or vibrated soldier piles may also be more economical alternative to drilled holes, and they can be used for supporting cuts that do not support existing structures.

For cantilever shoring with level backfill, the magnitude of active pressure is equivalent to the pressures imposed by a fluid weighing 38 pounds per cubic foot (pcf). For sloping backfill with a 1: 1 inclination, the active pressure would be 65 pcf.

In addition to the recommended earth pressure, the shoring should be designed for surcharge loads due the adjacent structures and construction traffic surcharge loads. The upper 10 feet of the shoring adjacent to streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge behind the shoring due to normal street traffic. If traffic is kept at least 10 feet from the shoring, the traffic surcharge may be neglected. Existing adjacent structures will impart a surcharge load on shoring. The location and depth of the adjacent building footings, as well as the loading, will need to be determined to estimate the surcharge pressure on the shoring.

For design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the soils below the excavation may be taken to be 550 pounds per square foot at the excavated surface, up to a maximum of 5,500 psf. To develop the full lateral value, provisions should be made to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavation below the excavated level may be a lean mix, but it should be of adequate strength to transfer the imposed loads to the surrounding soils.

The shoring contractor should evaluate the potential drilling conditions when planning the installation methods.

Driven or vibrated soldier piles may be a feasible and more economical alternative. If soldier piles are vibrated or driven, predrilling should not be allowed below the planned excavation level. Predrilling should be performed with a continuous flight auger capable of reversing the auger to minimize the removal of soil during the process. The diameter used for predrilling should not exceed 80 percent of the maximum depth of the soldier pile section. For design, the width of the driven or vibrated pile should be taken as the width of the flange.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of the anchor load, if used. The coefficient of friction between the soldier pile and the retained earth may be taken as 0.35. This value is based on the assumption that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean mix concrete and the retained earth. In addition, provided the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads. The frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken as equal to 400 pounds per square foot.

Continuous lagging will be required between the soldier piles. Careful installation of the lagging will be necessary to achieve bearing against the retained earth. We recommend that the voids between the lagging and retained earth be backfilled with a lean-mix sand-cement slurry prior to continuing the excavation deeper. The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less because of arching of the soils between piles. We recommend that the lagging be designed for the recommended earth pressure but limited to a maximum value of 400 pounds per square foot, provided the soldier beam spacing is 8 feet or less.

It is difficult to accurately predict the amount of deflection of the shored embankment. It should be realized, however, that some deflection will occur. Adjacent to city right-of-way, the shoring should be designed to limited deflection to 1-inch. We recommend limiting the lateral deflection of shoring adjacent to structures to ½-inch. If greater deflection occurs during construction, additional bracing may be necessary. In areas where less deflection is desired, such as adjacent to existing settlement sensitive improvements, the shoring should be designed for higher lateral earth pressures.

We recommend performing a detailed survey of the improvements to be supported above the planned shoring prior to and during the shoring installation. The survey should include topographic data and a video account of the condition of the existing improvements, including cracks or signs of distress. During construction, the monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of the soldier piles. We suggest weekly readings during the excavation and for the first three weeks after achieving the bottom of the excavation. After that time, the readings should be performed every other week until the completion of the basement walls.

4.8 CORROSION

Resistivity testing indicated that the on-site soils are moderately corrosive to ferrous metals. GPI does not practice corrosion engineering. We recommend that a corrosion engineering firm, such as HOR, be consulted if corrosion protection recommendations are required.

4.9 SURFACE DRAINAGE AND INFILTRATION

Positive surface gradients should be provided adjacent to structures so as to direct surface water run-off and roof drainage away from foundations and slabs toward suitable discharge facilities. Long-term ponding of surface water should not be allowed on pavements or adjacent to buildings.

Field infiltration testing was not included in our scope. The potential for water to infiltrate into a soil is based on the gradation and in-place density of a soil. Based on the subsurface conditions encountered, sandy soils were present within the upper 12 feet in our explorations. These soils may be suitable for infiltration, although the infiltration rates may be limited because of the presence of stiff silts and clays underlying the sandier materials. We recommend subsurface infiltration options be located a lateral distance of at least 30 feet from existing or proposed structures. Increased lateral offsets should be used for retaining walls or planned subterranean structures.

4.10 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

This report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for use by the Los Angeles Unified School District 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 project other than the currently proposed development as it may not contain sufficient or appropriate information for such uses.

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 by GPI during grading, excavation, and foundation construction. If construction phase services are performed by others they must accept full responsibility for 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, R.C.E.
Project Engineer

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

Paul R. Schade, G.E.
Principal

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

Thomas G. Hill, C.E.G. 1100
Consulting Geologist

REGISTERED GEOLOGIST
THOMAS G. HILL
No. 1100
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA
Exp 9.30.18

DJB/PRS:sph

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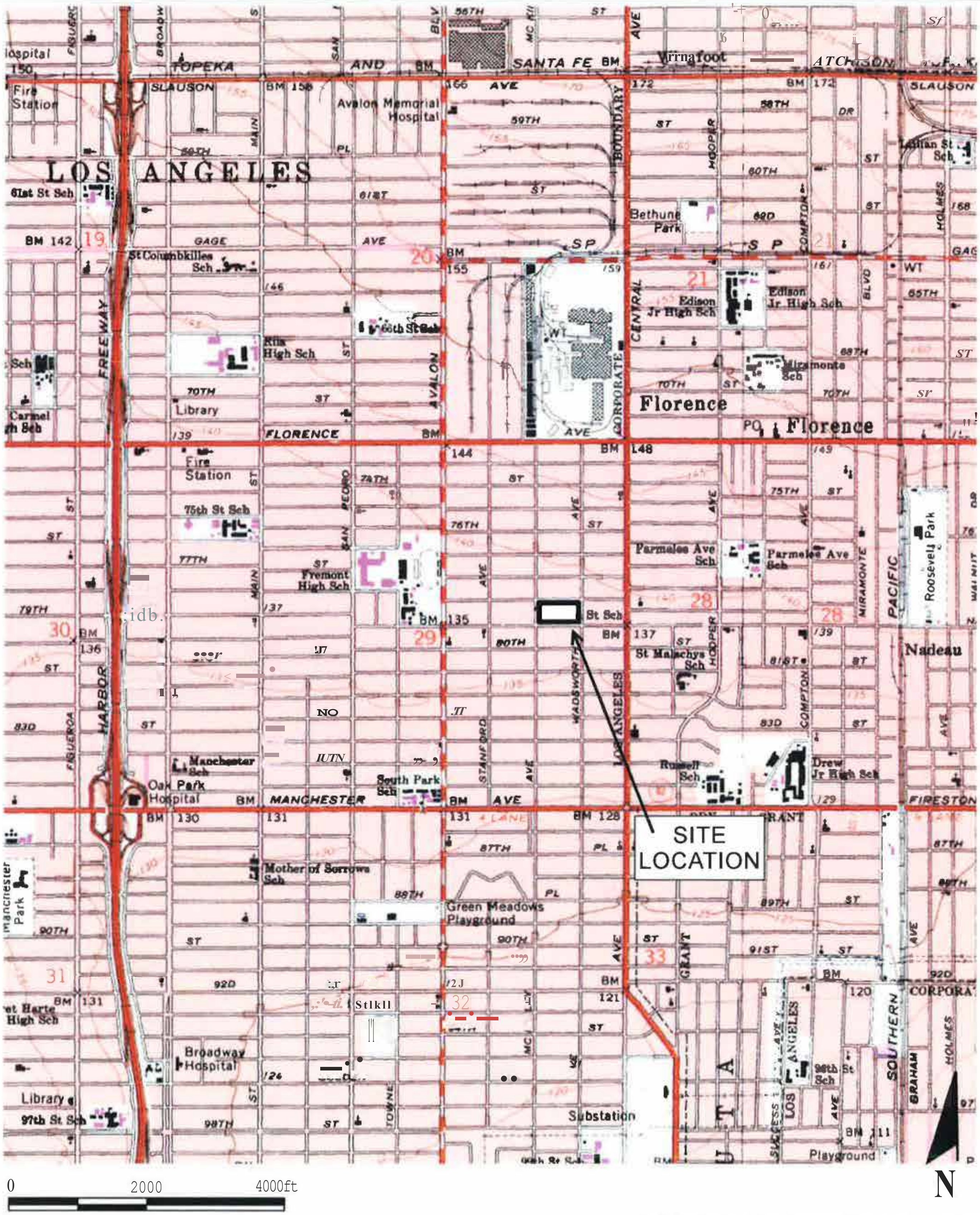


BASE MAP REPRODUCED FROM© CalTopo, USGS

GEOTECHNICAL PROFESSIONALS, INC.

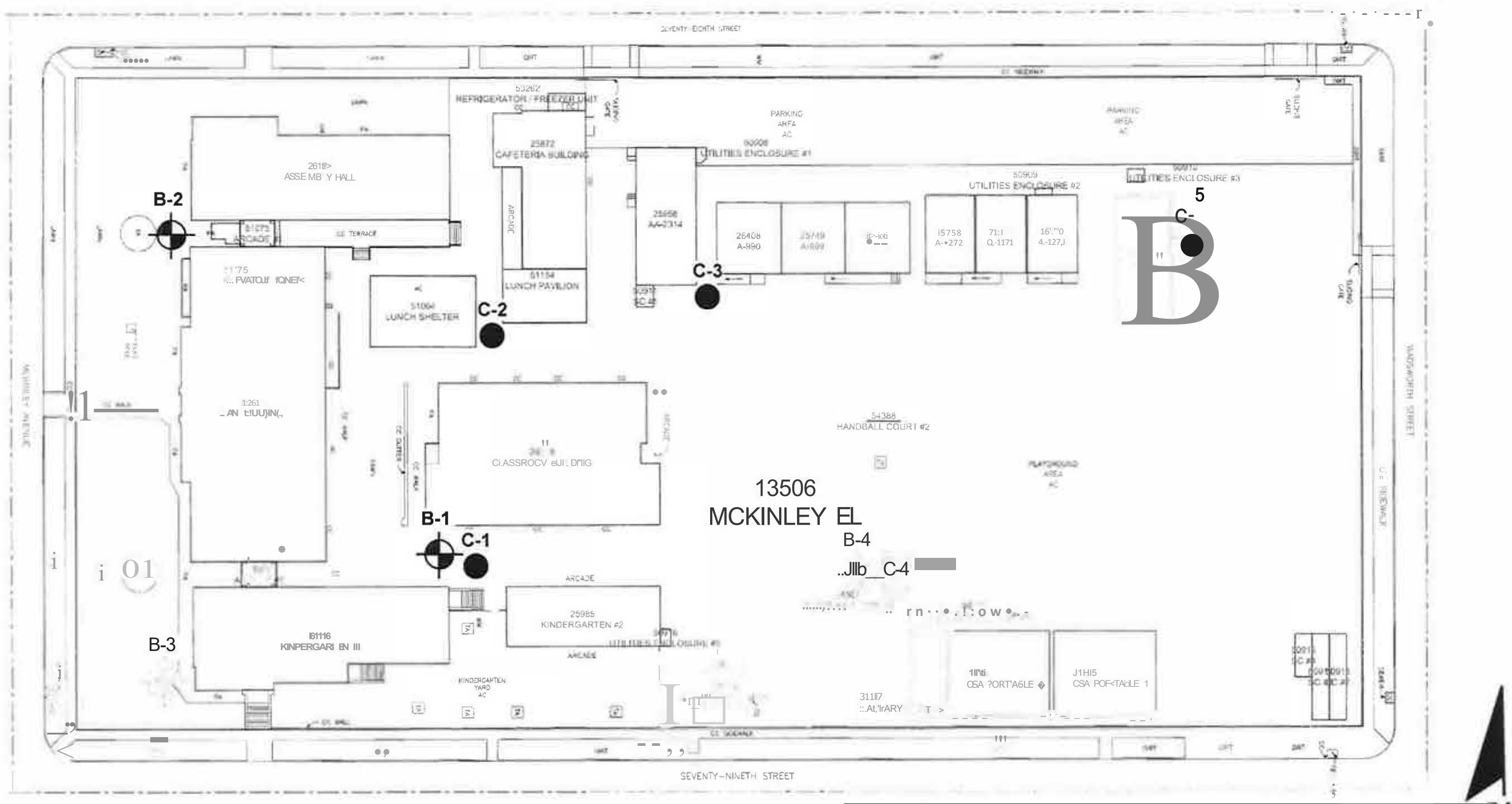
MCKINLEY AVENUE ELEMENTARY SCHOOL

GPI PROJECT NO. 2677.181

SCALE: 1" = 2000'

SITE LOCATION MAP

FIGURE 1



EXPLANATION

B-4

APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST

0 80 160 FEET

BASE MAP REPRODUCED FROM PLOT PLAN PRODUCED BY LAUSD ASSET MANAGEMENT BRANCH, DATED (JANUARY 1, 2017)

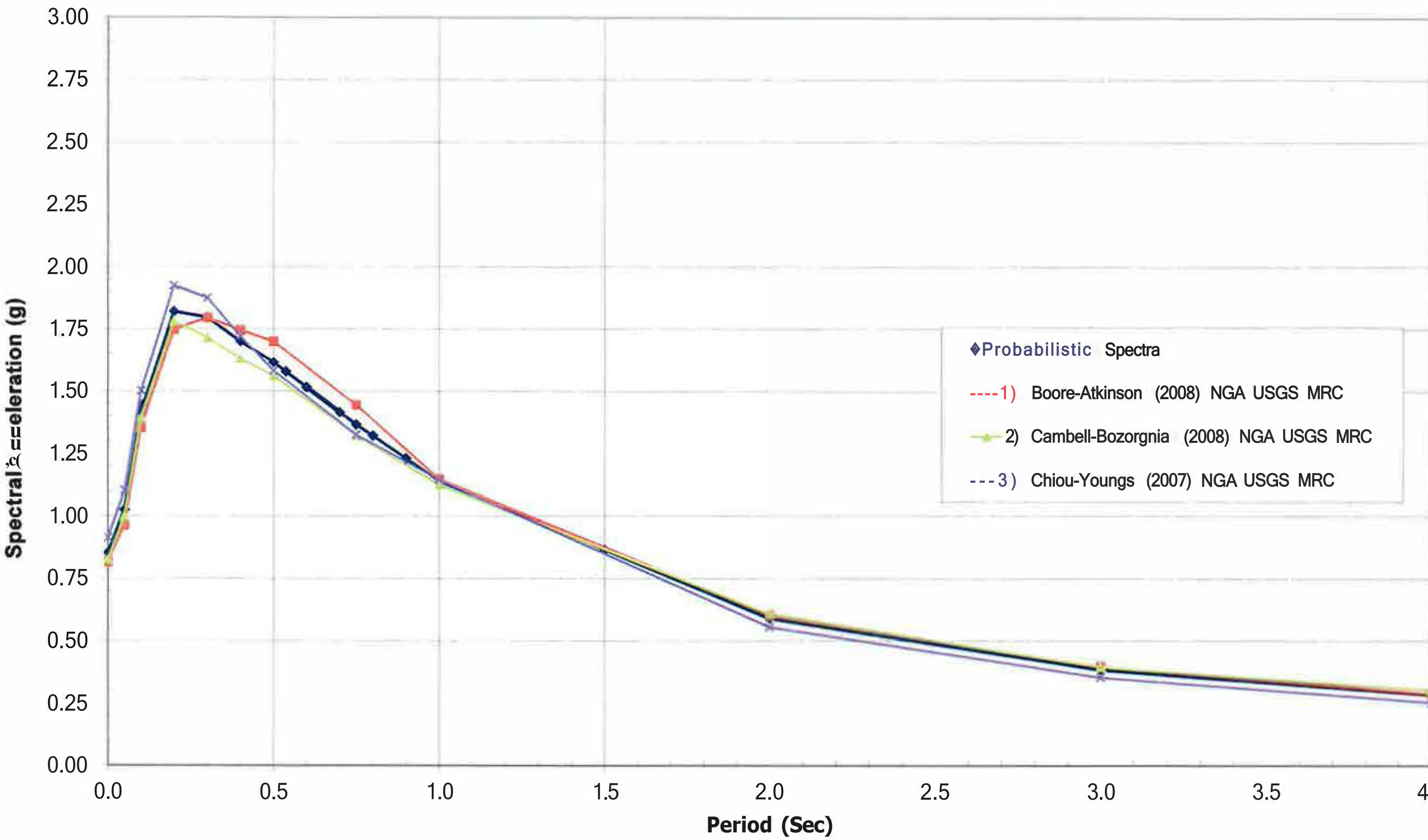
GEOTECHNICAL PROFESSIONALS, INC.

MCKINLEY AVENUE ELEMENTARY SCHOOL

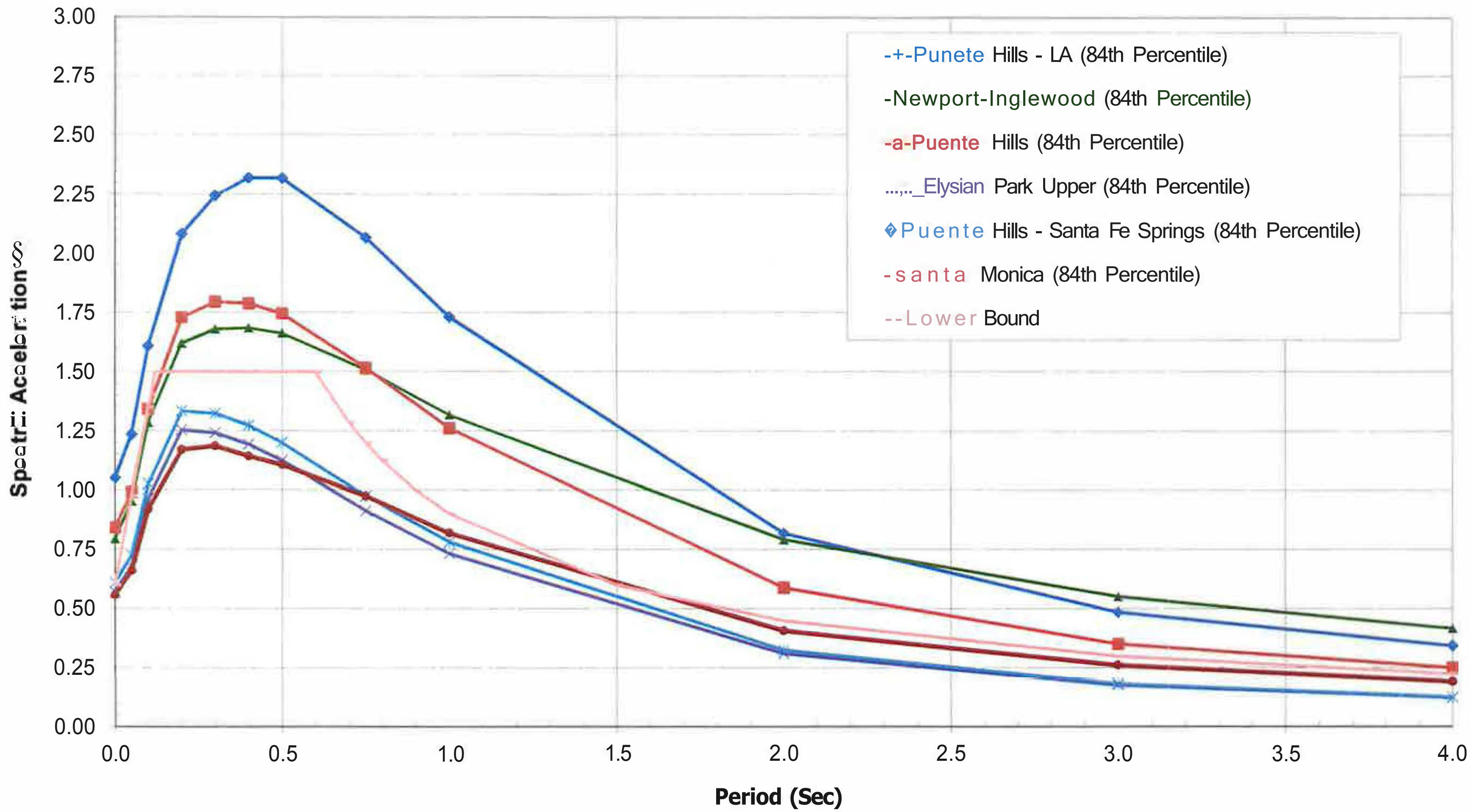
GPI PROJECT NO.: 2677.181 | SCALE: 1" = 80'

SITE PLAN

FIGURE 2



Probabilistic MCE_R Response Spectra at 5% Damping **McKinley Avenue Elementary School** **Los Angeles Unified School District** **Los Angeles, California**	GEOTECHNICAL PROFESSIONALS, INC. Project No. 2677.181 FIGURE 3

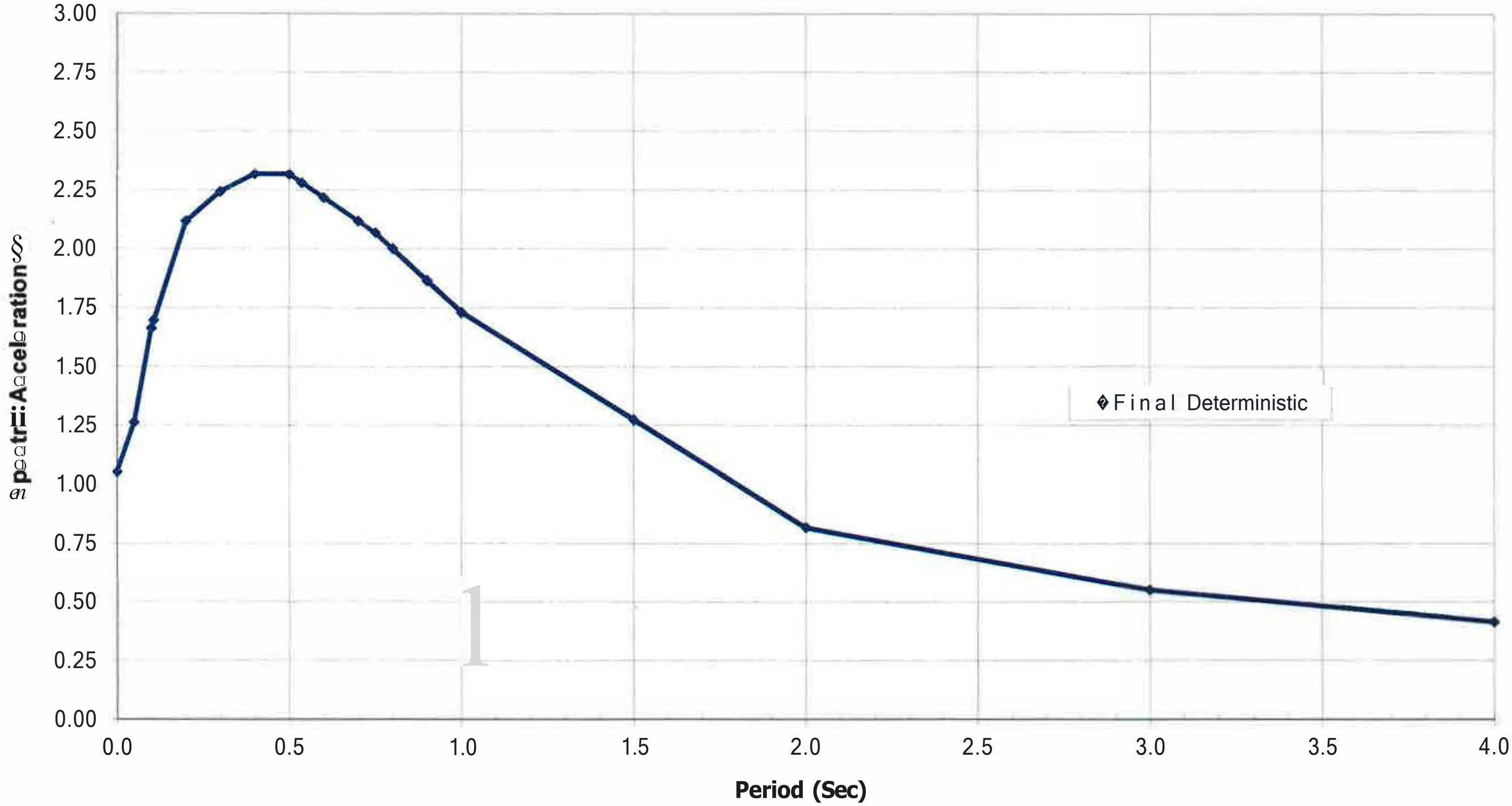


Deterministic Response Spectra at 5% Damping - Select Faults
McKinley Avenue Elementary School
Los Angeles Unified School District
Los Anaeles, California

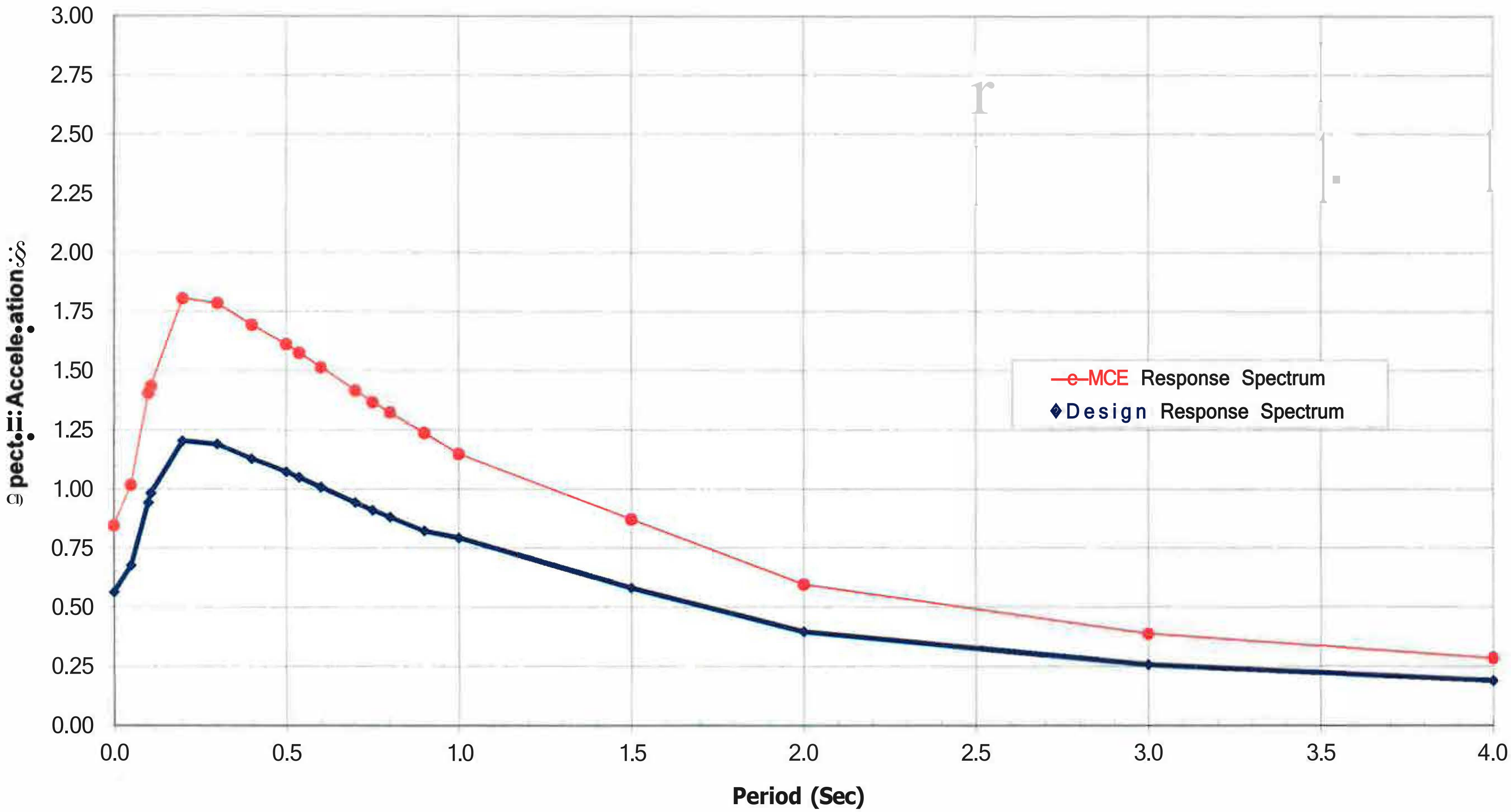
GEOTECHNICAL PROFESSIONALS, INC.

Project No. 2677.181

FIGURE4



Deterministic Response Spectrum at 5% Damping - Upper Bound McKinley Avenue Elementary School Los Angeles Unified School District Los Angeles, California	GEOTECHNICAL PROFESSIONALS, INC. Project No. 2677.181 FIGURE 5



MCE$_R$ and Design Response Spectra at 5% Damping Per 2016 CBC
McKinley Avenue Elementary School
Los Angeles Unified School District
Los Angeles, California

GEOTECHNICAL PROFESSIONALS, INC.

Project No. 2677.181

FIGURE 6

TABLE 1

RISK-TARGETED SITE SPECIFIC SEISMIC RESPONSE SPECTRA WORKSHEET

(DJB 7/28/14 Based Upon ASCE 7-10)

Project	**McKinley ES**
Proj. No.	**2677.181**
Latitude	**33.9679**
Longitude	**-118.2598**

Site Class	D
To	0.107 sec
T,	0.537 sec
TL	8.0 sec

Parameter	2016 CBC Value
S.	1.846
S1	0.661
Fa	1.000
Fv	1.500
SMs	1.846
S_{M},	0.992
Sos	1.231
S01	0.661
$S_{05}/2.5$	0.492

Parameter	2016 CBC Value
CRs	0.992
CR1	1.006
0.08 F_v!F_a	0.120
0.4F)F.	0.600
PGA_M	0.673

Attentuation Relationships

1) Boore-Atkinson (2008) NGA USGS MRC
2) Cambell-Bozorgnia (2008) NGA USGS MRC
3) Chiou-Youngs (2007) NGA USGS MRC

SITE-SPECIFIC PARAMETERS

SMs	**1.806**
SM1	**1.193**
Sos	**1.204**
So1	**0.795**
PGA	**0.57**

1)	2)	3)	4)	5)	6)	7)	8)	9)	10)	11)	12)
Period (sec)	Risk-Targeted MCE_R Spectrum (g)	2016 CBC Design Response Spectrum	Risk Coefficient C_R	MCE_R Deterministic Lower Limit Spectrum	Probabilistic Spectra; 2% in 50 years (g)	Probabilistic w/ Risk Coefficient (CR)	84th Percentile Deterministic Spectrum	Site Specific MCE_R Spectrum	2/3 Site Specific MCE_R Spectrum	80% of 2016 CBC Design Spectrum	**Design Response Spectrum**
0.000	0.673	0.449	0.992	0.600	0.855	0.848	1.052	0.848	0.565	0.359	**0.565**
0.050	1.254	0.836	0.992	0.975	1.027	1.019	1.263	1.019	0.679	0.669	**0.679**
0.100	1.769	1.180	0.992	1.350	1.418	1.407	1.664	1.407	0.938	0.944	**0.944**
0.107	1.846	1.231	0,992	1.406	1.448	1.436	1.698	1.436	0.958	0.985	**0.985**
0.200	1.846	1.231	0.992	1.500	1.821	1.806	2.118	1.806	1.204	0.985	**1.204**
0.300	1.846	1.231	0.994	1.500	1.797	1.786	2.245	1.786	1.191	0.985	**1.191**
0.400	1.846	1,231	0.996	1.500	1.702	1.694	2.320	1.694	1.130	0.985	**1.130**
0.500	1.846	1.231	0.997	1.500	1.617	1.613	2.319	1.613	1.075	0.985	**1.075**
0.537	1.846	1.231	0.998	1.500	1.580	1.576	2.282	1.576	1.051	0.985	**1.051**
0.600	1.653	1.102	0.999	1.500	1.517	1.515	2.219	1.515	1.010	0.881	**1.010**
0.700	1.416	0.944	1.001	1.286	1.416	1.417	2.119	1.417	0.945	0.755	**0.945**
0.750	1.322	0.881	1.002	1.200	1.366	1.368	2.069	1.368	0.912	0.705	**0.912**
0.800	◆.239	0.826	1.003	1.125	1.321	1,325	2.002	1.325	0.883	0.661	**0.883**
0.900	1.102	0,734	1.004	1.000	1.232	1.237	1.867	1.237	0.825	0.588	**0.825**
1.000	0.992	0.661	1.006	0.900	1.143	1.150	1.732	1.150	0.767	0,529	**0.795**
1.500	0.661	0.441	1.006	0.600	0.868	0.873	1.276	0.873	0.582	0.353	**0.582**
2.000	0.496	0.331	1.006	0.450	0.593	0.596	0.820	0.596	0.398	0.264	**0.398**
3.000	0.331	0.220	1.006	0.300	0.386	0,388	0.553	0.388	0.259	0.176	**0.259**
4.000	0.248	0.165	1.006	0.225	0.285	0.286	0.416	0.286	0.191	0.132	**0.191**

SITE SPECIFIC SEISMIC RESPONSE SPECTRA WORKSHEET
(DJB 7/28/14 Based Upon ASCE 7.10)

INPUT BLUE ONLY - BLACK CALCULATED

Column Descriptions

01) Periods including To and Ts calculated from US Seismic Design Maps (2010 ASCE)
02) USGS, U.S. Seismic Design Maps Web Application - MCEis Response Spectrum
03) USGS, U.S. Seismic Design Maps Web Application - Design Spectrum (2/3 of Column B)
04) Risk Coefficient, C_R, for 0.2s and 1.0s periods (Section 21.2.1.1); from Web Application
05) Deterministic Lower Limit on MCEis (Figure 21.2-1)
06) EZ-Frisk, 2% in 50 years Probabilistic Spectrum (Section 21.2.1.1)
07) EZ-Frisk, Probabilistic MCE_R Spectrum (Section 21.2.1.1)
08) EZ-Frisk, 84th Percentile Deterministic Spectrum (Section 21.2.2)
09) Site-Specific M C � (Section 21.2.3); Lesser of Column 7 and Greater of Columns 5 and 8
10) Uncorrected Design Response Spectrum (Section 21.3), 2/3 of Column 9
11) 80% of 2013 CBC Design Spectra (Column 3), (Section 21.3) Lower Limit of the Design Spectrum
12) Design Response Spectrum (Section 21.3); Greater of Columns 10 and 11

T_L = Figure 22-12 ASCE 7-10 (typically 8 sec Southern California)

Minimum Allowable Value of PGA: (80% of PG,¾)	**0.538**
Minimum Allowable Value of Sos: (90% of Sos at any period)	**1.071**
Minimum Allowable Value of S_{D1}: (200% of S. at 2 sec)	**0.795**

MUST CHECK THAT VALUES EXCEED MINIMUMS

APPENDIX A

APPENDIX A

CONE PENETRATION TESTS

The subsurface conditions were investigated by performing five Cone Penetration Tests (CPT's) at the site. The soundings were advanced to depths of 40 to 60 feet below existing grades. The locations of the CPT's are shown on the Site Plan, Figure 2.

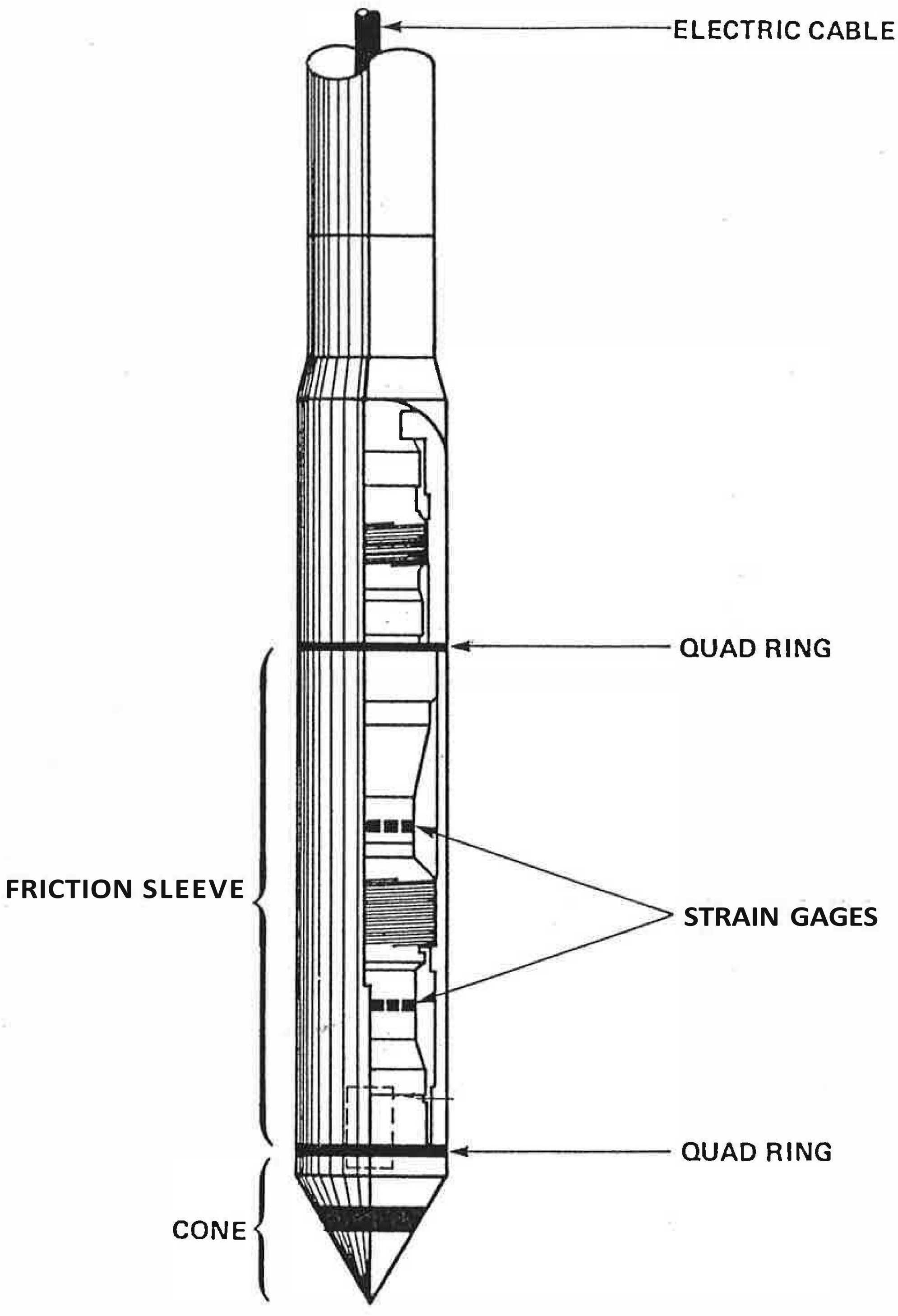
The Cone Penetration Test consists of pushing a cone-tipped probe into the soil deposit while simultaneously recording the cone tip resistance and side friction resistance of the soil to penetration (refer to Figure A-1). The CPT described in this report was conducted in general accordance with ASTM 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. 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.

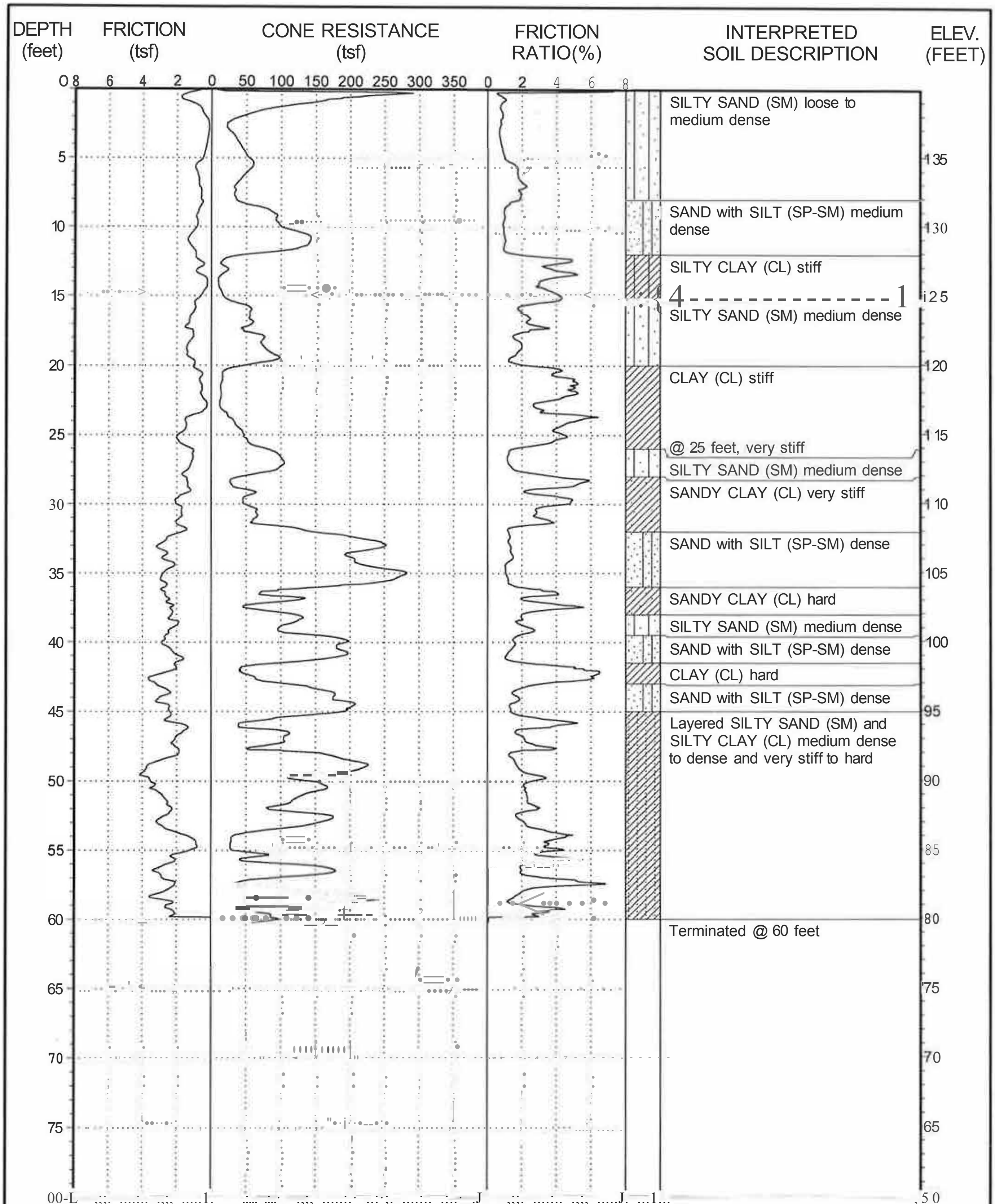
The CPT locations were laid out in the field by measuring from existing site features. Upon completion, the uncaved portions of the CPT holes were backfilled with bentonite chips. CPT's performed in asphalt or concrete areas were patched with cold-patch asphalt or rapid-set concrete, respectively. Ground surface elevations at the CPT locations were estimated from internet sources and should be.considered approximate.



GPI GEOTECHNICAL PROFESSIONALS INC.

CONE PENETROMETER

FIGURE A-1



Date performed:4-12-17

This summary applies only at the location of this cone penetration lest and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location wllh the passage of time. The interpreted soil description is derived from the friction ratio and co11e resistance and is a simplification of actual conditions encountered.

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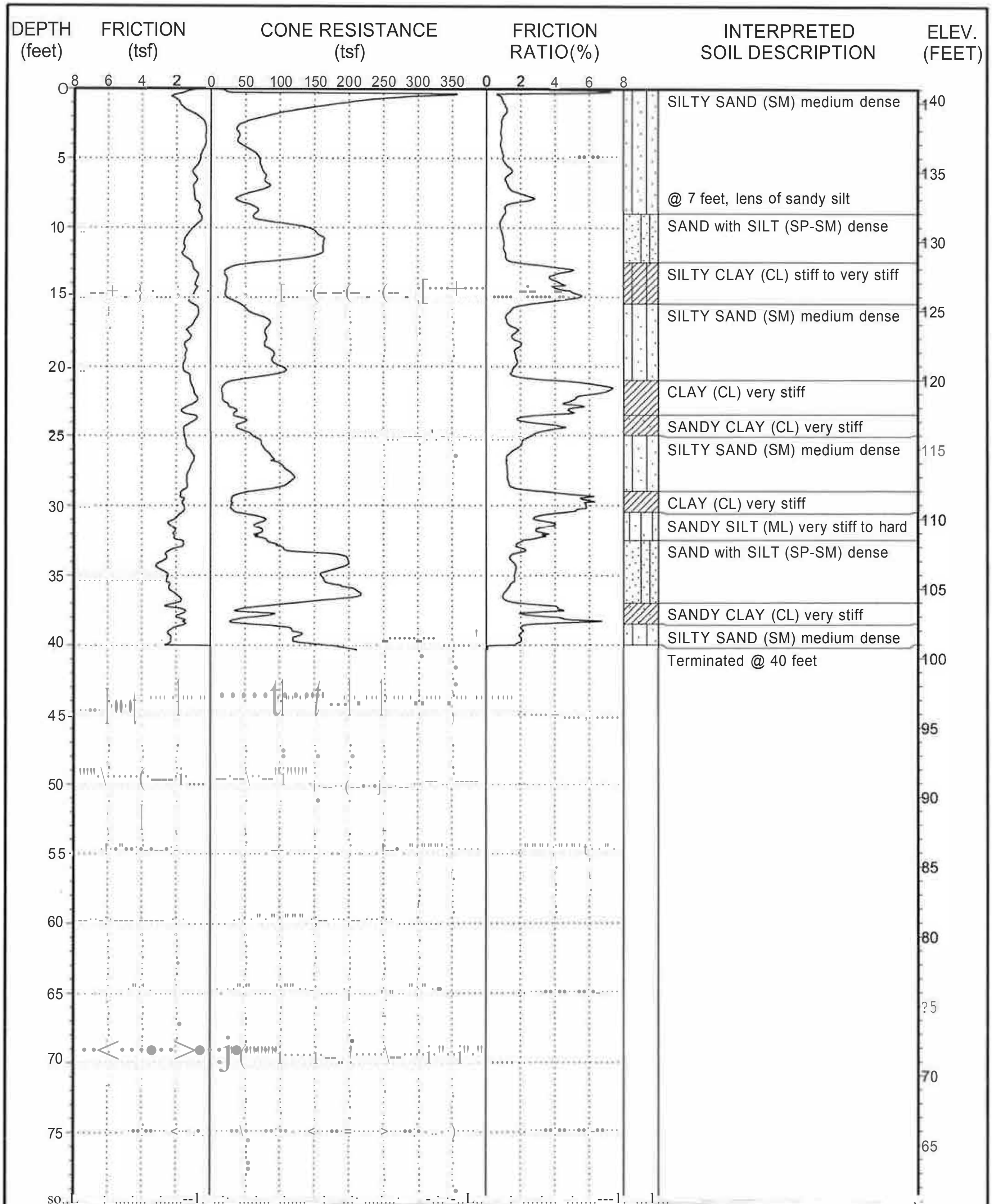
LOG OF CPT NO. C-1

FIGURE A-2



Date performed:4-12-17

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

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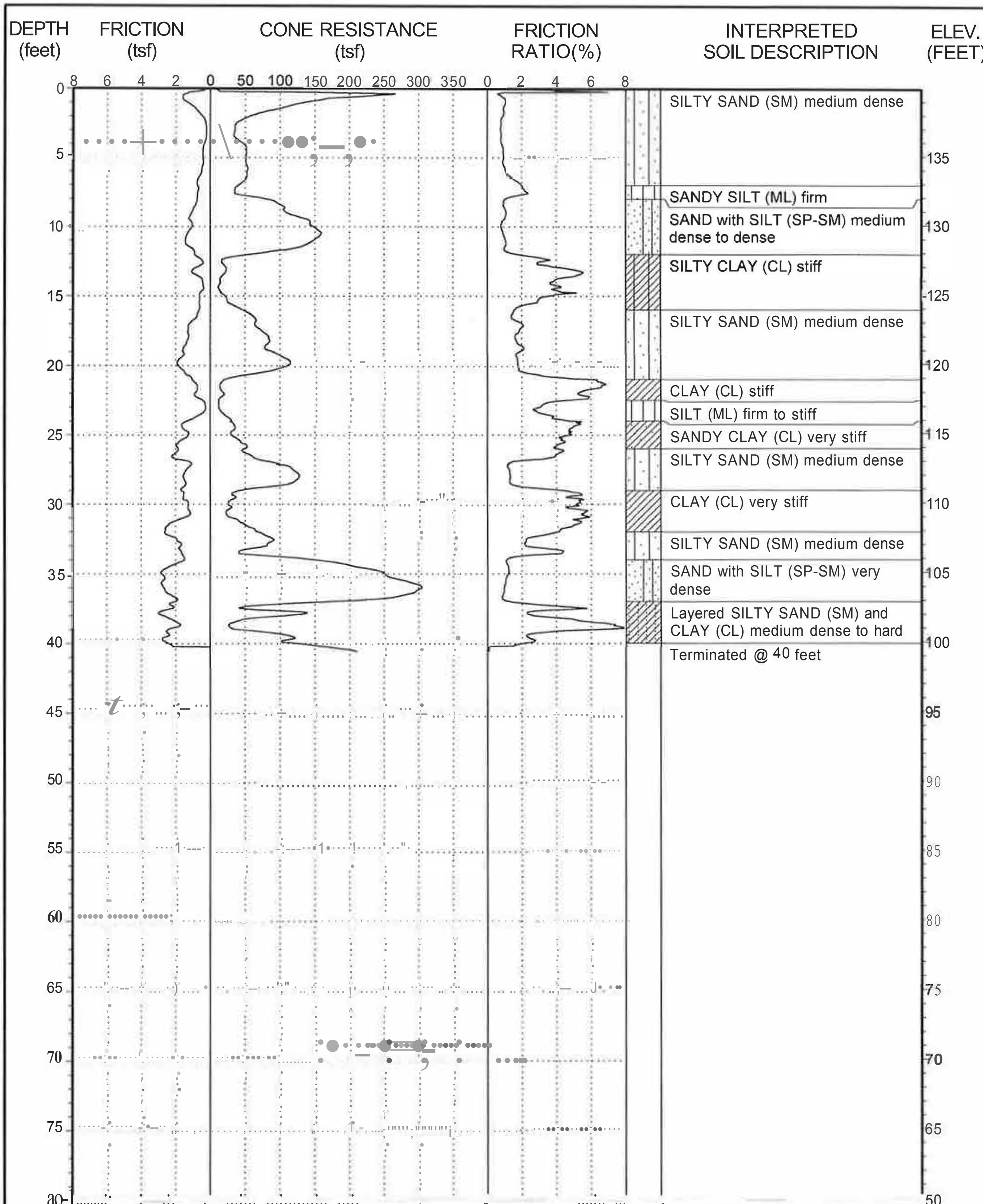
LOG OF CPT NO. C-2

FIGURE A-3



Date performed:4-12-17

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

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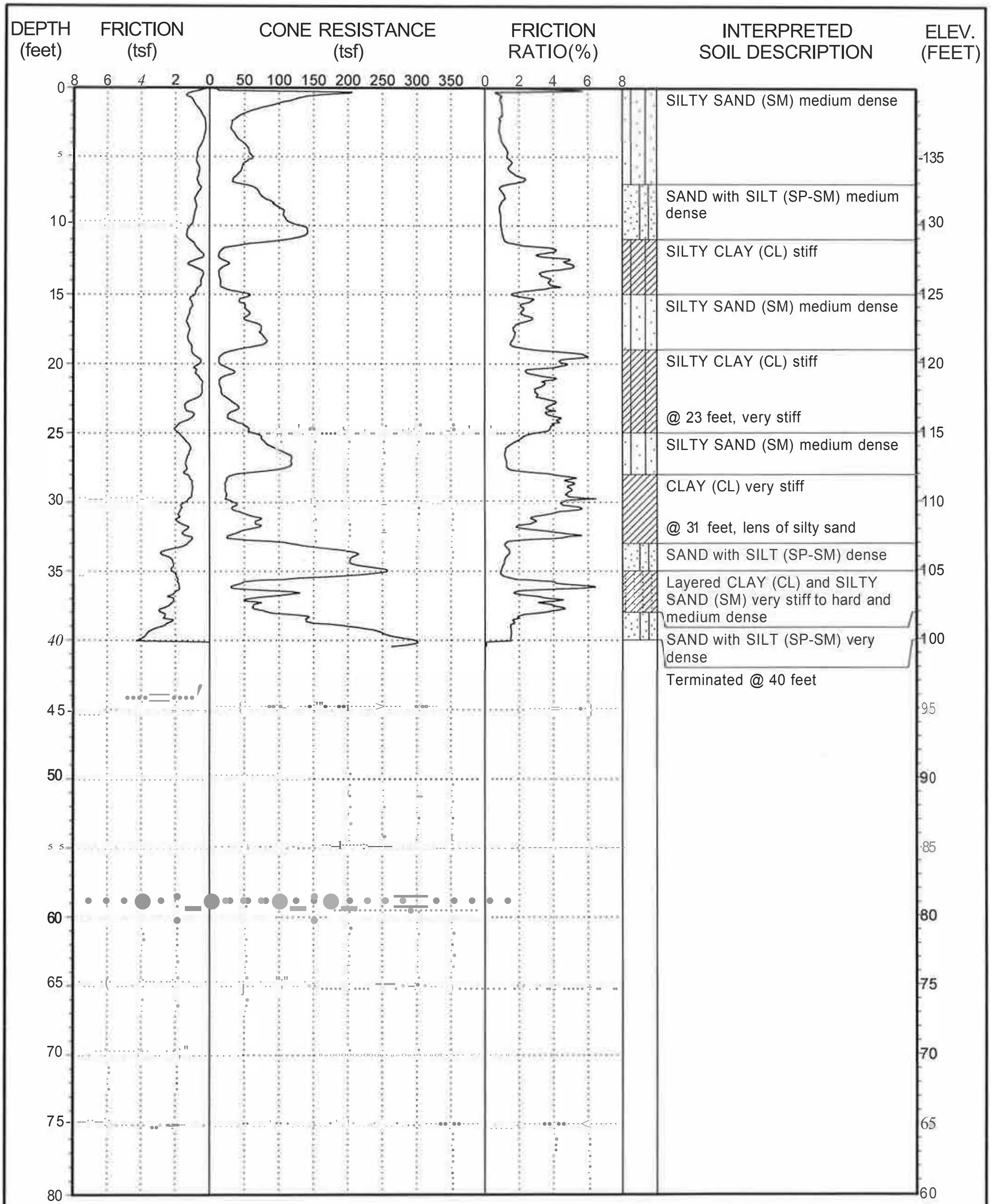
LOG OF CPT NO. C-3

FIGURE A-4



Date performed:4-12-17

This summary applies only at the location of this cone penetration test and at the lime of the exploration. Subsurface conditions may differ al other locations and may change at this location wi1h the passage of time. The interpreted soil description is derived from the friction rat.io and cone resistance and is a simplification of actual conditions encountered.

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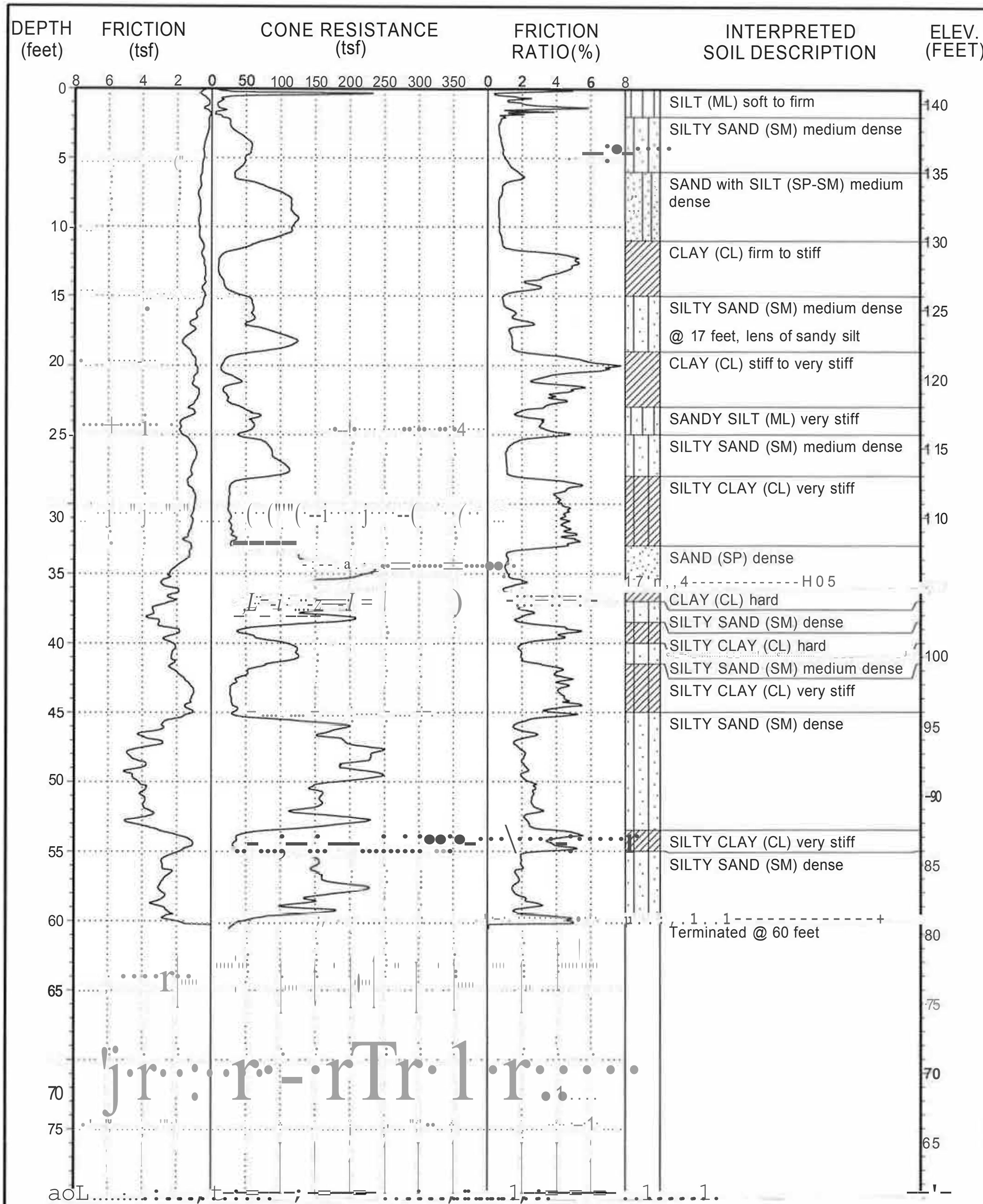
LOG OF CPT NO. C-4

FIGURE A-5



Date performed:4-12-17

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

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LOG OF CPT NO. C-5

FIGURE A-6

APPEND/XS

APPENDIX B

EXPLORATORY BORINGS

We investigated the subsurface conditions at the site by drilling and sampling four exploratory borings. The borings were advanced to depths ranging from 20 to 60 feet below the existing ground surface. The locations of the explorations are shown on the Site Plan, Figure 2.

The 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.

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

The field explorations for the investigation were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The 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-4 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 internet sources and should be considered approximate.

Moisture (%)	Dry Density (pcf)	Blow Count	Sample Type	Depth (ft)	DESCRIPTION OF SUBSURFACE MATERIALS	Elevation
					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 simplificahon of actual conditions encountered.	
			B	0	Pavement: 5" AC over 0" BASE	
					Fill: **SILTY SAND (SM)** brown, moist	
8.2	95	10	D		Natural: **SILTY SAND (SM)** brown, moist, loose	
19.5	90	18	D	5	@ 5 feet, wet, medium dense	135
15.7	97	15	D		@ 7 feet, very moist	
					SAND with SILT (SP-SM)brown, moist, loose	
9.4	97	12	D	10		130
					SILTY CLAY (CL) brown, very moist, stiff	
20.9	99	16	D	15	**SILTY SAND (SM)** brown, wet, medium dense	125
36.0		6	S	20	**CLAY (CL)** brown, wet, firm	120
19.3	106	16	D	25	**SILTY SAND (SM)** brown, wet, medium dense	115
18.1		18	S	30		110
					SILTY CLAY (CL) brown, dry, very stiff	
3.7	100	30	D	35	**SILTY SAND (SM)** brown, slightly moist, medium dense	105

SAMPLE TYPES
[9 Rock Core
ffi) Standard Split Spoon
[QI Drive Sample
@ Bulk Sample
IT) Tube Sample

DATE DRILLED:
4-14-17
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (fl):
Not Encountered

PROJECT NO.: 2677.181
MCKINLEY AVENUE ES

LOG OF BORING NO. B-1

FIGURE 8-1

						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 chan9e at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
	16.4		28	S	40	**SILTY SAND (SM)** brown, wet, medium dense	
						CLAY (CL) brown, very moist, hard	
						SILTY SAND (SM) brown, very moist, medium dense	
					45		95
						Layered **SILTY SAND (SM) and CLAVEY SILT (ML)** brown, very moist to wet, medium dense and very stiff	
	21.0		28	S	50		90
					55		85
	20.8		28	S			
					60	Total Depth 60 feet	80

SAMPLE TYPES
- Rock Core
- Standard Split Spoon
- Drive Sample
- Bulk Sample
- Tube Sample

DATE DRILLED:
4-14-17

EQUIPMENT USED:
B" Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2677.181
MCKINLEY AVENUE ES

LOG OF BORING NO. B-1

FIGURE B-1

						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.	
					0	Fill: **SANDY SILT (ML)** brown, moist	140
	9.8	94	23	D	5	Natural: **SILTY SAND (SM)** brown, moist, medium dense	135
	9.1	92	29	D	10	**SAND with SILT (SP-SM)** brown, moist, medium dense	130
						SILTY CLAY (CL) brown, very moist, stiff	
	18.6	103	20	D	15		
						SILTY SAND (SM) brown	125
	23.4	87	21	D	20	**SANDY SILT (ML)** brown, wet, stiff	
						Total Depth 20 feet	

SAMPLE TYPES
[g] Rock Core
[ID Standard Split Spoon
[Q] Drive Sample
[ID Bulk Sample
[TI Tube Sample

DATE DRILLED:
4-14-17
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GP-I

PROJECT NO.: 2677.181
MCKINLEY AVENUE ES

LOG OF BORING NO. B-2

FIGURE 8-2

					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 chanpe at this location with the passage of time. The data presented is a simplificat10n of actual condiLions encountered.	
			B	0	Fill: **SILTY SANO (SM)** brown, moist	
11. 1	101	14	D		Natural: **SILTY SANO (SM)** brown, moist, loose	
7.9	95	18	D	5	@ 4 feet, slightly moist, medium dense	135
6.3	103	24	D		@ 6 feet, very stiff	
7.3	97	32	D	10	**SANO with SILT (SP-SM)brown,** slightly moist, medium dense	130
17.0	103	16	D	1J	Layered **SANDY SILT (ML) and CLAY (CL)** brown, very moist, stiff	125
					SILTY SANO (SM) brown, very moist, medium dense	
30.1	93	17	D	20	**SILTY CLAY** (CL)dark brown, wet, stiff	120
17.2		13	S	25	**SILTY SANO (SM)** brown, wet, medium dense	115
				30		110
					SANO with SILT (SP-SM) light brown, slightly moist, medium dense	
5.2		34	S			
				35	Total Depth 35 feet	105

SAMPLE TYPES
Rock Core
[§] Standard Split Spoon
[Q] Drive Sample
ffi] Bulk Sample
IT] Tube Sample

DATE DRILLED:
4-14-17
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2677.181
MCKINLEY AVENUE ES

LOG OF BORING NO. 8-3

FIGURE B-3

					DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the lime of drilling. Subsurface conditions maYi differ at other locations and may change at this location with the passage oft me. The data presented is a simplification of actual conditions encountered.	
			B	0	pavement: 3" AC over 3.5" BASE	
					Fill: **SILTY SAND (SM)** brown, moist	
7.3	92	11	D		Natural: **SILTY SAND (SM)** brown, moist, loose	
32.4	78	13	D	5	@ 4 feet, lens of silt, wet	135
8.1	94	21	D		**SAND with SILT (SP-SM)** brown, wet, medium dense	
14.0	89	27	D		@ 9 feet, wet	130
				10	**CLAVEY SILT (ML)** brown, moist, very stiff, trace sand	
17.8	98	16	D			125
				15	**SILTY SAND (SM)** brown, very moist, medium dense	
18.9	99	20	D			120
				20	**SILTY CLAY (CL)** brown, very moist, stiff	
15.0		16	S	25	**SILTY SAND (SM)** dark brown to brown, moist to very moist, medium dense	115
					CLAY (CL) brown, moist to very moist, very stiff	110
				30		
					SAND with SILT (SP-SM)light brown, slightly moist, medium dense	
6.4		31	S			105
				35	Total Depth 35 feet	

SAMPLE TYPES
[gJ Rock Core
[§] Standard Split Spoon
[Q] Drive Sample
I]] Bulk Sample
IT] Tube Sample

DATE DRILLED:
4-14-17
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2677.181
MCKINLEY AVENUE ES

LOG OF BORING NO. B-4

FIGURE B-4

APPENDIXC

APPENDIX C

LABORATORY TESTS

INTRODUCTION

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

MOISTURE CONTENT AND DRY DENSITY

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

ATTERBERG LIMITS

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

GRAIN SIZE DISTRIBUTION

Select soil samples were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. That portion of the material retained on the No. 200 sieve was oven-dried and weighed to determine the percentage of the material passing the No. 200 sieve. A summary of the percentages passing the No. 200 sieve is presented below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
8-1	0 - 5	Silty Sand (SM)	25
8-1	10	Sand with Silt (SP-SM)	7
8-1	30	Silty Sand (SM)	43
8-2	4	Silty Sand (SM)	25
8-3	0 - 5	Silty Sand (SM)	37
8-3	24	Silty Sand (SM)	39
8-3	33.5	Sand with Silt (SP-SM)	8

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-4	0 - 5	Silty Sand (SM)	29
B-4	6	Sand with Silt (SP-SM)	8

DIRECT SHEAR

Direct shear tests were performed on select samples in accordance with ASTM D 3080. Tests were performed on relatively undisturbed samples and samples remolded to 90 percent relative compaction. 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 Figures C-2 to C-4.

CONSOLIDATION

One-dimensional consolidation testing was performed on selected undisturbed samples in accordance with ASTM D 2435. After trimming the ends, the samples were placed in the consolidometer and loaded to 0.4ksf. Thereafter, the samples were incrementally loaded to a maximum load of 25.6 ksf. The samples were inundated at 0.8 or 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 and C-6.

EXPANSION INDEX

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

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX
B-4	0 - 5	Silty Sand (SM)	1

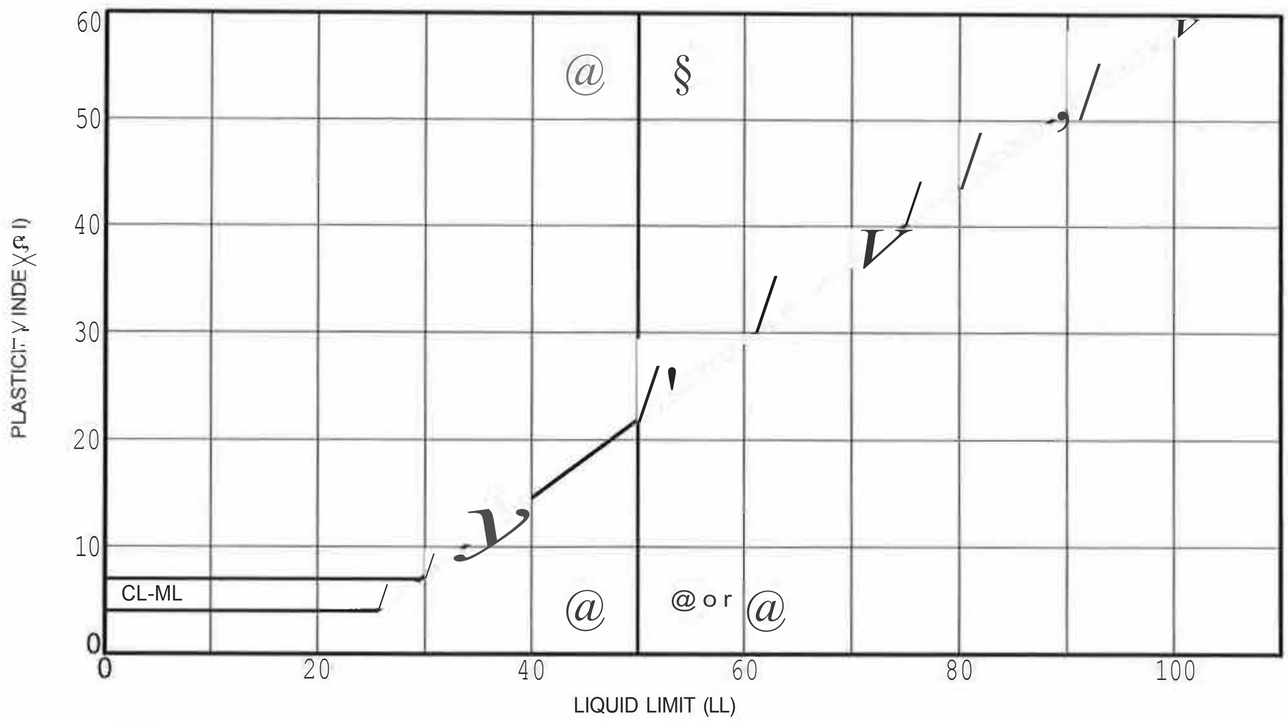
COMPACTION TEST

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

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	OPTIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)
B-3	0 - 5	Silty Sand (SM)	10.5	126

CORROSIVITY

Soil corrosivity testing was performed by HOR on selected soil samples provided by GPI. The test results and corrosion protection recommendations are summarized in Table 1 of this Appendix.



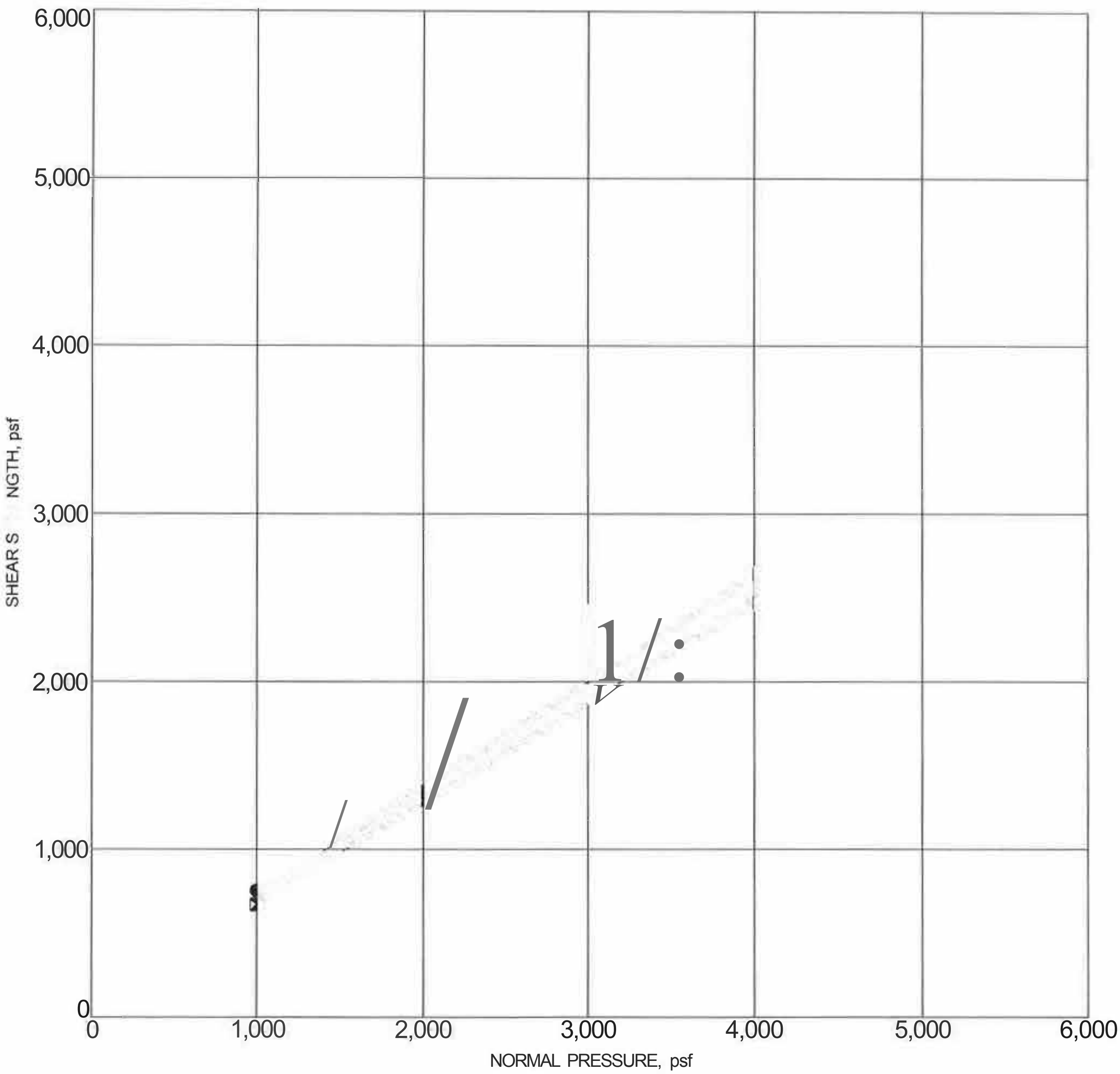
	SAMPLE LOCATION		LL	PL	PI	Fines,%	Classification
•	B-1	20.0	36	22	14		CLAY (CL)

PROJECT: MCKINLEY AVENUE ES

PROJECT NO. 2677.181

ATTERBERG LIMITS TEST RESULTS

FIGURE C-1



e PEAK STRENGTH
Friction Angle = 32 degrees
Cohesion = 108 psf

1%1ULTIMATE STRENGTH
Friction Angle = 31 degrees
Cohesion = 82 psf

Sample Location		Classification	DD,pcf	MC,%
8-1	5.0	SILTY SAND (SM)	90	19.5

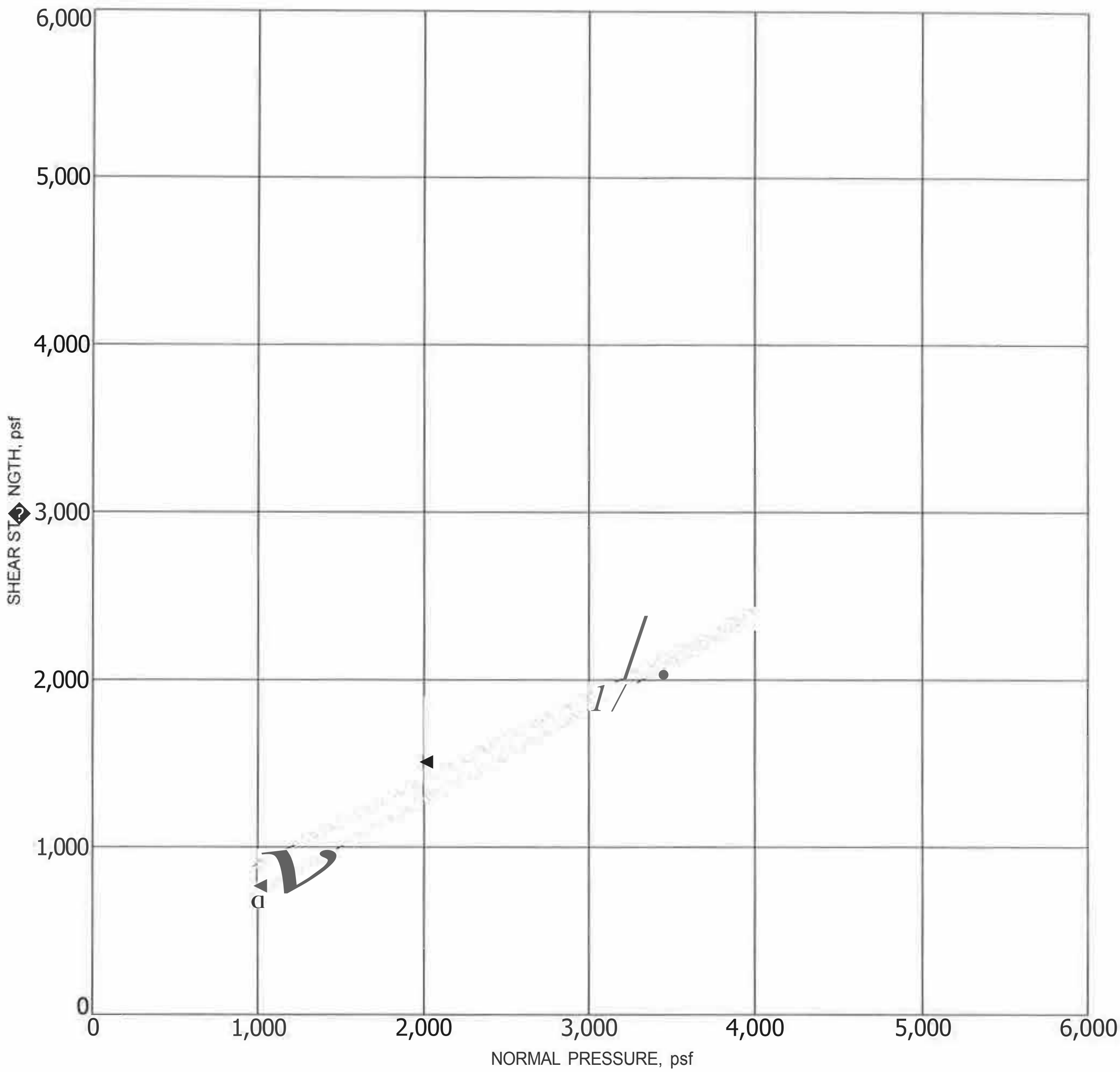
PROJECT: MCKINLEY AVENUE ES

PROJECT NO.2677.18I

.CII F

DIRECT SHEAR TEST RESULTS

FIGURE C-2



● **PEAK STRENGTH**
Friction Angle = 27 degrees
Cohesion = 396 psf

■ **ULTIMATE STRENGTH**
Friction Angle = 29 degrees
Cohesion = 180 psf

Note: Samples remolded to 90% of maximum dry density

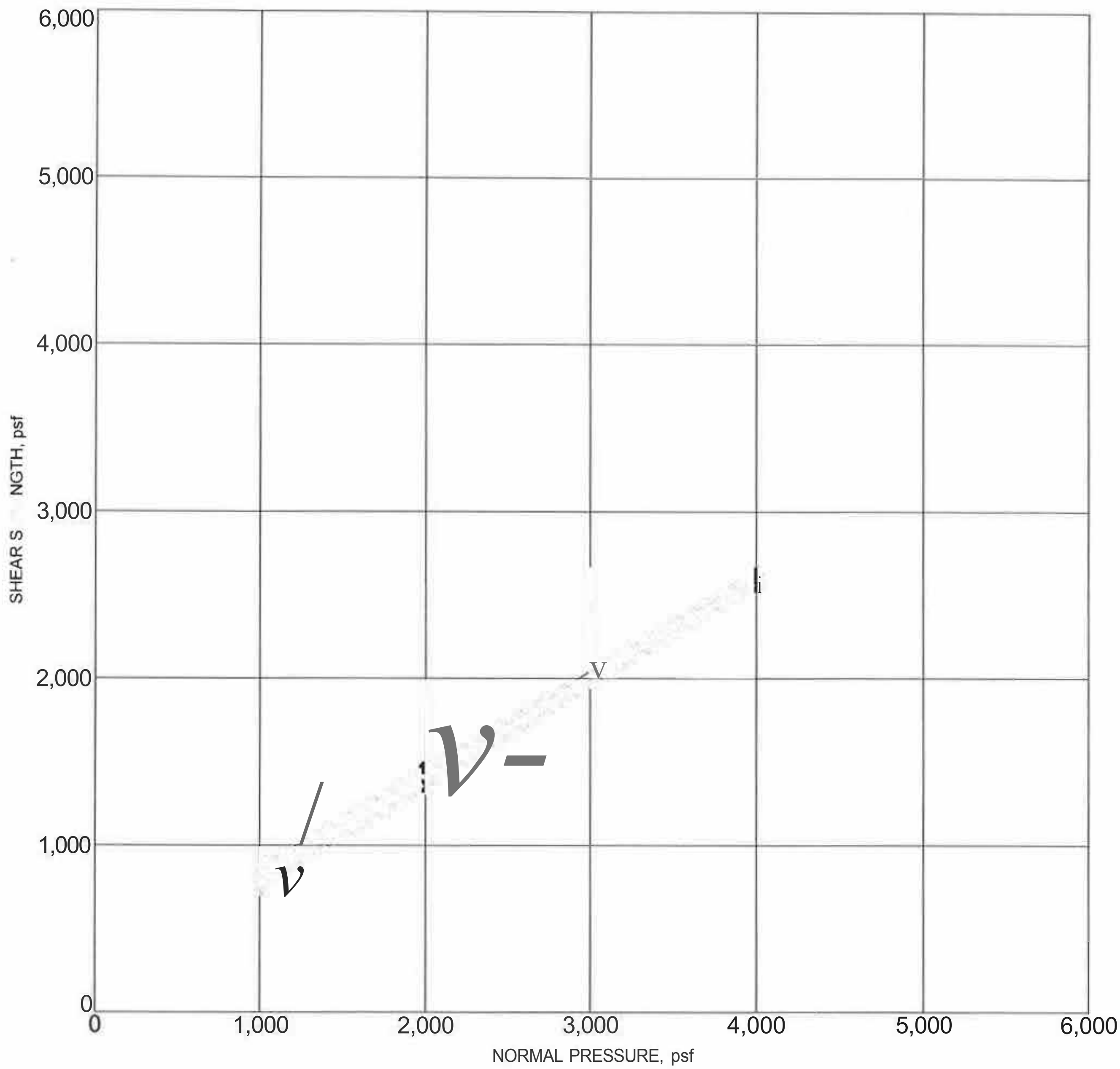
Sample Location		Classification	DD,pcf	MC,%
B-3	0-5	SILTY SAND (SM)	113	10.5

PROJECT: MCKINLEY AVENUE ES

PROJECT NO.2677.18I

DIRECT SHEAR TEST RESULTS

FIGURE C-3



e PEAK STRENGTH
Friction Angle = 30 degrees
Cohesion= 282 psf

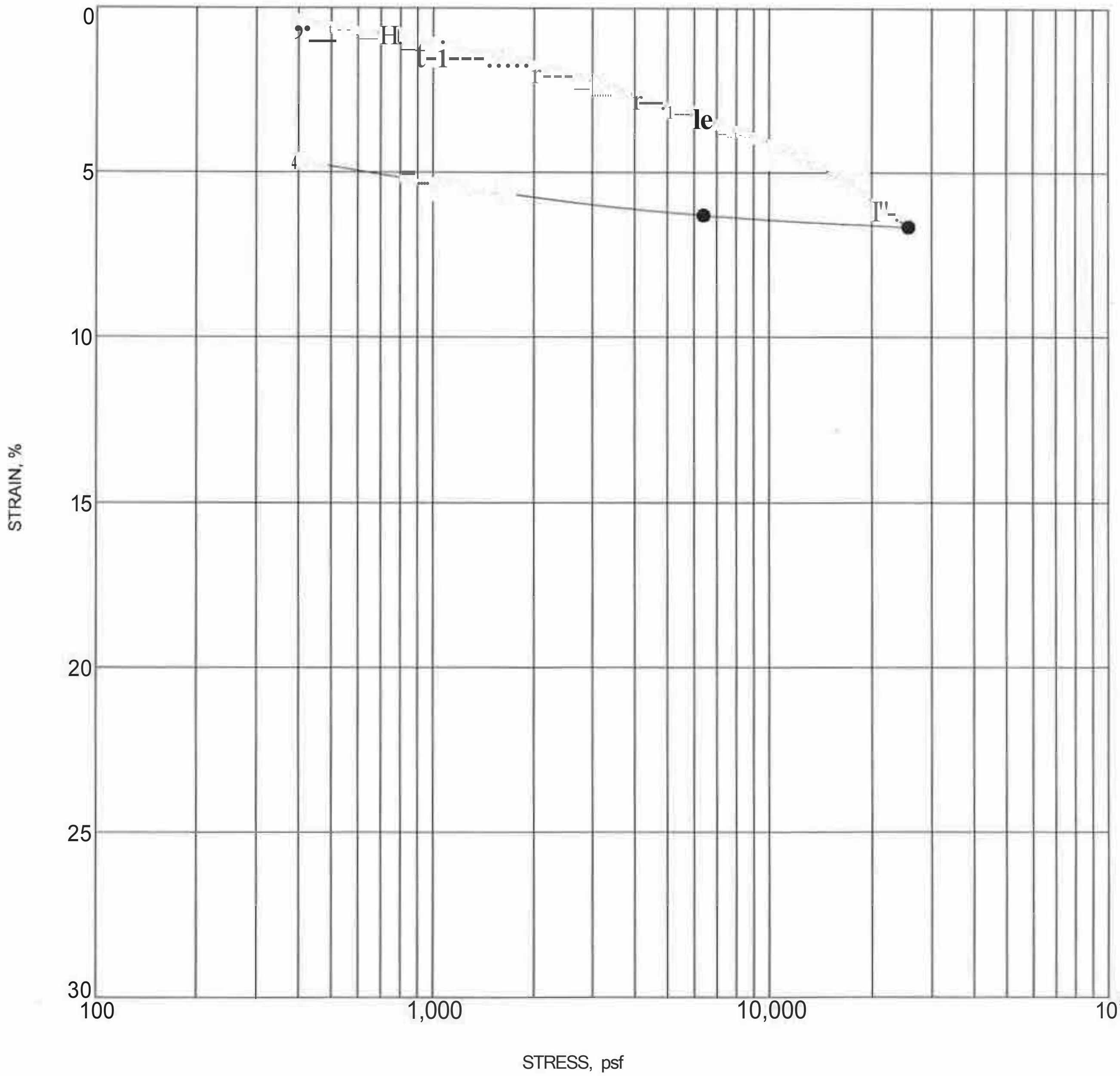
III ULTIMATE STRENGTH
Friction Angle = 31 degrees
Cohesion = 144 psf

Sample Location		Classification	DD,pcf	MC,%
B-3	14.0	SANDY SILT (ML)	103	17.0

PROJECT: MCKINLEY AVENUE ES PROJECT NO.2677.18I

DIRECT SHEAR TEST RESULTS

FIGURE C-4



Sample inundated at 800 psf

	Sample Location		Classification	DD,pcf	MC,%
●	8-1	7.0	SILTY SAND (SM)	97	15.7

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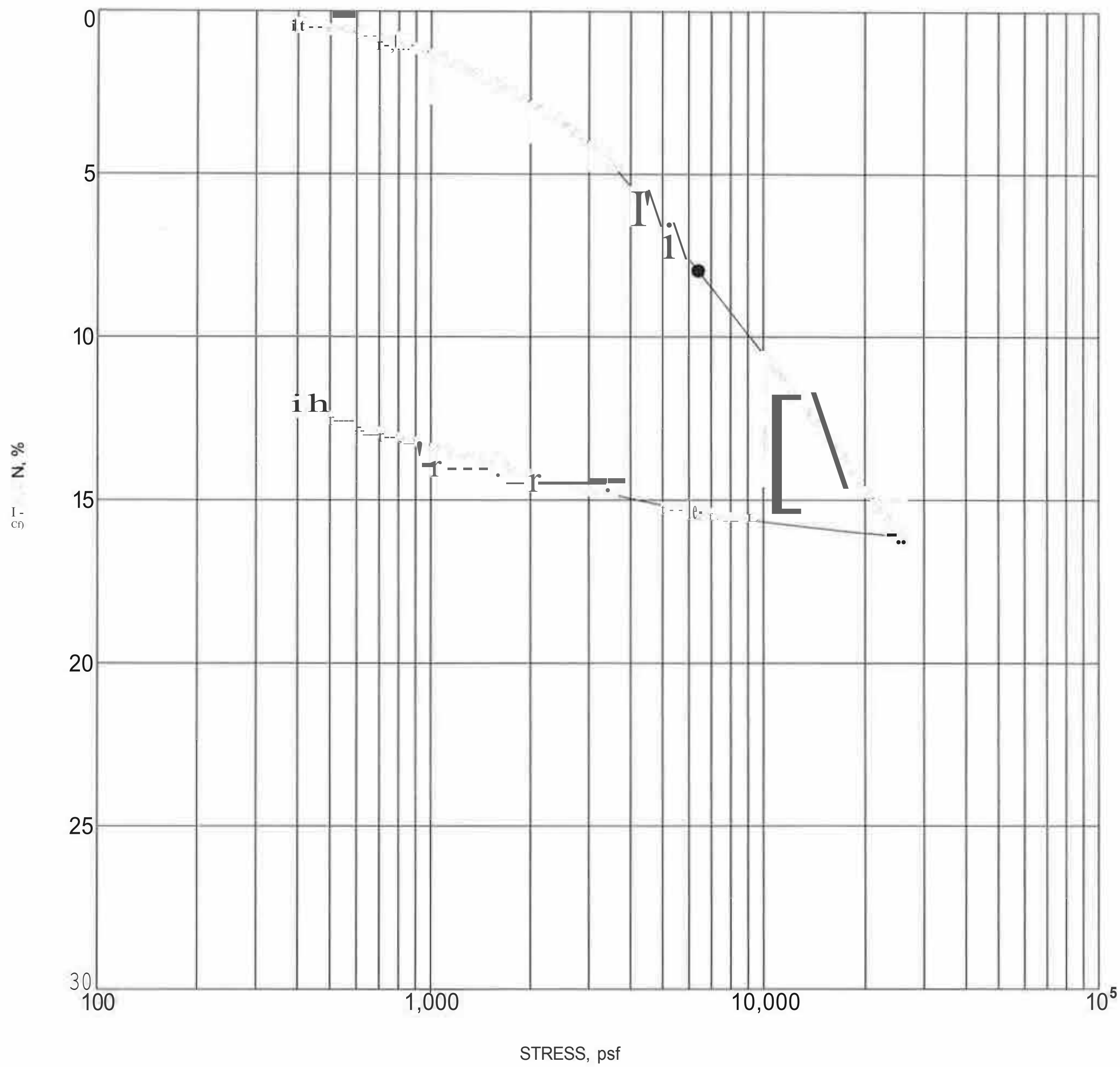
PROJECT NO.: 2677.181

CONSOLIDATION TEST RESULTS

FIGURE C-5



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	8-3	19.0	SILTY CLAY (CL)	93	30.1

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PROJECT NO.: 2677.181

CONSOLIDATION TEST RESULTS

FIGURE C-6

1-)

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
McKinley E.S.
Your#2677.1BI, HDR Lab #17-0273LAB
26-Apr-17

Sample ID			
			B-3 @0-5'
Resistivity		**Units**	
as-received		ohm-cm	35,200
saturated		ohm-cm	4,400
pH			7.4
Electrical Conductivity		mS/cm	0.10
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	74
magnesium	Mg^{2+}	mg/kg	8.1
sodium	Na^{1+}	mg/kg	55
potassium	K^{1+}	mg/kg	11
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	305
fluoride	F^{1-}	mg/kg	ND
chloride	Cl^{1-}	mg/kg	ND
sulfate	SO_4^{2-}	mg/kg	14
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
ammonium	NH_4^{1+}	mg/kg	**ND**
nitrate	NO_3^{1-}	mg/kg	13
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.
Electrical conductivity in millisiemens/cm and chemical analyses 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

APPENDIX O

APPENDIX D

GEOLOGIC-SEISMIC HAZARD EVALUATION

INTRODUCTION

This geologic and seismic assessment presents a summary of geologic and seismic conditions at McKinley Avenue Elementary School located at 7812 McKinley Avenue in Los Angeles, California. The purpose of this assessment was to identify and evaluate geologic constraints, which are likely to be factors with respect to the proposed development. In order to accomplish this objective, the following scope of services was performed:

- Research and review of available published and unpublished geologic literature and maps pertaining to the site and vicinity (see References), as well as subsurface exploration data from our recent investigation.
- Geologic analysis of the reviewed information.
- Preparation of this assessment report, which includes a summary of the researched information and a discussion of the possible geologic-seismic hazards that may affect the subject site and the proposed construction.

SITE CONDITIONS

The subject school site is approximately 4.85 acres and is located in an older residential neighborhood of suburban Los Angeles.

Site topography at the school is relatively flat, with ground surface elevations ranging from approximately 139 to 141 feet. Within the vicinity of the site, the ground surface slopes very gradually to the south.

REGIONAL AND LOCAL GEOLOGIC SETTING

Regional Geology

The proposed school site is located in the Central Block of a regional geologic structure termed the Los Angeles Basin, a northeast-trending structural basin filled with Tertiary age marine sedimentary rocks mantled by Recent and Pleistocene age non-marine alluvial sediments deposited by washes and streams flowing southward from the San Gabriel Mountains, Elysian, and Repetto Hills to the north.

In the area of the site, the marine deposits are overlain by approximately 20 feet of Holocene alluvium, which consist of loose to dense sands, silty sands, and silts. The Pleistocene alluvium consists of moderately to well consolidated, gravel, sand, silt and clay (Department of Water Resources, 1961).

The nearest geologic structures to the site are the Puente Hills Blind Thrust and Newport-

Inglewood Zone of deformation, both considered active fault zones. Deformation and uplift along the Newport-Inglewood fault zone has resulted in a northwest trending series of hills, including Signal Hill and the Dominguez Hills to the southeast of the site, and the Baldwin Hills to the west and northwest of the site. Based on published maps and the USGS Source Parameter website (see References), the site is approximately 2.1 miles and 5.7 kilometers from the closest known traces of the Puente Hills Blind Thrust and Newport-Inglewood fault, respectively (see Table 1).

Regionally the site is located near the border between two of California's geomorphic provinces, the Transverse ranges to the north and the Peninsular Ranges to the south. The Transverse Ranges are characterized by east-west trending mountain ranges, including the Santa Monica and San Gabriel Mountains, that are oriented oblique to the trend of the other major structural trends in California, including the San Andreas Fault, Sierra Nevada Mountains, and other mountain ranges in Southern California, which trend northwesterly.

The Peninsular Ranges are characterized by northwesterly trending active faults and mountain ranges related to the San Andreas and other major fault systems in the province. The province extends from the Los Angeles Basin, where the project is located, southeast to Baja California.

Site Geologic Conditions

The site is underlain by Quaternary age alluvial sediments mapped as younger, alluvial plain deposits. These sediments are described as gravel, sand, and clay derived mostly from the Santa Monica Mountains and minor stream channels (Dibblee, 2007). The geologic conditions in the site area are shown on the quaternary Geologic Map, Figure 0-1.

As encountered in our exploratory borings at depths ranging from 20 to 60 feet, the soils consist of shallow undocumented fill soils over natural younger and older alluvial soils. The fill soils at the boring locations consisted of moist silty sands. The fill soils are likely undocumented and relatively old, given the age of the high school.

The underlying natural materials consisted of loose to medium dense silty sands and sands and firm to very stiff clays, silty clays, and sandy silts. The natural soils within the upper 12 feet below existing grades consisted predominantly of loose to medium dense silty sands and sands. Below depths of 12 feet, the natural soils consisted of alternating layers of firm to very stiff fine-grained soils (clays, silty clays, and sandy silts) and medium dense coarse-grained soils (silty sands and sands). The soils become dense and very stiff to hard below approximate depths of 32 to 34 feet. The natural soils are generally moist to wet, with higher moisture contents encountered within the fine-grained soils.

Groundwater Conditions

Data published by the State of California indicates that historical high groundwater depth in the site vicinity is approximately 15 feet below existing grades. Groundwater was not encountered in our borings drilled to depths of 60 feet below the existing ground surface. Details of the groundwater depths in the vicinity of the site are shown on the Groundwater Map, Figure D-3.

TECTONIC SETTING

Regional Fault Systems

The geologic structure of southern California is dominated by northwest trending faults associated with the San Andreas Fault System. Faults such as the Newport-Inglewood, Whittier, Palos Verdes Hills and San Jacinto are all considered active and are all associated with the San Andreas, which collectively form the boundary between the North American and Pacific tectonic plates. Most of these faults have ruptured the ground surface historically and/or produced significant earthquakes.

Anomalous to the general northwest structural fabric are a series of active west trending reverse or thrust faults. The majority of these occur as north dipping planes projecting along the southern base of the Santa Monica and San Gabriel Mountains in the greater Los Angeles area. The known active thrust faults in the region include the Cucamonga, Sierra Madre, San Fernando, Raymond, Santa Monica and Hollywood faults.

Concealed Faults

Another category of fault known as "blind thrusts" was recognized as a significant seismic hazard following the 1987 magnitude 6.0 Whittier Narrows Earthquake and then again by the 1994 San Fernando magnitude 6.7 Earthquake. A blind thrust is a deeply buried shallow dipping thrust fault, which does not project to the ground surface. Blind thrusts are capable of generating a major earthquake that may cause uplift in the form of anticlinal hills. Some uplands that surround the Los Angeles Basin, including the Elysian Park and Repetto Hills, are products of blind thrusts. Because blind thrusts do not intersect the ground surface, primary surface fault rupture is considered unlikely. Major portions of the Los Angeles Basin are now believed to be underlain by various blind thrusts ramps. Due to continued north-south convergence (shortening) across the Los Angeles Basin, slippage along these features will generate earthquakes.

At the present time, the potential magnitudes and recurrence intervals of blind thrust produced earthquakes cannot be quantified with confidence due to the fact that many characteristics of these features (including areal extent and Quaternary slip rates) are poorly understood. Nonetheless, the proximity to densely populated urban centers and their history of producing damaging earthquakes clearly demonstrate the risk that blind thrusts pose to large metropolitan areas such as Los Angeles and surrounding cities.

Nearby Seismogenic Sources

We reviewed the 2008 National Seismic Hazard Maps Source Parameters (USGS, 2008) to identify known active faults within a 100 km radius of the project site. The names and distances of the faults lying within 25 kilometers of the project site are provided in the following table (Table 1). We present a map showing the significant regional faults in Figure D-3, Regional Fault Map.

Table 1 – Significant Regional Faults

Fault Name	Approximate Distance" (km)
Puente Hills Blind Thrust (Los Angeles)	2.1
Newport-Inglewood	5.7
Puente Hills Blind Thrust	7.5
Elysian Park (Upper)	11.3
Puente Hills Blind Thrust (Santa Fe Springs)	13.0
Santa Monica	15.4
Hollywood	16.4
Raymond	17.4
Elsinore	19.7
Palos Verdes	20.3
Verdugo	20.6
Puente Hills (Coyote Hills)	21.6

* Defined as the closest distance to projection of rupture area along fault trace.

The site does not lie within an Alquist-Priolo Earthquake Fault Zone as designated by the California Geological Survey (Hart, 1997) or as shown on Figure D-5, Seismic Hazard Map. Surface faults have not been mapped projecting towards or through the site area.

Brief details for some of the faults closest to the subject site are as follows:

Puente Hills Blind Thrust

The Puente Hills Blind Thrust (Shaw, 1999) is a north dipping blind thrust extending from the Santa Fe anticline northward to the Montebello anticline. Movement on the fault is responsible for the 1987 Whittier Narrows earthquake. Research on the earthquake and its aftershocks, as well as fault plane reflections, have resulted in the conclusions that the fault is located between 3 and 7 kilometers below sea level. Data on the slip rate and possible recurrence intervals are still being researched.

Newport-Inglewood Fault

The Newport-Inglewood Fault forms the southwesterly side of the Los Angeles Basin and is defined by a series of low disconnected hills and mesa surfaces. Strike slip faulting is associated with anticlinal folding. This has resulted in the accumulation of petroleum resources along its entire length from offshore Newport Beach to the

Santa Monica Mountains. In 1933 the destructive Long Beach Earthquake occurred on the fault just offshore of Newport Beach. The event caused considerable damage and a high loss of life. Since then the various strands of the fault have produced many minor earthquakes, all of which have been at a magnitude of 4.5 or less. The fault lies at a distance of approximately 5.7 kilometers to the southwest of the project sites at its closest approach. A maximum earthquake magnitude of 6.9 and slip rate of 1.0 mm/yr has been assigned to the fault.

Elysian Park Blind Thrust

The north to south structural convergence in the region is a result of deep-seated fault movement along features called "blind thrusts". These are buried low angle north and some south dipping faults which do not project to the ground surface but cause uplift by folding during major earthquakes. In 1987, the magnitude 5.9 Whittier Narrows Earthquake occurred on a previously unknown blind thrust, which has now been given the name Elysian Park Blind Thrust or Structural Zone. This fault underlies the Elysian Park Hills at 3 km and deepens northward to 10 km of depth. Because of the 1987 event, the fault has been placed into an active category and has been tentatively mapped to underlie a major portion of the eastern Los Angeles Basin and adjacent San Gabriel Valley to the north. Subsequent to this earthquake was the 1994 M6.7 Northridge Earthquake in the San Fernando Valley. This earthquake occurred along a previously unknown similar blind thrust fault. This type of active faulting and resulting earthquake activity are considered relatively common in regions undergoing convergence. The Elysian Park Thrust has a length of 34 km, slip rate of 1.50 mm/yr and is capable of generating a maximum earthquake of M6.7 (Shaw and Suppe, 1996).

SEISMIC EXPOSURE

As is the case with most locations in Southern California, the subject site is located in a region that is characterized by moderate to high seismic activity. The project site and vicinity has experienced strong ground shaking due to earthquakes in historic time. The locations of earthquake epicenters with respect to the subject site are shown graphically on Figure D-4, Regional Seismicity.

SECONDARY SEISMIC EFFECTS

General

Secondary effects of seismic activity normally considered as possible hazards to a particular site include several types of ground failure as well as induced flooding. Various types of ground failures, which might occur as a consequence of severe ground shaking of a site include landsliding, ground subsidence, ground lurching, shallow ground rupture and liquefaction. The probability of occurrence of each type of ground failure depends on the severity of the earthquake, distance from faults, topography, subsoils and groundwater conditions, in addition to other factors. Based on a review of available published literature, landsliding, ground subsidence, ground lurching and shallow ground ruptures are considered unlikely at the site.

Various types of seismically induced flooding, which may be considered as potential hazards to a particular site, include flooding due to a tsunami (seismic sea wave), a seiche, or failure of a major water retention structure upstream of the project. Since the site is located approximately 10½ miles inland from the Pacific Ocean at an elevation of approximately 140 feet above mean sea level, and since it does not lie in close proximity to an enclosed body of water, the probability of flooding due to a tsunami or seiche is considered to be nonexistent.

Liquefaction Considerations

Loosely compacted/deposited granular soils located below the water table can fail through the process of liquefaction during strong earthquake-induced ground shaking. In this process, there is a rapid decrease in shearing resistance of cohesionless soils, caused by a temporary increase in the pore water pressure. Factors known to influence liquefaction potential include soil type and depth, grain size, relative density, ground-water level, degree of saturation, and both intensity and duration of ground shaking.

As a result of liquefaction, a typical building structure may be exposed to several hazards, including liquefaction-induced settlement, foundation bearing failure, and lateral displacement or lateral spreading. The surface manifestation of liquefaction in deeper soil deposits often takes place in the form of sand boils and ground subsidence. Such phenomena often lead to loss of adequate support for building foundations (bearing failures) and cause tilting, excessive movement and cracking of superstructures. The severity of ground subsidence depends largely on the relative thickness of the surficial non-liquefiable layer compared to the thickness of layers undergoing liquefaction.

According to the published State Seismic Hazard Zones map for the Los Angeles Quadrangle, the site is located in an area designated by the State Geologist as a "zone of required investigation" due to the potential for earthquake-induced liquefaction. Details of the liquefaction potential in the vicinity of the site are shown on Figure D-5, Seismic Hazard Map. For details and results on our liquefaction and seismic settlement evaluation, refer to Section 4.2.3 in the text of our report.

SUMMARY OF GEOLOGIC CONSTRAINTS

Based on the results of our geotechnical investigation and a review of the information provided in the referenced literature, it is recommended that the following geologic constraints be taken into account during the initial planning stages of the proposed development.

- The subject site is located in a seismically active area of southern California. The type and magnitude of seismic hazards that may affect the site are dependent on both the distance to causative faults and the intensity and duration of the seismic event. The subject site will likely experience strong ground shaking caused by earthquakes on active, regional faults in the future. The proposed project should be designed and constructed in accordance with the seismic design parameters provided in the building code and our final geotechnical investigation report.

- Faults have not been mapped projecting towards or through the site.

- The site is located in an area designated by the State Geologist as a "zone of required investigation" for liquefaction potential. Details of our liquefaction and seismic settlement evaluation are presented in Section 4.2.3 of the report. Based on this analysis, the anticipated liquefaction-induced seismic settlement at the site is on the order of 1 to 1¼-inches. Therefore, the potential for liquefaction settlements to negatively impact the proposed site modifications is considered to be moderate.

- Based on a review of available published literature, landsliding, ground subsidence, ground lurching and shallow ground rupture are considered unlikely at the site.

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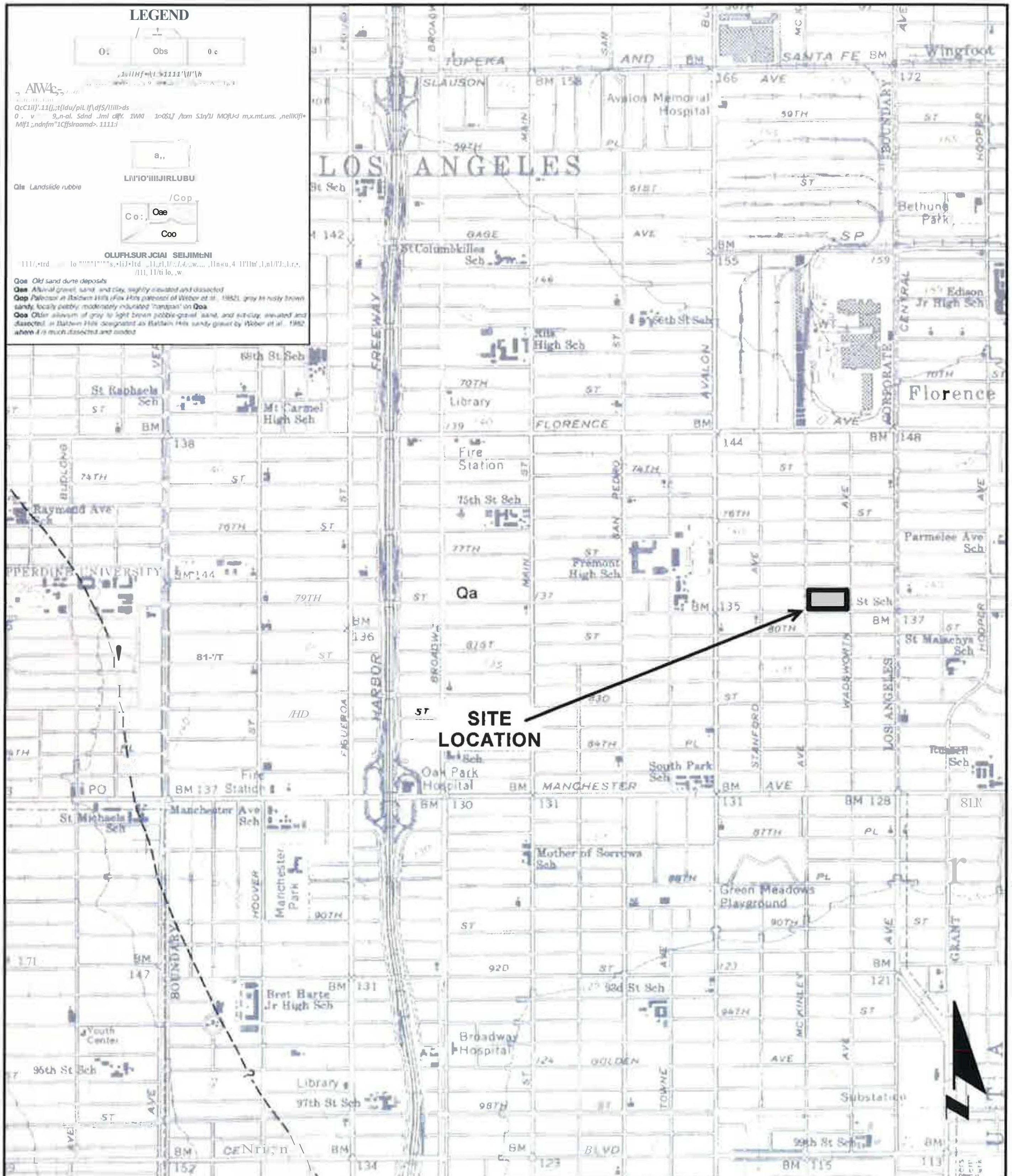


BASE MAP REPRODUCED FROM THE GEOLOGIC MAP OF THE VENICE AND INGLEWOOD QUADRANGLES PREPARED BY THE DIBBLEE GEOLOGICAL FOUNDATION: DATED 2007

GEOTECHNICAL PROFESSIONALS, INC.

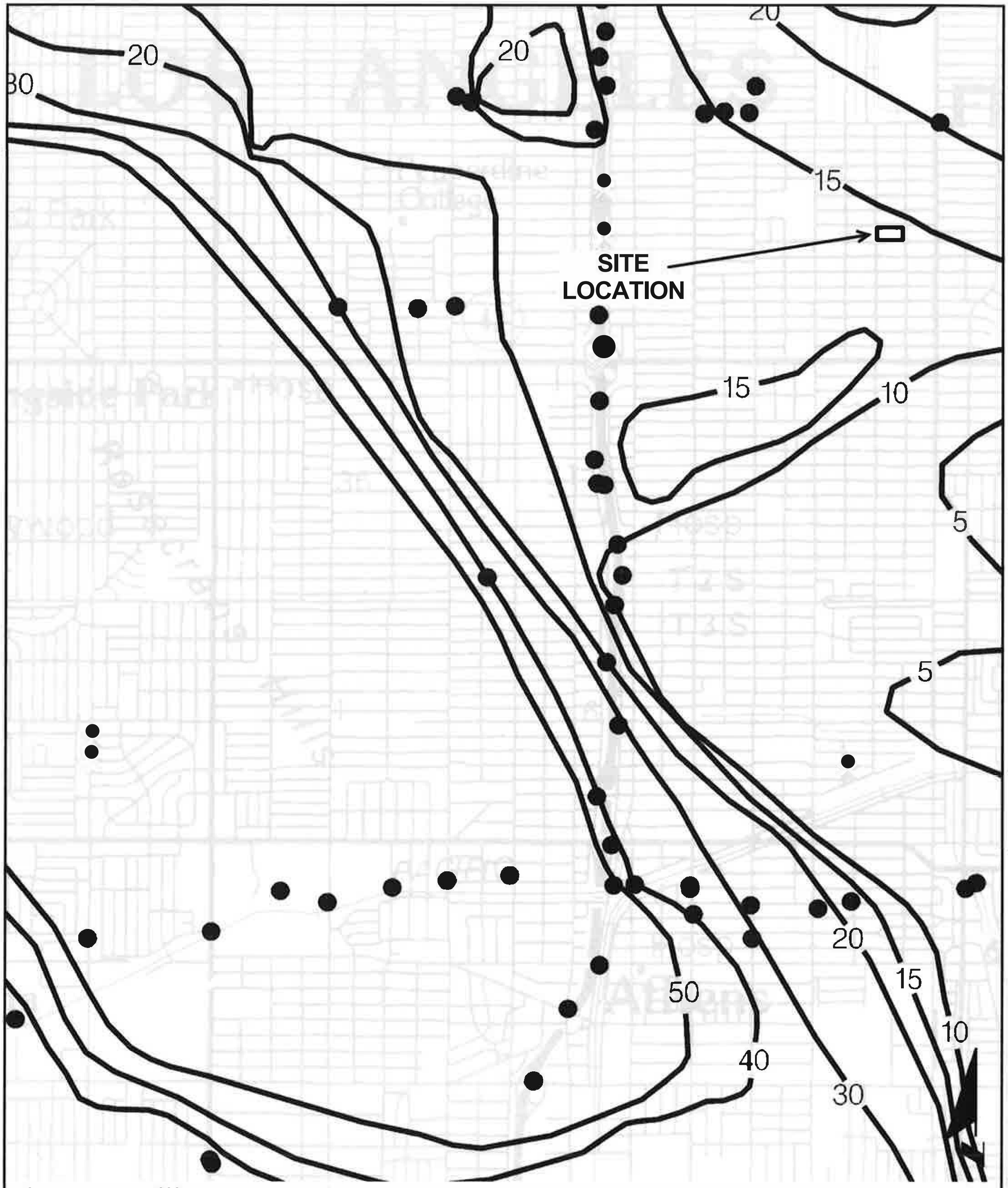
MCKINLEY AVENUE ELEMENTARY SCHOOL

GPI PROJECT NO.: 2677.181

SCALE: 1" = 2000'

REGIONAL GEOLOGIC MAP

FIGURE D-1



BASE MAP REPRODUCED FROM HISTORICAL GROUNDWATER MAP (PLATE 1.2) FROM SEISMIC HAZARD ZONE REPORT 027 (1998)

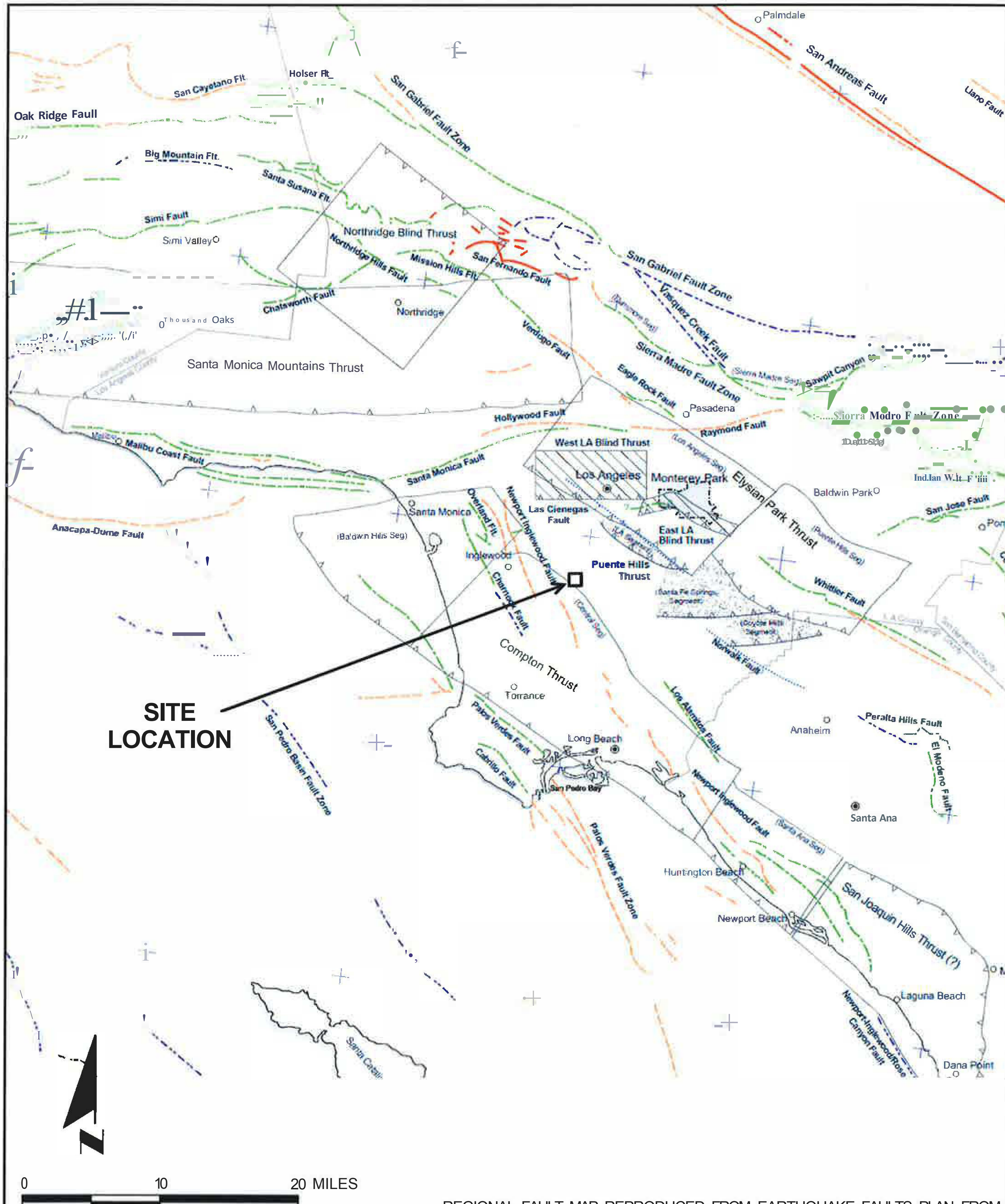
GEOTECHNICAL PROFESSIONALS, INC.

MCKINLEY AVENUE ELEMENTARY SCHOOL

GPI PROJECT NO. 2677.181	SCALE: 1" = 4000'

GROUNDWATER MAP

FIGURE D-2



REGIONAL FAULT MAP REPRODUCED FROM EARTHQUAKE FAULTS PLAN FROM MONTEREY PARK GENERAL PLAN BY BING YEN &ASSOCIATES, INC.: DATED JULY 2001

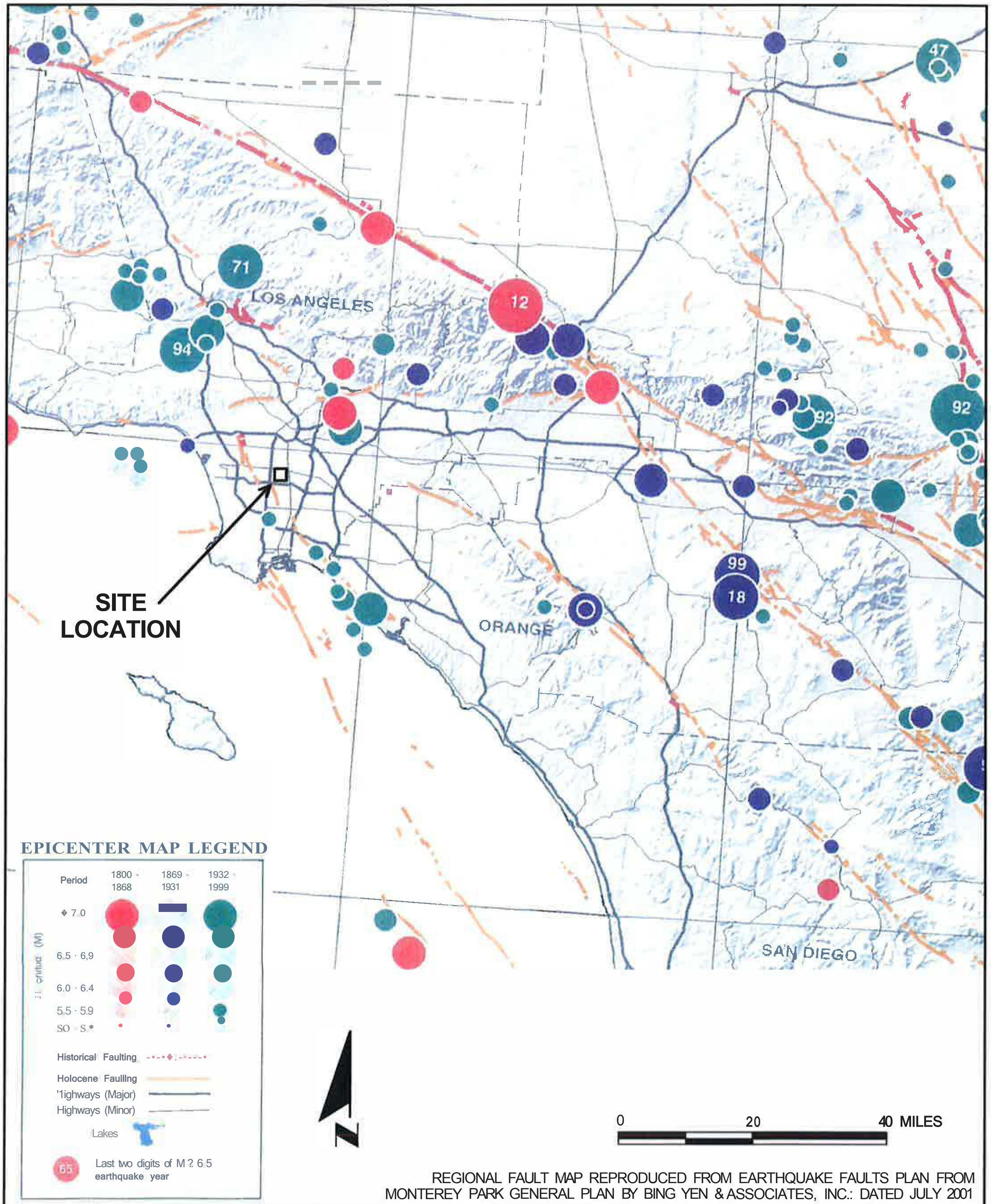
GEOTECHNICAL PROFESSIONALS, INC.

MCKINLEY AVENUE ELEMENTARY SCHOOL

GPI PROJECT NO. 2677.181	SCALE: 1" = 10 MILES

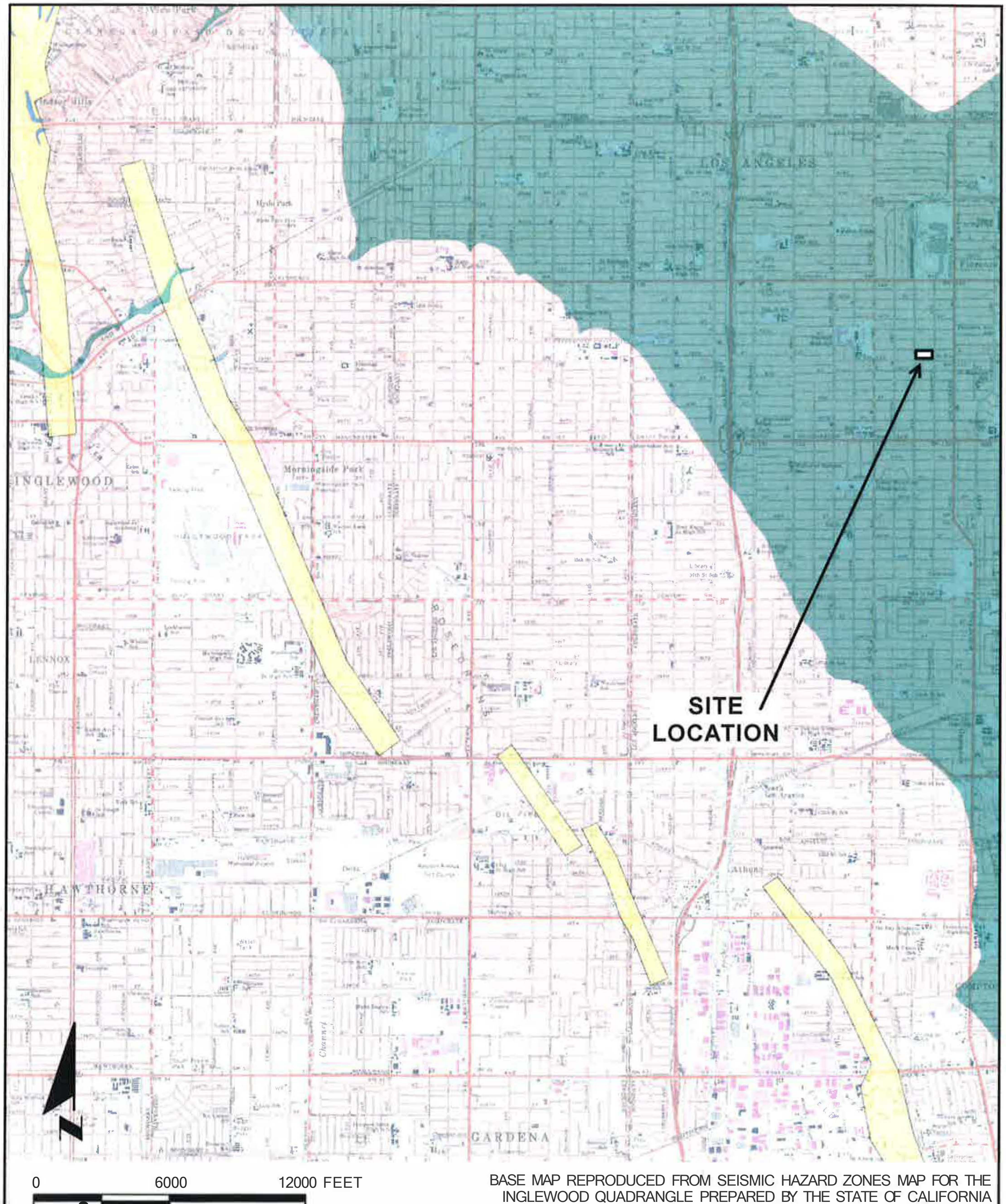
REGIONAL FAULT MAP

FIGURE D-3



REGIONAL FAULT MAP REPRODUCED FROM EARTHQUAKE FAULTS PLAN FROM MONTEREY PARK GENERAL PLAN BY BING YEN & ASSOCIATES, INC.: DATED JULY 2001

GEOTECHNICAL PROFESSIONALS, INC.	REGIONAL SEISMICITY
MCKINLEY AVENUE ELEMENTARY SCHOOL	
GPI PROJECT NO. 2677.181 — SCALE: 1" = 20 MILES	FIGURE D-4



BASE MAP REPRODUCED FROM SEISMIC HAZARD ZONES MAP FOR THE INGLEWOOD QUADRANGLE PREPARED BY THE STATE OF CALIFORNIA DIVISION OF MINES AND GEOLOGY: DATED 03/25/1999

GEOTECHNICAL PROFESSIONALS, INC.

MCKINLEY AVENUE ELEMENTARY SCHOOL

GPI PROJECT NO. 2677.181	SCALE: 1" = 6000'

SEISMIC HAZARD MAP

FIGURE D-5